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FIELD STUDY OF PILE AXIAL CYCLIC DEGRADATION
AND AGING

BY
TIMOTHY KEEFE

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
MASTER OF SCIENCE
IN
CIVIL ENGINEERING

UNIVERSITY OF RHODE ISLAND

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MASTER OF SCIENCE

OF

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2020

ABSTRACT

With the emergence of the offshore wind energy sector in the United States with the Block Island Wind Farm, and future commitments to offshore wind development, understanding of the impacts the environmental loads have on these structures is important in predicting their performance. In the case of offshore wind structures with pile-supported jacket systems, such as at the Block Island Wind Farm, the wind, wave, and current loads cause overturning moments that are resisted by the axial capacity of the supporting piles.

These environmental loads, present in the form of cyclic loads varying in magnitude with time, are hypothesized to cause a loss of capacity of the supporting piles through a process known as cyclic degradation. Another factor to consider is the loss of expected pile capacity during installation due to disturbances in the soil strength in a process known as friction fatigue. However, after installation, piles are known to increase in capacity with time due to pile-soil shear band setup and aging. The objective of this investigation is to evaluate the effects of these factors on model piles driven in a coastal environment, and while subjected to cyclic axial loads on the order of 10^4 0.125 Hz frequency cycles, similar to 0.1 Hz frequency that would be placed on a wind turbine from wave loading (Jardine et al., 2012). The research conducted in this study is part of a larger study involving the United States' Bureau of Ocean Energy Management (BOEM), the Norwegian Geotechnical Institute (NGI), and the University of Texas at Austin.

In this investigation, a hydraulic load control system was developed, and paired with a standard tensile load frame setup to apply monotonic and cyclic loads. The testing

schedule consisted two sets of three pile tests, approximately one-week and ten-weeks after installation, to evaluate the effects of friction fatigue, setup, and cyclic degradation.

It was found during the one-week (short-term) testing that the piles may have encountered “smooth” interface-soil conditions, aligning with maintained post-cyclic monotonic capacities between sets of tests. Furthermore, interface roughness testing on a pile installed 39 days at the site showed considerable increases in roughness in the intertidal zone, which may have produced some “rough” dilative post-cyclic monotonic performance.

In terms of capacities, one-week strength results determined that the test piles surprisingly presented a pure monotonic capacity an order of magnitude lower than predicted amounts from CPT based correlations, most likely due to installation friction fatigue effects. Low-level cyclic loading at a similar time after installation determined a post-cyclic monotonic capacity of three times higher than the pure monotonic capacity, and outside the realm of site variability. This leads to the conclusion that cyclic loading after significant amounts of friction fatigue causes a densification of the shear band towards minimum void ratio levels, producing a stronger dilative post-cyclic monotonic response.

Taking into account the aging response, the pure monotonic long term (ten-week) test results show that the capacity of the site piles increased along a linear scale with respect to logarithmic time. In terms of long-term cyclic test results, the rate of capacity increase over monotonic strength on nearby piles was much lower, indicating aging strengthened the soil-pile system in a manner similar to short-term cycling. It was determined that the maximum rate of capacity increase for the long-term cyclic tests

were similar, whether the ultimate loads were presented in cyclic or post-cyclic monotonic form. No cyclic degradation of capacity was noted throughout the tests, except when mobilized displacements had been already produced cyclically.

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In terms of technical advice on the project, I would like to thank Dr. James Schneider for his many discussions of results received on the project, as well as for his suggestions provided during test implementation. I would also like to thank the sponsors

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understand that it is one thing to think I know a subject myself, but to explain it concisely is a whole different skill. I would separately like to thank my friends Hunter Jarvis and Alex Dickie; our late night study sessions, Friday nights, and times living on our own were some of my best experiences from college.

As my time here at URI comes to a close and I move into the “real world”, I would like to reflect on the opportunities URI has provided. Fresh from high school, my thoughts of URI were those that my gym teacher always would always opine “if you go to URI, it will be just like high school all over again”. Truth be told, my experience could not have been further from that. I have met people from everywhere, all walks of life, from up the street, to across the world. URI has provided me the experience of making new friends, travelling outside of this little state I call home, learning and teaching with world-class professors, taking part in global research, while still being an hour from my home. Some may think nothing more of URI’s slogan, “Think Big, We Do”, but to me, it could not have been truer.

In a closing thought, I distinctly remember Dr. Baxter jesting me after helping with a side project, “your whole time here you have been at the wrong place at the wrong time” and the following figure from this project may support that:

For me, this whole experience has been the “*right* place, at the *right* time”.

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CHAPTER 1-INTRODUCTION

In the last several years, offshore structure development in the United States has expanded into the wind energy sector with the construction of the Block Island Wind Farm off the coast of Block Island, Rhode Island. These turbines generate 30 MW of electric power through five turbines (Orsted Offshore North America, 2019). Into the future, the need for renewable energy will increase. As of 2019, the U.S. state-level commitment for offshore wind turbine development totals 20,000 MW of capacity by 2035, in thirty locations along the Atlantic Ocean seaboard alone (NREL, 2019). Figure 1 details these locations:

Figure 1: Proposed offshore wind development on the Atlantic Ocean Coast of the United States (NREL, 2019)

Of the design challenges presented by offshore structures for engineers, the foundation systems supporting the turbines are one of the areas of focus. In Europe, where the offshore wind energy sector is more developed, several types of structures exist to connect turbines to the seabed. These types are largely chosen based on seabed depths and soil characteristics. Figure 2 details these different types:

Figure 2: Typical offshore wind structures and pile foundations implemented in Europe (O'Kelly and Arshad, 2016)

The turbines located at Block Island are supported by a lattice support jacket structure system (e.g. Figure 2e), each consisting of four legs through which piles are driven into the seabed. The use of the jacket structures in that location was likely the result of prior experience in the oil and gas industry developing these structures in the Gulf of Mexico.

It should be noted that even though the jacket structure systems may have been developed based on oil rig foundation systems, there are inherent differences in the loading imparted by the two structure types. In addition to the dead loads of the turbine itself, foundation systems are required to support significant wind, wave, and sea current

loads (Dean, 2010). These loads vary in magnitude cyclically, as the wind, wave and sea current action is cyclic in nature. As the design life for offshore wind structures is upwards of thirty years, millions of high magnitude cycles of each of these loads will be transferred into the seabed, which will need to be supported by the skin friction offered by the foundation piles.

These wind, wave and sea loads are also notably smaller in magnitude than the loads present on oil and gas structures. This is because offshore wind turbines are much lighter and smaller than oil and gas platforms. Typical wind and wave loads on a 3MW wind turbine vary from 0.1 to 1 MN, and 1 to 3 MN, respectively, whereas for an oil and gas platform, the horizontal loads vary from 50 to 100 MN (Schneider and Senders, 2010). However, these loads on offshore wind structures create large moments since the slenderness of a wind turbine concentrates the loads higher. The relative locations and magnitudes of these loads are detailed in Figure 3, in comparison to oil and gas platforms:

Figure 3: Magnitude and location of (a) oil and gas platform loads compared with (c) wind turbine loads (Schneider and Senders, 2010).

Thus, to be able to effectively design foundation systems to support wind turbines, accurate predictions of cyclic load effects on piles are necessary, so that accurate predictions of capacity and performance (e.g., movement) can be made.

There are several components and effects of cyclic loading that require accounting for in the design of offshore structure piles, to be described in detail later. Referencing the timeline of installation of piles for a structure, these effects include; friction fatigue, which decreases soil strength during pile installation, followed by changes in pile capacity due to cyclic environmental loading (known as cyclic degradation). These cyclic environmental loads will cause an accumulation of displacements if large enough in magnitude (known as hysteresis loops). Piles are also known to witness capacity increases with time due to the set-up of radial soil around the pile (known as aging).

Of these components, cyclic degradation and aging effects on piles are being investigated at the University of Rhode Island as part of a broader study funded by the U.S. Bureau of Ocean Energy Management (BOEM), and in partnership with the Norwegian Geotechnical Institute, and University of Texas at Austin. Research at the University of Rhode Island involves the installation and testing of reduced-scale model piles in a coastal environment to evaluate these cyclic effects.

As such, the objectives of this thesis are to discuss the design, implementation, approaches, results, and findings of pile load tests undertaken at the University of Rhode Island at the Allen Harbor Educational Facility.

Chapter 2 of this thesis will introduce the effects of cyclic loading on pile capacity. Chapters 3 and 4 will discuss soil investigations and soil characteristics at the University of Rhode Island Allen Harbor site where the investigation occurred, as well

as predicted capacities of the reduced-scale piles installed at the site. Chapter 5 will introduce the load testing equipment implemented in this study, followed by Chapter 6 detailing pile characteristics and installation details. Chapter 7 provides the results of the pile load tests. Chapters 8 and 9 will then detail the interpretations of the load tests, as well as provide conclusions of the study.

CHAPTER 2-REVIEW OF THE LITERATURE

This chapter presents a review of the literature immediately relevant to the study. Detailed are sections on the pile-soil interaction mechanics, and relevant factors that have impacts on short-term and long-term capacity of installed piles.

2.1 Loads on Jacket Piles

As noted previously, loads from wind turbines consist of the weight of the turbine itself, as well as wind, wave, and sea current loads (Dean, 2010). These loads are presented in horizontal loads on the structure above the piles, creating moments at the mudline level, as well as horizontal and vertical loads on the supporting piles. Figure 4 details the typical loading conditions an offshore wind turbine generates on a monopile supported structure. The reactions of a monopile are different than those of a piled jacket structure, however the environmental load cases are similar.

Figure 4: Typical offshore wind structure loading conditions (O’Kelly and Arshad, 2016)

The loads in Figure 4 create vertical, horizontal, and moment reactions on the supporting piles. For the horizontal reactions on wind jacket structure piles, the

surrounding soil provides resistance through confinement and subsequent strength of the material in the radial area around the pile. These lateral reactions are often well-distributed due to the expanse and depth of mobilized soil in the radial direction. The axial reactions, as detailed in Figure 5, are much more critical to the stability of the wind structure, as the only vertical support the piles receive are through soil-pile shaft resistance (also known as skin friction), as well as end-bearing resistance provided at the base of the pile by the soil beneath it, if the load is compressive.

Figure 5: Axial pile reactions for wind jacket structures (Bradshaw and Coulson, 2018).

In the tensile axial loading direction, the tip of the pile offers no resistance, as the loading direction is away from the bearing soil, and the shaft resistance is the only component supporting the structure. This tensile shaft resistance is one of the critical design aspects to consider (Jardine et al., 2012), as decreases in such may cause structure serviceability failures.

Natural cyclic loads on offshore structures are known to be irregular in occurrence with time, due to variations in the environment. Figure 6 provides a natural variation in cyclic loads for three piles of a tripod supported wind turbine structure:

Figure 6: Natural variation in environmental load cycles with time on the three piles of a tripod offshore wind structure (Puech and Garnier, 2017)

These variable cycles are then grouped together into similar “packets” of loads with time, which can then be converted into a repeating equivalent cyclic load through the use of weighted averaging of the packets with respect to time. This process is shown in Figure 7:

Figure 7: Conversion of natural cyclic loads to equivalent cyclic loads for analysis (Puech and Garnier, 2017)

The equivalent cyclic loads displayed in Figure 7 allow model tests to be setup under regularly occurring loads that are similar to the variable loads encountered in marine environments.

2.2 Friction Fatigue and Installation Methods

During pile installation, the strength a soil mass can provide a pile is severely decreased. This is known as friction fatigue and begins as a pile is advanced through a soil mass. Figure 8 presents the stresses a pile encounters during and upon completion of installation when a jacking type method is implemented.

Figure 8: Stress conditions encountered by sandy soil around a pile during installation and loading (White and Deeks, 2007).

As Figure 8, point A details, a soil is initially presented with a vertical effective stress from the soil above, and a horizontal stress due to the vertical stress. Thus, the soil has some natural value of shear and normal stress (y and x-axis of Figure 8, respectively). As the installed pile approaches the soil element from above, the vertical and horizontal stresses increase significantly and begin to fail the soil, as seen by point

B. As the pile reaches the soil mass and is driven through, the soil begins to “flow” around the pile tip, relieving some of the overburden stress encountered at point B, creating point C, though still at failure. The flow of soil around the pile and the mechanics of the shear band developed is seen in Figure 9:

Figure 9: Flow of soil around pile tip during installation, and impacts on shear band and far-field soil confinement (White and Deeks, 2007)

In the case of close-ended piles and open-ended piles with compacted soil plugs, the soil around the pile tip will move away from the pile and contract as the pile passes. The confinement presented by the far-field soil will reach a maximum just as the pile tip approaches the level of the soil element in question, and decreases afterwards as the pile slides by.

This loss of capacity is known as “friction fatigue”, to be described in detail after. As the pile is advanced passed the soil mass at point D, further driving cycles subject the soil to rapidly increasing and decreasing shear and normal stresses, with a degradation of strength as the pile advances further. In the case of tensile and

compressive “two-way” driving cycles, used to avoid creation of a soil plug in open-ended piles (White and Deeks, 2007), negative shear stresses are created, moving the shear stresses below the x-axis. After termination of driving at point E, driving stresses reduce to zero, leaving a decreased normal stress and shear stress due to unloading. When the wind turbine structure is constructed upon the pile, a static compressive load from the dead weight of the structure is created, mobilizing the soil’s resistance to point F, with cyclic environmental loading causing increases and decreases in the shear and normal stress along the dashed line from point F.

Briefly discussed in Figure 8, Figure 10 details the effects of friction fatigue on radial stress with distance from the pile shaft as a pile is installed. The performance was determined through model soil-pile testing during installation with Particle Image Velocimetry (PIV) analysis of soil displacement (White and Bolton, 2004).

Figure 10: Radial stress on pile during installation from friction fatigue and subsequent set-up (White and Bolton, 2004)

In Figure 10, as the pile advances passed a given soil element, the radial stress increases significantly, and decreases with increasing radial distance from the pile shaft (represented in line AO). The soil interface around the pile then contracts, creating a significant decrease in radial stress near the shaft (represented in line BO), with

increased preservation of radial stress further away from the pile. With increasing time however, the large disparity in radial stress from the far-field zone to the interface zone begins to equalize as continued shear band contraction dissipates, leading to creation of line CO, which is known as pile set-up (White and Bolton, 2004). Overall, this pile set-up is known to increase pile capacities over time, and will be discussed later.

As noted in Figure 8, as installation cycles on a pile become more numerous, the soil around the pile degrades further in strength. It should be noted that there are several different methods that can be implemented to install piles, all which have different effects on both short and long-term pile capacity. Some of these methods include: jacking, driving, vibrating, and boring, with driving being the most common type implemented around the world. Pile jacking, as is presented in Figure 8, is where a large hydraulic press “pushes” the pile into the soil. This method is popular in Europe and in closely-developed urban areas due to the avoidance of the significant disturbances attributed to the more traditional high impulse methods, such as driving (White and Deeks, 2007).

The more commonly implemented installation method is driving, in which large weights, known as a hammer, are attached to a track and are released from large heights above the pile being installed. The weights strike the pile and cause it to displace into the soil and is performed until the pile reaches refusal. Each cycle of this action is called a “blow”. As this method provides relatively similar amounts of energy per blow, large numbers of cycles per depth of installation indicate denser soils, and will cause more friction fatigue as more driving cycles are induced.

Various different types of pile installation hammers exist, with driving energy often being related to rig size. In the case of a smaller rig, with smaller hammers, large numbers of cycles are necessary to drive the pile to refusal if dense soil is presented, increasing friction fatigue as cycles become more numerous.

Analysis was performed by Anusic et al. on load tests of model piles installed at the University of Western Australia's (UWA) Shenton Park test site consisting of thirty three piles that were driven through several installation methods, resulting in different amounts of driving cycles induced (2018) per unit shaft area. It was noted that piles requiring more driving cycles per unit area of embedment were determined to have much lower initial capacities. This is detailed in Figure 11:

Figure 11: Tensile capacities of piles installed at the UWA Shenton Park site illustrating the decrease in short-term resistance with higher numbers of driving cycles (Anusic et al., 2018)

As can be seen in Figure 11, for piles with lower numbers of driving cycles incurred, higher short-term tensile strength is presented than in those requiring more driving cycles to reach embedment. It should be noted that Figure 11 represents tensile strength as an averaged shear strength unit on the y-axis, as the tensile capacity of a pile

is due to shear resistances in the shear band. Jacked piles, which incur no driving blows, since a “pressing” installation method is involved, were noted to have the largest initial capacity, as shown by data points SP24 and SP25, as the soil in the shear band has less fatigue.

2.3 Cyclic Load Performance and Displacement Rates

When cyclic service loads of significant magnitude are placed on piles, displacements occur, albeit at rates based on pile-soil strength. In all cases, a general load-displacement curve can be produced plotting the variation in load as the y-axis variable and the displacement as the x-axis variable. A typical cyclic load-displacement plot is presented in Figure 12, where the load and displacement are both measured on the pile head:

Figure 12: A typical Pile-head load-displacement plot detailing load levels and displacement conditions throughout testing (Rimoy et al., 2013).

As detailed in Figure 12, as a cyclic load is applied to a pile, the load increases. With increases in load, increased displacement of the pile occurs. Once the peak cyclic load ($Q_{\max}$) is achieved, a peak in displacement is noted, and the load then decreases to

the minimum cyclic load ($Q_{\min}$), where some recovery in displacement occurs as the pile is released. The load then increases to complete one cycle. In the distribution of data produced, displacements following the cyclic nature of the loading are termed a “hysteresis loop”.

During loading, as cycles continue to occur, piles encounter limited recovery levels, with further cycles causing continued displacements. These limited recovery levels are indicated in Figure 12 by the two stiffness values provided, the loading cyclic stiffness (k_l) and the unloading stiffness (k_u). The unloading stiffness is significantly less than the loading stiffness if the same cycle is referenced, leading to some permanent displacement since the full cyclic load is reduced, but less displacement is recovered. As such, permanent displacements occur with each cycle, labelled as offset “a” in Figure 12, called accumulated displacement. In the case of sustained numbers of cycles, the hysteresis loops shift further along the displacement axis, and both the loading and unloading stiffness decrease (however never becoming equivalent) as the pile becomes mobilized.

As noted, accumulating displacements occur in piles subject to constant cyclic loads. However, in true environmental loading cases, such as hurricanes and larger storms, few cyclic loads actually occur of the exact same magnitude. With this being said, cyclic loads may increase or decrease, or be sustained for longer periods of time. Thus, as cyclic loads deviate, changes in how displacements occur are necessary to understand.

Monotonic and cyclic load tests performed by Puech et al. in silty clays and loose sands in Plancoet, France, at different cyclic load ratios (cyclic axial load with respect

to capacity) produced the following data as determined in Figure 13 (Jardine et al. 2012), in regards to rates of displacement under changing cyclic loads.

Figure 13: Cyclic pile head displacement rates versus axial load on a pile driven in silty clays and loose sands (Puech et al., 1982)(Jardine et al., 2012)

As is detailed in Figure 13, under lower cyclic and monotonic (static) load levels, the displacement rates that occur proceed linearly in magnitude and remain relatively stable. Once a certain load level is reached however, the pile head displacement rate increases towards failure at a much larger slope. It can be said that a pile in this larger magnitude loading range is displaying an unstable type of behavior. Determining this load level for piles is critical in predicting when failure will occur.

2.4 Pile Interface Conditions and Constant Normal Stiffness Test Results

Cyclic soil-pile material loading can be evaluated in the laboratory using interface friction tests. These tests often note a contraction of the shear band known as “cyclic degradation”. Cyclic degradation is the loss of pile capacity due to the reduction of interface shear strength that occurs as the soil shear band against the interface contracts volume and subsequently reduces the normal stress acting on the pile (DeJong et al., 2006). As previously noted, this loss of capacity will cause piles to fail under lower monotonic loads, or in the case of sustained large magnitude cyclic loads, in a low

number of cycles. This amount of shear band thickness change is highly correlated to the roughness of the pile interface material in contact with the shear band. Figure 14 details the results of pile roughness on normal stress (radial stress) surrounding the pile interface under applied constant normal stiffness as determined in direct shear tests against a series of steel surfaces (Mortara et al., 2007).

Figure 14: Normal stress on soil-pile interface of “rough” interface material, GT40, and “smooth” interface material, GT10 (Mortara et al., 2007)

The tests from Mortara et al. in Figure 14 show that cyclic loading on “smooth” pile interfaces cause a reduction of normal stress on the soil-pile interface under repetitive displacement amounts, the final reduction of which is sustained when the pile is monotonically displaced upon termination of the cyclic load (2007). This is seen in the data for pile material GT10 in Figure 14. As for “rough” interfaces, under similar cyclic displacement amounts, reduction of normal stress occurs, followed by a small recovery with each cycle (shown by material GT40 in Figure 14). This behavior is induced since the shear displacement encompasses both the shear band and more distant soil surrounding the pile, whereas with a “smooth” pile, displacements occur in the shear band. Upon the transition to monotonic displacement for the rough interface, the normal stress the interface experiences greatly increases due to radial soil dilation, and in the

case of Figure 14, is much greater than the initial normal stress (Mortara et al., 2007). Thus, pile material choices impact resistance amounts greatly.

In the constant normal stiffness interface tests previously noted, the roughness of the pile material was loosely defined as “rough” and “smooth”. Such descriptions are vague and open to interpretation, and thus, classification has been developed to determine relative soil-pile system roughness ratios through the use of both soil grain size analysis and pile material roughness testing.

In grain size analysis, coarse-grained soil samples are passed through a stacked series of mesh sieves, of decreasing sizes (gradations). The mass retained above each sieve, subtracted from the whole sample mass, produces a set of values labelled “percent passing”, which can be plotted against sieve size, more commonly labelled as “particle diameter”. From the mean particle percent passing value (50%), a corresponding diameter can be determined known as the mean particle size, D_{50} .

In materials roughness testing, a surface metrology instrument consisting of a needle-like probe is run across the surface of a material, reading the surface profile left by the material’s machining processes. Statistically analyzing the distribution of the results delivered, different parameters can be defined including; the mean roughness over the entire measurement (R_a), the root mean squared roughness (R_q), mean roughness depth between corresponding divisions of measurements (R_z in ISO standards or R_y in JIS standards), and the total height difference from the highest peak to the lowest valley (R_t or R_{max}) (Mitutoyo, 2016).

To compare the soil diameter and pile roughness, a term has been developed known as the “normalized relative roughness” (R_n), in which the total roughness (R_{max}) over

the mean particle diameter (D_{50}) is divided by the mean particle diameter to produce a ratio. The normalized roughness appears as the following equation (Uesugi and Kishida, 1986):

$$R_n = \frac{R_{\max}(L = D_{50})}{D_{50}} \quad (1)$$

This normalized roughness parameter allows site soil and pile specific roughness comparisons to be made. For the study presented by Mortara et al. in Figure 14, soil and material combination GT40, classified as “rough”, has a normalized roughness of 0.157, and combination GT10, classified as “smooth”, has a normalized roughness of 0.030 (2007). Figure 15 depicts a cross-sectional view of a “rough” combination (a), and a “smooth” combination (b):

Figure 15: Cross-section of: (a) "rough" soil-pile interface, (b) "smooth" soil-pile interface (Mortara et al., 2007).

In Figure 15(a), where the rough interface is presented, the soil particles are small enough to sit inside the peaks and valleys of the interface material. Thus, during shearing, the particles pivot around the peaks and into the valleys and vice versa, causing both contraction when in the valleys, and dilation, noted as “recovery” of normal stress before, as shearing terminates.

As for Figure 15(b), where the “smooth” interface is presented, the soil particle diameter is much larger than the peak-to-valley volume and the particles sit outside of the pile. As shearing occurs, the particles more easily slide over the interface material, leading to a continual contraction of the shear band with each loading cycle.

2.5 Pile Aging

It was noted before that an increase in pile capacity related to pile set-up is a phenomena known as pile “aging”. Pile aging consists of an increase in pile capacity over time due to a combination of altered stress states with time around the pile, changes in soil properties to produce a more dilative response, and chemical reactions of the pile to produce a rougher interface (Gavin et al., 2013). Gavin et al. implemented a series of static and cyclic load tests on open-ended piles after installation, which noted increases in initial capacities immediately following installation of 85% at 12 days and 185% after 220 days (2013). Capacity tests at different time intervals can be used to produce a trend line of aging response labelled the “Intact Aging Characterization” line, an example of which is shown in Figure 16.

Figure 16: Plot of an example Intact Aging Characterization line from static pile capacity tests compared to the ICP prediction method (Gavin et al., 2013).

When plotted with respect to logarithmic time, the increase in capacity with increasing time appears linear in relation.

2.6 Pile Design

One of the first steps in the design process for pile structures is the estimation of the ultimate axial monotonic capacity of the pile. This is accomplished through static capacity analysis and there are a number of models that have been developed for offshore applications. These include the American Petroleum Institute design method (also known as the API method), the UWA-05 method, the NGI-05 method, the ICP-05 method, and the Fugro-05 method (Randolph and Gourvenec, 2011). The API method is based on the soil-pile shaft resistance solely being a factor of the vertical soil effective stress and the friction generated between the soil and pile and is formatted as follows:

$$\tau = \sigma_{v0}' K \tan(\delta) \quad (2)$$

Where τ = soil-pile shaft friction (shear resistance), σ_{v0}' = vertical effective stress of the surrounding soil at the full pile length, K = horizontal earth pressure coefficient of the soil, and δ = soil-pile interface friction angle (API, 2007).

The other four methods presented use similar stress factors as the API method, however are based on in-situ Cone Penetration Test (CPT) tip resistance results and include resistance components. These components reflect mobilized soil-pile resistance terms such as shear band dilation or contraction during static loading, and friction fatigue during pile installation (Randolph and Gourvenec, 2011) (Lehane et al., 2005).

Looking at the performance of soil surrounding a vertically loaded pile, resistance is offered to the pile by the horizontal stiffness of the radial soil, as indicated by the far-field stiffness in Figure 17, quantified as the quotient of four times the soil's shear

modulus, (G), and the pile diameter, (D). As the loading causes the pile to displace from the original embedment depth, a thin band of soil surrounding the pile wall, known as the “shear band” begins to contract or dilate in thickness (Lehane and White, 2004). Figure 17 details this process. The shear band is called the “interface zone thickness” in Figure 17, and displaces by Δt .

Figure 17: Soil-pile shear band performance during loading (Lehane and White, 2004)

The shear band along the pile, with a thickness of five to ten soil particle diameters, provides most of the axial resistance a pile experiences (DeJong et al., 2006). In the case of particle dilation, the soil within the shear band increases in volume, compressing the soil spring and increasing the horizontal stress, and thus, capacity of the pile. However, in the case of contraction of the shear band surrounding the pile under loading, the shear band volume reduces and the normal stress is reduced, causing a decrease in friction and pile capacity.

As previously noted, the five pile design methods evaluate pile capacity under static loading. In the case of offshore wind structures however, this loading appears in cyclic form, as the critical loadings wind turbines experience; current, wind, and wave, are cyclic in nature. Thus, there are processes to evaluate the effects of cyclic loading on pile capacity. One primary method of determining the effects of cyclic loading are

through the use of cyclic stability diagrams, as detailed in the following section. Another method, proposed by Jardine et al., involves the use of cyclic stability diagrams to determine an initial design, with a series of in-situ, laboratory, and previous experimental data, used to evaluate cyclic degradation (2012). A flowchart of this design process is provided in Figure 18:

Figure 18: Cyclic loading flowchart, involving laboratory tests to prove cyclic degradation effects (Jardine et al. 2012).

This process outlined in Figure 18 will not be discussed at length in this study, however provides a visualization of the components of the cyclic design process discussed throughout.

2.6 Cyclic Stability Diagrams

The previously mentioned components of pile strength design mainly focus on sustained loading conditions with time. The factors affecting cyclic pile capacity response are complex and detailed analyses presented prior are cumbersome for simple pile installations. In-situ cyclic testing of design loads on installed piles is quite

expensive, and thus, stability diagrams have been developed to incorporate the results of previous pile tests for new piles to be designed with cyclic service loads considered (Poulos, 1988). Looking at the range of cyclic loads with time, key magnitudes can be determined, as seen in Figure 19, that are necessary to define a stability diagram.

Figure 19: Typical cyclic load performance with time used in cyclic stability diagrams (Tsuha et al., 2012).

Important magnitudes are Q_{max} , the maximum cyclic load experienced and Q_{min} , the minimum cyclic load experienced. Q_{mean} is the average of the maximum and minimum loads, and Q_{cyclic} is the amplitude of the wave. These magnitudes are incorporated into a typical pile stability diagram as shown in Figure 20:

Figure 20: A cyclic stability diagram used to evaluate pile performance. The data points are cyclic load tests and the numbers next to the points are the total cycles applied (Tsuha et al., 2012).

Q_{cyclic} and Q_{mean} are normalized by the total monotonic pile capacity (Q_T), which is either known from pile testing, or estimated using the pile capacity estimation methods. There are sections for one-way loading, which is either tensile or compressive only, and two-way loading, which includes load direction changes. Previous pile load tests have determined performance into three categories: stable, meta-stable, and unstable. Tsuha et al. notes stable loading is where pile displacements do not accumulate significantly over hundreds of cycles, meta-stable performance is where displacements accumulate over tens to hundreds of cycles without stabilization, and unstable cyclic loading is where displacements occur quickly, with significant drops in capacity (2012). The preferred design option is to remain in the stable zone. Typical of most stability diagrams, individual soil conditions and types of piles implemented determine the stability diagram to be used (Tsuha et al., 2012).

2.9 Pile Load Tests

As noted previously, pile load tests provide a method to determine if installed piles meet required in-service strength levels. These tests have become commonplace in the deep foundations industry with the American Society for Testing and Materials (ASTM) producing standard D3689 to describe standard test setups and procedures.

In the case of most load tests, a load frame is developed to place one-way (either compressive or tensile) loads on test piles. The frame allows loads to be applied via a hydraulic jack, with reaction loads consisting of either large weight stacks, in the compressive case, and bearing points on the nearby soil in the tensile case. The frames also allows measurements for load and displacement via transducers. Figure 21 details a typical load frame setup for tensile conditions according to ASTM D3689.

Figure 21: A typical pile load test frame schematic for tensile conditions (ASTM, 2013).

In the general tensile load setup provided by Figure 21, a test pile is connected via a tensile connection rod to a reaction frame that bears against a hydraulic jack. To apply a tensile load, the jack piston move upwards against the reaction frame, and the jack creates a bending moment load on the test beam underneath it. The test beam is then supported by either reaction piles in soft soils, or cribbing blocks in areas where in-situ soils have sufficient bearing capacity. A minimum test beam span of at least five times the pile diameter is necessary to allow the bearing supports to have minimal impacts on the near-pile soil.

2.10 Realm of Study

The research and test methods discussed prior have been analyzed under lower numbers of applied load cycles or in monotonic load cases. Tests performed by Rimoy et al. under one-way loading were applied with cyclic load ratios such that pile failure was reached in one to more than one thousand cycles over several separate pile tests (2013). A 2.0 MW turbine is known to generate around 100 cycles of large 2.0 MN

loads and 10^7 cycles of 1.4 MN loading on the structure over the service life (O’Kelly and Arshad, 2016). In the case of large hurricanes, which often generate the largest magnitude loads seen and last for significant amounts of time, tests for larger numbers of cycles are paramount to determining performance of the supporting foundation.

Prior research in the United States has mainly focused on clayey soil performance as seen in the Gulf of Mexico for jacket structures. Prior research in Europe on sands has avoided large numbers of cycles as strong hurricanes, which create large cycle numbers, are not present in the form as seen in the United States’ offshore areas. The objective of this research is to perform in-situ load tests and analyze the cyclic degradation and aging responses of model piles subjected to 10^4 numbers of cycles and evaluate the Intact Aging Characterization line as well as related monotonic and cyclic capacities and displacement rates under cyclic loading as it applies to sandy soils.

CHAPTER 3-SITE INVESTIGATIONS

The goal of this chapter is to present geotechnical site investigations performed at the University of Rhode Island (URI) testing site. Investigations performed at the URI site include geotechnical borings, Standard Penetration Tests (SPT), and standard and seismic Cone Penetration Tests. An overview of the test site is presented first, followed by methodology and results determined by each of the respective investigations. Supplemental site investigation data is presented in Appendix A.

3.1 URI Allen Harbor Site Overview

The scope of this study will mainly focus on site investigations and pile test results from the URI Allen Harbor Educational Facility in Davisville, North Kingstown, Rhode Island. The site is as located on a state map in Figure 22 and aerial view in Figure 23:

Figure 22: Satellite Map of the State of Rhode Island with location of the URI Allen Harbor test site presented. The yellow line presented is 5 kilometers in length (Google Earth Pro, 2020)

Figure 23: Aerial view of Allen Harbor test site from northwest to southeast (Marinas.com, 2020)

As can be seen in Figure 23, the test site is located in a coastal environment, bordered by Allen Harbor to the west and north, a channel from Allen Harbor to the Narragansett Bay to the north and east (Clams Channel), and Bruce Boyer Road and Allen Harbor Road to the south. The test site sits at the tip of a peninsula, known as Spink Neck. The location is approximately 2.0 kilometers due north of the Quonset State Airport. Figure 24 details a site plan of the property and bounds of the geotechnical investigations and pile installations, as determined by the Rhode Island Geographic Information System (RIGIS, 2019) through statewide LIDAR mapping of elevation contours. A subsequent tracing of surface features, performed by the Author based on scaled aerial imagery, allowed creation of a full site plan.

Figure 24: Site plan of Allen Harbor Educational Facility and surrounding properties

The site of the Allen Harbor Educational Facility is owned by the University of Rhode Island and was previously owned by the federal government. The land encompassing the site is 0.73 hectares (1.8 acres) in area (Vision Government Solutions, 2020). The general topography of the site is relatively flat, varying from sea-level at the northern edge of the site, to about 2.4 meters of elevation towards Bruce Boyer Road and the interior portions. At the northern most tip of the site, at the point of where Clams Channel transitions into Allen Harbor, a significant portion of land exists to provide sufficient area for test pile installation and site investigations. This location of the test site is displayed in Figure 25 and called out in Figure 24:

Figure 25: Site Plan of immediate area where geotechnical investigations and test pile were installed. The locations of these elements are also shown on the plan.

In Figure 25, the locations of the Standard Penetration Test boreholes are presented in yellow, and the Cone Penetration Test locations are presented in black. The SPT boreholes have naming convention “URI” and the CPT locations have conventions “SCPT” and “CPT”. The locations are approximate as they were determined by taping off to identifiable features at the site. The beach area is bounded by the high tide level to the north, and a tree-line to the south, leaving approximately 400 square meters of area for investigations and testing. It should be noted that the site is tidally influenced, with a tidal variation of about a meter on average (NOAA, 2019). Six Standard Penetration Tests were performed in this area, with soil samples recovered, as well as four seismic Cone Penetration Tests (SCPT19-02-SCPT19-04), and one standard Cone

Penetration Test (CPT19-05). Further sections will detail these investigations to a greater level.

3.2 Geotechnical Borings

Geotechnical borings were performed at the site on December 4, 2018 by Hoffman Environmental Services. Six borings were drilled on the site (URI-1 through URI-6 as detailed in Figure 25). The borings were performed with the use of an automatic hammer attached to a track rig as displayed in Figure 26:

Figure 26: Diedrich D50 Turbo Drill rig used to install the geotechnical borings and perform Standard Penetration Testing

The drill rig detailed in Figure 26 is a Diedrich D50 Turbo rig with a hollow stem auger and a 50 mm diameter split spoon sampler. With this setup, Standard Penetration Tests at each borehole were conducted.

3.2 Standard Penetration Tests (SPT)

At each of the geotechnical boreholes drilled at the site, Standard Penetration Tests were performed. Full Standard Penetration Test results are detailed in Appendix A. In each location, a 0.61 meter long split spoon sampler was advanced through each borehole to recover soil samples at 1.5 meter intervals. Blow counts were recorded per every 0.15 meters of sampler advancement. Final depth of collection was between 4.57 and 5.18 meters of depth for boreholes URI-1 through URI-4. Tests performed at URI-5 and URI-6 reached refusal a 0.46 and 0.91 meters below the surface grade, respectively, due to refusal presented by boulders. Of these two tests, only URI-6 produced a recoverable sample.

Samples recovered from testing were placed in 270 mL covered glass jars, labelled for borehole and sample identification, as well a SPT uncorrected blow counts (N-values) and transported to the URI Narragansett Bay Campus for storage. Uncorrected blow count values with depth are presented in Figure 27 for each of the borings drilled at the site.

Figure 27: Uncorrected blow count values with depth for each of the Standard Penetration Tests

As is indicated by comparing the data in Figure 27 with the borehole location in Figure 25, there is a clear increase in soil strength from the western side of the site towards the eastern side when comparing resistances from URI-2, URI-1, URI-3, and URI-4, which proceed from west around the point between Allen Harbor and Clams Channel to east, respectively.

General Burmister Classification of retrieved soil samples indicated loose brown medium sand towards the surface, followed by medium dense silt and sand at depth. The water table was encountered from 0.6 to 1 meter below the surface grade, however, this depth varies as the site is tidally influenced.

3.3 Index Test Results on Soil Samples

Soils samples recovered from the geotechnical borings were brought to the URI Marine Geomechanics Laboratory for index testing after retrieval. A full list of samples retrieved is found in Appendix A. Borehole locations URI-1 and URI-2 were located in the expected areas where test piles were to be installed, and samples from these boreholes were chosen for moisture content testing and grain size analysis.

3.3.1 Moisture Content Testing (ASTM D2216)

Moisture content testing of the soil samples from borings URI-1 and URI-2 were performed about six months after sample retrieval. It should be noted that samples were placed in sealed glass jars to retain moisture as much as possible. Moist soil samples were weighed before placing in an oven at 110 degrees Celsius, in accordance with ASTM D2216. Samples were dried for 24 hours before removal and subsequent weighing. Moisture content values were then calculated from the masses as plotted with depth in Figure 28, and display much higher levels of moisture below the water table:

Figure 28: Moisture content results from samples from boreholes URI-1 and URI-2 six months after retrieval

3.3.2 Dry Sieve Analysis and USCS Classification (ASTM D6913 and D2487)

Grain size analysis of all samples from borings URI-1 and URI-2 was performed in accordance with ASTM D6913. Sieves ranging in mesh size from 4.75 mm down to 0.075 mm were stacked and samples were passed through. The sieves were shaken in a Gilson Model SS-8R mechanical shaker for five minutes. Mass of soil retained on each sieve was recorded, with percent passing values then calculated. From this analysis, the following grain size distribution is produced in Figure 29:

Figure 29: Grain size distribution determined from samples from borings URI-1 and URI-2

From the particle size distribution presented in Figure 29, USCS classification can be determined based on ASTM D2487. Coefficient of uniformity and coefficient of curvature values were determined. Classification of the fine-grained portion relied on knowledge of presence of silt in the area. Table 1 presents these classifications:

Table 1: USCS classification of soil samples obtained from the URI Allen Harbor site

Soil Sample	D10 (mm)	D30 (mm)	D60 (mm)	Cu	Cc	USCS Group Symbol	USCS Group Name
URI-1 S1	0.19	0.33	0.64	3.37	0.90	SP	Poorly Graded Sand
URI-1 S2	0.17	0.3	0.6	3.53	0.88	SP	Poorly Graded Sand
URI-1 S3	0.076	0.12	0.21	2.76	0.90	SM	Silty Sand
URI-1 S4	0.076	0.17	0.55	7.24	0.69	SM	Silty Sand
URI-2 S1	0.17	0.24	0.36	2.12	0.94	SP	Poorly Graded Sand
URI-2 S2	0.076	0.21	0.92	12.11	0.63	SM	Silty Sand
URI-2 S3	0.1	0.28	0.74	7.40	1.06	SW-SM	Well-Graded Sand with Silt
URI-2 S4	0.086	0.21	0.37	4.30	1.39	SP-SM	Poorly Graded Sand with Silt

As detailed in Table 1, the general soil profile at boreholes URI-1 and URI-2 ranges from poorly graded sands at the surface, to well graded and poorly graded sands with silt and silty sands at depth.

3.3.3 Wet Sieve Analysis (ASTM D1140)

Upon the completion of dry sieve analysis, fines content levels warranted testing by wet sieve analysis (ASTM D1140) to evaluate whether flocculated aggregation of particles impacted particle size results. In this case, sample URI-1 S3 and URI-2 S2 were soaked in a sodium hexametaphosphate solution and filtered through a 0.075 mm diameter sieve until the effluent was clear. URI-1 S3 and URI-2 S2 were determined to have fines contents of 9.03 of 20.72 percent, respectively, as shown in Figure 29. In comparison with the fine-grained material content for the respective samples from the dry-sieve analysis, URI-1 S3 lost fines content due to a testing error with a leaking container, leading to decrease of 4.52 percent material passing, and URI-2 S2 gained 4.67 percent material passing, indicating some flocculated particles existed in the dry sieve analysis.

3.3.4 Carbonate Content Testing (ASTM D4373)

Due to the location of the test site in a marine environment, with a large presence of saltwater shellfish, carbonate content of samples from boring URI-2 were tested to determine if calcareous soils are at the site. This testing was performed by the Norwegian Geotechnical Institute in Houston, Texas in accordance with ASTM D4373. It was determined that there were zero carbonate content in all of the samples from boring URI-2 (S1-S4), indicating calcareous soils are not present (Rahim, 2020, personal communication).

3.4 Cone Penetration Testing (ASTM D5778)

Due to the relative imprecision provided through Standard Penetration Testing, Cone Penetration Testing was performed at the site to determine a complete profile of variation of in-situ soil properties with depth. Testing was performed by ConeTec Inc. on May 2, 2019 consisting of four seismic Cone Penetration Tests (SCPT19-01 through SCPT19-04) and one standard Cone Penetration Test (CPT19-05), at locations detailed in Figure 25. Full results of testing are presented in Appendix A.

Testing involved the use of a track rig (see Figure 30) to drive a penetrometer with a 15-square centimeter cone area and 0.8-cone area ratio to a depth of approximately 10 meters. The penetrometer contained tip and friction sleeve strain load transducers, a pore pressure transducer with a 6 mm thick polyethylene filter, a temperature sensor, an inclinometer, and a seismic geophone sensor. It was advanced with a speed of 2 +/- 0.5 cm/s in accordance with ASTM D5778.

Figure 30: Track rig used to perform CPT testing at the Allen Harbor site

During cone advancement; depth, uncorrected tip resistance (q_c), sleeve friction (f_s), and dynamic pore pressure were determined and post processed. Results are detailed in Figure 31. One pore water dissipation test was performed during advancement of SCPT19-01 to gauge pore pressure response for the rest of the site.

Figure 31: Interpreted CPT results determined at the Allen Harbor site to a depth of 5m, including uncorrected cone tip resistance, friction ratio, pore pressure response, shear wave velocity, and Robertson Soil Behavior Type Index (Robertson, 2010b)

As noted, four of the Cone Penetration Tests contained a shear wave velocity testing component. This component measures the vertical shear wave travel time of a surface generated horizontal hammer blow on a steel beam. The beam was located 2.0 meters horizontally from the center of the track rig, where the CPT penetrometer was advanced. The vertical shear wave was then recorded by the geophone in the penetrometer, which was located 0.20 meters above the cone tip. The general set-up and parameters determined from a shear wave velocity test are detailed in Figure 32.

Figure 32: Shear wave velocity calculation schematic (ConeTec Inc., 2020).

To calculate the shear wave velocity, two sets of assumed shear wave travel distances and recorded travel times are necessary, and the quotient of the net travel distance and net travel time is the shear wave velocity. During the testing at the URI Allen Harbor site, seismic readings were determined every 0.5 meters of driving.

3.5 Site Investigation Interpretations

3.5.1 Interpreted Soil Profile

A general profile of soil properties can be determined based on results detailed in Figure 31 for CPT analysis, as well as the SPT results. It should be noted that SCPT19-01 was inadvertently performed at the location of Boring URI-1, and thus, results from this test were not typical of surrounding tests in the area. As such, the data from this test is ignored.

This general profile can be made based on both SPT relative density values and the cone tip resistance and friction ratio results from CPT testing and is detailed in Figure 33, and lies along Section A-A' in the site plan in Figure 25:

Figure 33: Interpreted soil profile of Allen Harbor site based on SPT and CPT results along Section A-A' from Figure 25

Based on Figure 33, the general profile of soil layering that exists at the Allen Harbor site is interpreted as follows. From the surface to a depth of approximately 2.3 meters lies a medium sand layer, which is underlain by an approximately 1.4-meter thick fine sand layer. Underneath these two layers lies a layer of sand that extends over the western edge of the site, approximately 1.0 meter thick, and a pocket of sand and silt towards the center part of the profile, approximately 1.2 meters thick. The eastern section of the test site contains fine sand over this portion. At this point, the limits of SPT investigations were reached, and the profile to a depth of 10 meters can be classified as sand by the Robertson Soil Behavior Type Index.

3.5.2 Small-Strain Shear Modulus Results

The seismic portion of the Cone Penetration Testing at the Allen Harbor test site also provided insights into the small strain shear modulus (G_0 or $G_{\max}$). The small-strain shear modulus is produced when a soil is loaded with shear stresses, and shear strain is monitored. A generalized plot of a shear stress against shear strain for soil is presented in Figure 34 (Kramer, 1996):

Figure 34: Shear stress-strain curve detailing small-strain shear modulus performance of soils (Kramer, 1996)

Soil behaves elastically under small levels of strain, which based on elastic theory, determines that the initial portion of the shear stress-strain plot will be linear. Thus, the slope of this line can be determined, which will determine the rate of strain produced under shear loading up to yield. This value is highly practicable for piles, as the small strain shear modulus will aid in providing predications of static capacity.

Also by elastic theory, the small strain shear modulus can be related to the total density (ρ) of the soil and the shear wave velocity (V_s) by the following relation (Robertson, 2010b):

$$G_0 = \rho V_s^2 \quad (3)$$

For the preceding relation, the total density can be determined as the quotient of the total unit weight of the soil and the acceleration due to gravity (9.81 m/s^2). With shear

wave velocity values determined from SCPT results in Section 3.4, and total unit weights of the soil from CPT relations (19.48 kN/m³), a plot of small-strain shear modulus with depth can be produced for the soil at the site, as shown in Figure 35.

Figure 35: Small strain shear modulus with depth based on shear wave velocity determinations at the Allen Harbor site

As shown in Figure 35, there exists large scatter in the data for small strain shear modulus with depth, with values up to 500 MPa determined, with most values ranging from 0 to 200 MPa.

In order to determine the accuracy of the results in Figure 35, empirical correlations of shear modulus were implemented. The first correlation is as follows, based on the UWA-05 pile capacity method (Lehane et al. 2007):

$$\frac{G}{q_c} = 185(q_{c1N})^{-0.75} \quad (4)$$

Where q_c = uncorrected cone tip resistance and q_{c1N} = the normalized cone tip resistance normalized to overburden effective stress. Since uncorrected cone tip resistance values were determined for a full profile of depth, a plot of small-strain shear modulus from

SCPT shear wave velocity and the UWA-05 cone tip resistance relation can be determined. This plot is found in Figure 36:

Figure 36: Small-strain shear modulus results from CPT shear wave velocity measurements and UWA-05 cone tip resistance correlation

As can be noted in Figure 36, for the first 2.5 meters of depth, the UWA-05 estimation provides similar results as values determined from the in-situ shear wave velocity measurements. However, for the full profile of values, the SCPT shear wave velocity small strain shear modulus produces much more scatter and inconsistency.

Also due to the wide scatter of small-strain shear modulus results from CPT shear wave results, shear wave velocity measurements were performed at the University of Rhode Island Marine Geomechanics Laboratory on soil samples obtained from Boring URI-1 (Samples 1-4).

This testing involved a prepared soil sample being placed in a confining chamber, which produces K_0 (zero lateral strain) conditions for the soil. The confining ring rests above a baseplate containing a bender element, which is a piezoelectric element that when vibration is induced back and forth, a voltage is produced. Bender elements can

also vibrate when an alternating excitation voltage is placed upon them, which allowed a bender element to be placed in a top cap above the prepared sample, to generate shear waves in the soil mass (Kramer, 1996) (Donaghe et al., 1988). A cross-sectional view of a bender element generating a shear wave is presented in Figure 37:

Figure 37: Cross-sectional view of bender element generating a shear wave (Donaghe et al., 1988).

The bender element placed in the top cap, above the soil sample, is connected to a RIGOL Technologies, Inc. DG1022 function generator, which imparted alternating voltage to the bender element. The shear wave generated by the top bender element travels through the soil mass, to the bender element placed in the baseplate, which then generates a voltage due to the wave. The voltage produced travels to a Tektronix TDS2000C oscilloscope, where the output shear wave is produced. A set of wire leads from the function generator is also connected to the oscilloscope, so that the output (received signal) and input (driving signal) waves can be compared. Figure 38 provides a view of this setup:

Figure 38: General bender element shear wave velocity test setup (Donaghe et al., 1988)

The equipment setup implemented for this study is detailed in Figure 39:

Figure 39: Bender element test setup as a part of this study. The oscilloscope is at the top of the figure, with the function generator beneath it, and the soil-bender element assembly in the foreground

To test the soil under similar stress conditions as presented in the field, soil was air pluviated from a funnel into the confining ring, and then dry tamped in three layers to produce a lower void ratio.

It should be noted that in-situ void ratios of the tested soils were not known, however relative densities as determined by CPT correlations indicate the soil at site is near e_{min} , warranting use of a denser soil preparation manner. Relative density

estimations with depth from CPT results provided by ConeTec Inc. for the Allen Harbor site are detailed in Figure 40:

Figure 40: Relative density with depth from CPT data at the Allen Harbor test site (after ConeTec Inc., 2019)

Stresses were then applied to the soil sample with the use of a load frame. Samples were applied vertical loads based on the estimated vertical overburden stress present in the field from unit weights estimated from CPT results. The load frame setup is detailed in Figure 41.

Figure 41: Load frame setup for bender element testing. Load is applied by the platen underneath the soil-bender element assembly, with a reaction force applied through the load cell and crossbar system above the assembly.

With this setup, “burst” excitation voltages of 20 volts (10 volt amplitude), at a frequency of 4.0 kHz and a period of 10 ms, were generated on the bender element in the top cap, creating a shear wave, which induced a voltage on the lower bender element. The generated waveform for each sample is detailed in Figure 42, and output waveforms produced are detailed in Figure 43:

Figure 42: Generated waveforms for bender element shear wave velocity testing of Allen Harbor boring URI-1 samples

Figure 43: Output waveforms for bender element shear wave velocity testing of Allen Harbor boring URI-1 samples

Note that the generated voltage in Figure 43 is at a different magnitude than the generated wave in Figure 42, since the output waveforms were not amplified. To determine the shear wave velocity, the difference in arrival times of the first peak of both the generated voltage waveform and the output voltage waveform are analyzed, as well as the distance between the bender element tips. This process is known as the “peak-to-peak” method. Data for each of the samples is tabulated in Table 2:

Table 2: Results of shear wave velocity and small-strain shear modulus from bender element testing of soil samples from URI-1

Allen Harbor Soil Shear Wave Velocity Results - URI-1					
Sample No.	e0	Vertical Effective Stress (kPa)	Mean Effective Stress (kPa)	Vs (m/s)	G0 (MPa)
S1	0.60	5.94	3.56	120.64	23.71
S2	0.52	26.66	15.96	159.56	43.89
S3	0.77	41.41	24.80	170.56	43.07
S4	0.50	56.15	33.62	188.43	62.06

The small strain shear modulus results from Table 2 can then be compared with the results of the in-situ SCPT shear wave velocity determinations, as well as the UWA-05 estimations. This comparison is presented in Figure 44:

Figure 44: Small-Strain shear modulus results from in-situ SCPT shear wave velocity, UWA-05 cone tip correlations, and Allen Harbor soil sample bender element testing

Results from Figure 44 indicate that the small strain shear modulus is very similar in magnitude to estimations made by the UWA-05 correlation. This being said, it is appropriate to know that the bender element testing is an estimation of the in-situ small strain shear modulus. This is because the natural fabric of the soil has been disrupted in laboratory testing, as well as uncertainties in void ratio and shear wave versus compressional wave arrival times. The uncertainty of compressional wave and shear wave arrival time is present since compressional waves will travel faster than shear waves, with typical velocities ranging from 1450 to 2200 m/s, and 150 to 800 m/s, respectively (Schumann et al., 2014). Compressional waves are generated in the bender element system by the bender elements vibrating, compressing the soil horizontally, creating waves that reflect off of the testing cylinder walls, which eventually arrive at the lower bender element. Since it is difficult to remove the compressional waves in the test setup, the potential disturbances in results are plausible.

As another set of small-strain shear modulus values to consider, Hardin and Drnevich proposed the following equation to determine the small-strain shear modulus empirically (1972):

$$G_{max} = 1230 \left(\frac{(2.973 - e)^2}{(1 + e)} \right) (OCR)^k \bar{\sigma}_m^{0.5} \quad (5)$$

Where e = soil void ratio, OCR = the overconsolidation ratio of the soil, k = an exponential parameter based on plasticity index (0 for granular soils), and $\bar{\sigma}_m$ = the mean effective stress of the soil (in U.S. Imperial psi).

Using the void ratios and mean effective stresses from the bender element shear wave velocity tests presented in Table 2, estimates of the small-strain shear modulus to compare with bender element results are possible.

To be able to compare the all of the previous sets of small-strain shear modulus results with in-situ results, Imai and Tonouchi have suggested the following equation which is correlated to SPT blow counts (1982) (Kramer, 1996):

$$G_{max} = 325N_{(60)}^{0.68} \quad (6)$$

Where N_{60} = standard penetration resistance corrected for field conditions. This standard penetration resistance was obtained through correction of blows counts in Figure 27. Since four SPT tests were performed to full depth at the site, the minimum and maximum small strain shear modulus results for all boring depths are compiled to produce a bound of values. Results of the equations suggested by Hardin and Drnevich, as well as Imai and Tonouchi, in comparison the other evaluated results are presented in Figure 45:

Figure 45: Small-strain shear modulus values as determined by in-situ shear wave velocity testing, UWA-05 correlations, soil sample bender element testing, and correlations determined by Hardin and Drnevich (1972) and Imai and Tonouchi (1982)

As is determined in Figure 45, the Hardin and Drnevich correlation results are similar or slightly larger in magnitude than those determined by the UWA-05 and

bender element results. The bounds of the Imai and Tonouchi correlations are similar on the lower end, however, the spread of the upper bound becomes amplified with depth. The relative imprecision of using corrected SPT blow counts is magnified in the Imai and Tonouchi approach, but may suggest the wider spread of in-situ small strain shear modulus values with depth (up to approximately 150 MPa) may be reasonable for the site soils.

CHAPTER 4- STATIC PILE CAPACITY ESTIMATIONS

With determinations made of soil conditions located at the URI Allen Harbor test site, static pile capacity estimations were made to determine the capacity expected from piles driven at the site. This chapter will present results of the two methods used to estimate capacity: the API method and the UWA-05 CPT-based method. It should be noted that the goal of this analysis was not to evaluate the results of the different estimation methods, rather to size components for load tests performed.

4.1 API Pile Capacity Estimations

To provide a basic estimation of static capacities of piles installed at the URI Allen Harbor site, the API RP-2A 2007 estimation method was first implemented. Two iterations of the API Method were performed, the first assuming the piles to be installed were low displacement ($K=1.4K_0$) and the second assuming the piles were high displacement ($K=1.8K_0$).

For both analyses, the in-situ soil was discretized into four separate layers, the lower three layers containing different drained friction angle values based on CPT interpretations from the site. The first and second layers are differentiated by the high water table level, with Layer 1 being located above, and Layer 2 being located below the high water table. Table 3 details the layer properties for both API method iterations.

Table 3: API Method soil layer properties

Soil Layer	Layer Depth (m)	Layer Thickness (m)	Friction Angle of Layer (Degrees)	Interface Friction Angle (Degrees)	Unit Weight (kN/m ³)
1	0-0.91	0.91	37.0	24.7	19.5
2	0.91-2.13	1.22	37.0	24.7	9.7
3	2.13-3.66	1.53	43.0	28.7	9.7
4	3.66-4.57	0.91	41.5	27.7	9.7

The total unit weight for Table 3 values is 19.5 kN/m³ and the submerged unit weight is 9.67 kN/m³, based correlations made from CPT results. Further analysis

details not included in Table 3 are the assumption that the drained interface friction angle of the soil-pile system is two-thirds of the drained soil friction angle, the outside pile diameter is 0.114 meters and the pile depth is 4.57 meters.

The unit shaft friction equation based in the API RP-2A Method is as follows (2007):

$$F_s = \beta \sigma'_v = \sigma'_v K \tan(\delta') \quad (7)$$

Where σ'_v = overburden effective stress at the layer midpoint, K = lateral earth pressure coefficient, which is related to the at-rest earth pressure coefficient, and δ' = drained pile-soil interface friction angle, assuming the pile is drained during loading. The tangent of the interface friction angle multiplied by the lateral earth pressure coefficient complete the term, β .

The total static pile capacity (R_s) is then determined to be the summation of the unit shaft friction (F_s), multiplied by the surface area of the pile in that layer (A_s), calculated as follows (API, 2007):

$$R_s = \sum F_s A_s \quad (8)$$

Based on the values presented in Table 3, and the unit shaft friction and total static capacity equations indicated above, the low displacement pile and high displacement pile static skin friction estimates were determined to be 12.28 kN and 15.79 kN, respectively. These results are indicated in Table 4. It should be noted that these values do not include to the weight of the pile itself, assume the pile is plugged, and do not account for soil plug weight (which total approximately 1.0 kN for each of the piles

installed at the site, at a maximum if the thickness of the soil plug is the full embedded depth).

Table 4: API 2007 static capacity estimations for low and high displacement piles

API Pile Capacity Estimations: Allen Harbor Site (2007 API RP2A)		
	Low Displacement Pile (K=1.4*K0)	High Displacement Pile (K=1.8*K0)
Estimated Capacity (kN)	12.28	15.79
Pile Outside Diameter (m)	0.114	
Pile Depth (m)	4.57	
Total Unit Weight (kN/m ³)	19.48	
Submerged Unit Weight (kN/m ³)	9.67	
Water Table Depth (m)	0.91	

It should be noted that only the skin friction component of the API RP-2A method was implemented in this study, as the piles tested at the Allen Harbor site are loaded in tension, with skin friction as the only resistive component.

4.2 UWA-05 Static Capacity Method

The UWA-05 method is a CPT-based static capacity estimation method. The method seeks to relate CPT cone tip resistance provided with pile capacity due to the similar strength mechanics involved between piles and CPT tests (Lehane et al., 2007). The main advantage over API is that the UWA-05 method explicitly includes the effects of friction fatigue and shear band dilation. Full derivation of the UWA-05 method for the URI Allen Harbor Analysis is detailed in Appendix B. These estimations were performed before piles were driven at the site, so all calculations were based on CPT results, and known pile geometry. The total skin friction is defined by the following equation (Lehane et al., 2007):

$$TSF = \left(\frac{f}{f_c} \right) (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan(\delta_{cv}) \quad (9)$$

Where the $\left(\frac{f}{f_c}\right)$ = empirical constant that relates tensile resistance to compressive cone tip resistance (unity in the case of compressive loading, 0.75 in the case of tensile loading), σ'_{rc} = radial effective stress after pile installation and equalization, $\Delta\sigma'_{rd}$ = change in radial effective stress due to soil dilation during application of pile loads, and δ_{cv} = constant volume (critical state) interface friction angle of the soil-pile system.

The method also takes into account the small strain shear modulus of the site soil to determine the dilatory change in effective stress. Values can either be input based on shear wave velocity determinations or calculated from the following relation of shear modulus to the uncorrected cone tip resistance, also presented earlier (Lehane et al., 2007):

$$\frac{G}{q_c} = 185(q_{c1N})^{-0.75} \quad (10)$$

Due to the wide variation and small sampling frequency of small-strain shear modulus values presented in Section 3.5.2, the equation indicated above was implemented.

Sampling frequency for uncorrected cone tip resistance values from SCPT testing was every 0.025 meters of tip advancement, providing 182 unit skin friction values per CPT test over the 4.57 meter estimated embedment depth of the piles at the site. Due to the large number of results, the mean unit skin friction value for all 182 values was multiplied by the embedded surface area to produce the estimated pile capacity for the piles at the site. Table 5 presents the UWA-05 estimated static capacities for piles installed at the site to 4.57 meters of embedment:

Table 5: UWA-05 CPT-based static capacity results for 4.57 meters of pile embedment

UWA-05 Results: Standard Conditions, 4.57m Embedment		
CPT Test	Average Qs (lbs)	Average Qs (kN)
SCPT19-02	14693.41	65.36
SCPT19-03	11563.83	51.44
SCPT19-04	11341.89	50.45
CPT19-05	15818.37	70.36
Average	13354.38	59.40

It should be noted that values presented in Table 5 are for the immediate locations of the CPT tests in Figure 25, and thus, some judgment is necessary in application to piles located between CPT locations. The piles installed at the site are located approximately halfway between the CPT locations, so that two estimates of capacity can be made for each pile.

Piles installed at the Allen Harbor test site reached depths of 4.27 meters after driving, with 0.30 meters of pile stickup. Table 6 presents the UWA-05 method static capacity estimates reflected for the embedment of 4.27 meters:

Table 6: UWA-05 CPT-based static capacity results for 4.27 meters of pile embedment

UWA-05 Results: Standard Conditions, 4.27m Embedment		
CPT Test	Average Qs (lbs)	Average Qs (kN)
SCPT19-02	13122.0	58.4
SCPT19-03	11224.5	49.9
SCPT19-04	10127.3	45.0
CPT19-05	12778.3	56.8
Average	11813.0	52.5

Analyzing the data presented in Table 6, and comparing with locations at the Allen Harbor site in Figure 25, it is clear that the soil at the site increases in capacity moving from west to east. SCPT19-04 determined a capacity of 45.0 kN, which is 20.7 percent lower than the capacity determined for CPT19-05, on the eastern side of the test site.

This is in general agreement with the blow count resistances encountered during SPT testing in Figure 27. With this being said, the mean capacity estimate across the site is 52.5 kN, which is 333% of the API estimated capacity for a high displacement pile.

CHAPTER 5- LOAD TEST EQUIPMENT DEVELOPMENT

With pile capacity estimations performed to indicate expected 7-day monotonic loads, testing equipment was designed to be able to achieve these load levels. The following chapter details the development and assembly of the monotonic and cyclic test equipment implemented for this project.

5.1 Overview of Equipment

As noted in Section 2.9, ASTM D3689 provides a test setup that is applicable in loading piles monotonically. Referring to Figure 21, general load test equipment for a tensile load test includes a tensile connection member to transfer hydraulic jack loads to the pile, a reaction frame to allow the tensile connection to bear on the jack, the hydraulic jack itself to apply tensile loads, a test beam to support the jack and reaction frame under loading, and cribbing blocks to distribute the test beam loads into the surrounding soil (ASTM, 2013).

To provide measurements of load and deflection of the pile, instrumentation in the form of calibrated load and displacement transducers are implemented. The load cell is to be located above the hydraulic jack that so that the load created by the jack is obtained directly. The displacement transducer is to be mounted to a reference beam placed separate of the test beam, so as to provide an independent measurement of displacement. Figure 46 provides an elevation plan of the monotonic test system implied in this study.

Figure 46: Monotonic test setup implemented for study

This setup is sufficient to provide reaction forces based on loads generated during monotonic and cyclic testing. However, the source application of jack loads is a critical element for design. A hydraulic jack functions through converting hydraulic pressure offered by an incompressible fluid, into load based on a basic piston and cylinder setup. The incompressible fluid is injected into the jack cylinder at a set pressure, which causes the relatively weightless jack piston to advance until sufficient resistance is offered by the pile system against the hydraulic pressure. Knowing the hydraulic pressure in the jack (in terms of load per unit area), as well as the cross-sectional area of the piston in the jack (unit area), allows pile load to be determined separate of a load cell.

In the case of a monotonic load test, the hydraulic pressure the jack receives per load step is a set amount. A simple hydraulic pump can apply and hold the predetermined pressure for as long as necessary to determine drained pile displacement. This method of applying a pressure and holding it for a preset period of time is defined

in this study as a “load step”. A simple hydraulic jack can then raise the pressure higher to create a series of load steps until pile failure has been achieved.

This monotonic loading creates a relatively simple, non-dynamic function for a hydraulic jack and pump to accomplish. In the case of cyclic loading however, the jack load, and thus hydraulic pressure, are required to proceed along a sinusoidal function with respect to time. This function requires that a jack system with complex pressure regulation is implemented. Many hydraulic jacks and pumps on the market do not have sophisticated pressure regulation systems, and thus a pressure regulator, placed along the flow line from the hydraulic pump to the jack can be used to achieve the cyclic loads required for this type of test. Detailed further in this chapter is the implementation of a backpressure regulator.

5.2 Load Frame Development

The following subsections in Section 5.2 will detail the development of the load test equipment from the pile upwards to the jack and then downwards to the reaction cribbing blocks.

5.2.1 Pile Cap and Tensile Rod Development

The test piles, which will be described in much greater detail in Chapter 6, are composed of HW-series casing sections, with threaded male and female ends to allow connection of several members. Figure 47 details the male and female ends for the casing sections. The inner and outer diameters are 101.6 and 114.3 mm, respectively (N&N Drilling, 2019). The threaded male and females ends of the sections are 76.2 mm long and contain a 6.35 mm thread pitch (4 threads per inch).

Figure 47: Pile casing section connection detail for: (a) male end and (b) female end

A steel casing cap was obtained from N&N Drilling Supply, with Part Number: 16-HWCP. The cap features a male threaded end designed to fit the female end of the HW casing sections. A 1.905 cm diameter hole was drilled through the center of the casing cap to allow pass-through of a tensile rod. A cross-section of the pile cap and threaded connection system is detailed in Figure 48.

Figure 48: Casing cap cross-section with tensile rod connection passed through. A cutaway view of the female casing end is detailed at the bottom of the Figure

Tensile strength evaluations are required of the pile cap-tensile rod assembly to ensure failure will not be reached during testing. The pile cap is constructed of the same material as the casing sections (AISI Grade 4130 steel) and has a tensile yield stress of 460 MPa (AZO Materials, 2012). Knowing the inner diameter and outer diameter of the casing is 101.6 and 114.3 mm, respectively, the net area of the female steel connection is 0.00215 m². Thus, the tensile yield load that the casing can withstand is 990 kN, much larger than the anticipated pile loads. Even though the threads have slightly less diameter, the casing is sufficient to handle axial loads from the system.

As for the threaded rod, the pile cap was drilled to accept two diameters, a 1.905 cm (U.S. Imperial #6) rod, and a 1.588 cm (U.S. Imperial #5) rod with a 1.588 cm inside diameter and 1.905 cm outside diameter brass pipe placed over the rod to act as a centralizer. Based on the UWA-05 static capacity estimates, approximately 56 kN of tensile strength is required for one-week testing, not incorporating a factor of safety.

For the larger rod, obtained from Williams Form Engineering Corporation was a 1.52 meter long, 1.905 mm diameter (U.S. Imperial #6) ASTM A615 Grade 75 steel rod, part number: R61-06. Minimum yield strength is 147 kN (Williams Form Engineering Corporation, 2020a), which would withstand anticipated ten-week tensile loads offered by the casing sections.

For the smaller rod, also obtained from Williams Form Engineering Corporation was a 1.52 meter long, 1.588 cm diameter (U.S. Imperial #5) ASTM A29 Grade C1045 steel rod, part number: B7S-05. Minimum yield strength is 90 kN (Williams Form Engineering Corporation, 2020b). This rod was anticipated to be sufficient for the one-

week tensile tests, however were weaker than necessary for ten-week estimated aged resistances.

For both rods, several sets of washers and nuts were obtained to match the size of the rod, and were constructed of similar grades of steel, and similar strength was estimated. A set of washers and nuts is placed above and below the pile cap in Figure 48 to hold the rod fixed to the pile cap when cycling loads.

5.2.2 Hydraulic Jacks

As hydraulic jacks were to apply the testing load on the installed casing lengths, and were to operate under high hydraulic pressures, significant consideration went into development. The jacks implemented for the project required a through-hole design to be able to allow the threaded rods to bear on the jack top plates.

Implemented by the research group on previous pile load tests was an OTC Tools Power-Twin 30 Ton Ram, model number 4121. Capacity of the jack is 312.9 kN (31.4 U.S. tons) at a maximum hydraulic pressure of 68948 kPa (10,000 psi). Maximum piston stroke is 6.35 cm (2.5 in.) and effective cylinder area is 40.52 cm² (6.28 in²). Center-hole diameter is 3.30 cm (1.3 in.) (SPX Flow, 2018). Figure 49 details this jack:

Figure 49: OTC Tools 30 Ton Ram, implemented for the project

This hydraulic jack was estimated to be sufficient for the ten-week tests, however too powerful for the one-week tensile tests. Thus, obtained from SPX Flow Power Team was a model number RH102 cylinder. Capacity of the cylinder is 109.6 kN (11.0 U.S. tons) at a maximum hydraulic pressure of 68948 kPa (10,000 psi). Maximum piston stroke is 6.35 cm (2.5 in.) and effective cylinder area is 14.26 cm² (2.21 in²). Center-hole diameter is 1.96 cm (0.77 in.) (SPX Flow, 2018). Figure 50 provides a view of this cylinder.

Figure 50: SPX Flow Power Team RH102 10 ton cylinder

It should be noted that the pressure regulator that controls the hydraulic pressure has a minimum control pressure of 1379 kPa (200 psi), to be described further in the chapter. This pressure translates to 5.59 kN (1256 lbs) of load for the OTC jack, and 1.97 kN (442 lbs) for the SPX Flow Power Team jack. To overcome this additional load, a weight stack consisting of center-hole weight plates was placed above the jack. The tensile rod was then set the bear on the top weight plate through a washer and nut of similar material to the rod.

5.2.3 Test Beam

To apply loads to the piles through the jack, a reaction force needs to support the jack from the bottom. This is accomplished through the use of a test beam. From a previous pile load test at the university, a test beam had been assembled. The beam consists of two 1.83 m (6 ft.) long W6x8.5 steel beams oriented vertically, welded together by 0.476 cm (0.187 in.) thick steel plates on top and bottom at the ends. The beams were separated by 5.4 cm (2.125 in.) of clear spacing. Figure 51 displays orthogonal views of the test beam and section properties.

Figure 51: Orthogonal views of the test beam implemented for the project. Section properties are from the American Institute of Steel Construction (2006).

As the jack would be placed at center span on the test beam, strength calculations were necessary to determine the maximum load the beam that could be placed in elastic conditions. The grade of steel the beams were constructed of was unknown, however, it

was surmised either to be A36 or A992 steel, with A992 being more likely. The yielding stress of A36 steel is 248.2 MPa (36.0 ksi). The allowable bending stress of steel is 0.6 of the yield stress, 148.9 MPa (21.6 ksi). Assuming the beam is supported at the ends with a 0.15 m (6 in.) cribbing block, a moment arm of 0.91 m (2.5 ft) from the jack is generated. Thus, taking into account the two beams, the estimated load the beam system can support is 32.7 kN in the elastic region. For the A992 steel beam, the elastic support is 45.2 kN. This is sufficient for the loads offered by the piles.

5.2.4 Cribbing Blocks

To support the test beam at both ends, 0.15 m wide, 0.15 m tall, and 0.60 m long (6 in. wide, 6 in. tall, and 2 ft. long) nominal wooden blocks were stacked three in a row for two layers on each side of the beam. Then, a similarly sized CMU block was stacked, with a wooden block above it, on which the test beam rested. This layout is detailed in Figure 46.

The wooden cribbing consisted of No. 2 Southern Pine, from the grading stamp provided in Figure 52.

Figure 52: Wooden cribbing grading stamp

Since the sections will be loaded perpendicular to the grain, the reference design strength value for No. 2 Southern Pine is 2585.5 kPa (375 psi) (American Wood

Council, 2019). All reference design adjustment factors are unity. The block directly supporting the beam will support the test loads with the least bearing area, making it the critical design member. Knowing that the steel beam setup is 25.4 cm wide, 13.97 cm is the bearing width provided by the wooden block (as the test beam plates are that width), and there are two ends supporting the beam, the blocks can withstand 175 kN of load. This amount is sufficient for the tests considered.

Another major design concern for the cribbing support blocks is bearing capacity on the relatively loose surface soil at the test site. To account for these concerns, a bearing capacity calculation using Meyerhof's generalized bearing capacity equation was implemented. This equation takes the form of (Meyerhof, 1963):

$$q_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + 0.5\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i} \quad (11)$$

Since the soil on the site is primarily sand, the undrained strength term at the beginning of the equation is ignored, and since the bearing blocks are resting on the soil surface, the overburden stress term is ignored. This reduces Meyerhof's bearing capacity equation to:

$$q_u = 0.5\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i} \quad (12)$$

From the CPT analysis presented in Chapter 3, the total unit weight, γ , of the in-situ sand is 19.48 kN/m³. The base width (B) of the cribbing members resting on the soil was assumed to be 0.61 m. N_γ was determined to be 66.19, based on an assumed drained friction angle of 37 degrees provided by CPT analysis (Das and Sivakugan, 2017). Since the layout of the bearing blocks was assumed to be square, the shape factor, $N_{\gamma s}$, was determined to be 0.6 based on the equation provided by (De Beer, 1970):

$$F_{\gamma s} = 1 - 0.4 \frac{B}{L} \quad (13)$$

Since the loads were to be applied vertical and on the surface, the inclination factor, $F_{\gamma i}$, and depth factor, $F_{\gamma d}$, were 1.0.

Thus, the bearing capacity of the in-situ soil is determined to be 236.0 kPa (4.93 ksf). Using the average pile capacity determined through the UWA-05 method in Chapter 3 (56.16 kN) divided by two for the two sides of the supports (28.08 kN), and distributed over the area of the cribbing setup (0.372 m²), the demand bearing pressure is 75.5 kPa. Thus, the factor of safety against a bearing capacity failure for the cribbing setup is 3.13, sufficient for use.

The final aspect of determining placement of the cribbing blocks is in regards to spacing requirements provided by ASTM D3689. The minimum clear spacing from the cribbing to the testing pile is required to be at least five times the outside pile diameter, or in this case, 0.572 m (22.5 in) (ASTM, 2013). Thus, during cribbing setup, this clear spacing was maintained.

5.3 Mechanical and Electrical Equipment

With the load frame itself developed to support pile loads, a system of mechanical and electrical devices were required to produce hydraulic pressure for the jack, as well as record pile loads and displacements. The subsections of Section 5.3 will first outline data acquisition instruments and then describe the creation and regulation of hydraulic pressure to the jack.

5.3.1 Load Transducers and Calibration

During the load tests, pile head load and displacement required monitoring to determine when failure was reached. To accomplish this, an electronic load cell and

Linear Variable Differential Transformer (LVDT) were implemented to determine loads and displacements, respectively.

Two load cells were implemented during this investigation, the primary being an Omega Engineering Inc. Model LC8151-750-10K load cell with serial number: 234271. This load cell can read loads up to 44.13 kN (10 kips), and is a through-hole design, with an inside diameter of 1.91 cm (0.75 in), to fit around the smaller tensile pile head rod. The load cell has an excitation voltage of 10 V DC current and an output of 2 mV/V. Repeatability is +/- 0.1% Full Scale Output and can safely handle an overload of 300% of capacity (Omega Engineering Inc., 2020a).

A calibration of the load cell was provided by Omega Engineering Inc. during manufacturing, several years before this study. The calibration equation of this load cell, in U.S. Imperial pounds versus output millivolts is:

$$y = 475.36x + 95.692 \quad (14)$$

Calibration from a previous study indicated an equation of:

$$y = 453.67x + 78.43 \quad (15)$$

With an R^2 value of 0.9996 (Reyes Mejia, 2015). During this investigation, a calibration was performed using an oedometer with a lever arm amplification ratio of 10:1. Figure 53 details the calibration setup:

Figure 53: LC8151-750-10K oedometer calibration setup with load cell

During a calibration, a maximum load of 10.0 kN was placed on the load cell, approximately a quarter of the full range. A calibration equation was produced of:

$$y = 455.28x - 25.3 \quad (16)$$

The equation had an R^2 value of 0.99995. Thus, this calibration proved sufficient and was implemented during the pile load tests. Figure 54 provides a plot of this calibration in U.S. Imperial units, as used during data collection.

Figure 54: LC8151-750-10K load cell calibration performed as part of this study

The second load cell, which was only used for the first pile load test, was an Omega Engineering Inc. Model LC8250-875-30K load cell with serial number: 336229. The maximum load this load cell can read is 133.45 kN (30 kips), and implements a through-hole design with an inner diameter of 2.22 cm (0.875 in) to fit around the larger pile head tensile connection. The load cell has an excitation voltage of 10 VDC, and an output signal voltage of 2mV/V. Repeatability is +/- 0.1% Full Scale Output and the cell can safely handle an overload of 300% of its capacity (Omega Engineering Inc., 2020b).

The load cell was obtained for the project directly from the manufacturer, with a calibration that had been performed in 2015, and determined the following equation relating U.S. Imperial pounds to millivolts:

$$y = 1568.14x - 41.74 \quad (17)$$

To confirm if the calibration was still true, another calibration was performed. To achieve load levels that could be safely placed in a laboratory, a hydraulic pump and the SPX Flow Power Team RH102 jack were used to apply loads in a triaxial testing load frame. Thus, the pressure from the jack was known, as well as the area of the jack piston, from which a load could be determined. Voltages from the load cell were then read. The

following calibration equation was determined for the load cell with an R^2 value of 0.9987, as determined in Figure 55:

$$y = 1267.7x - 603.98 \quad (18)$$

Thus, this calibration was implemented on the pile load tests. It should be noted that effective areas provided by the manufacturer for the jacks had been tested against the calibration of the smaller load cell, which was calibrated against dead weight, and was determined to be accurate.

Figure 55: LC8250-875-30K load cell calibration as implemented for this project

With the calibration of the load cells determined, it was possible to implement the transducers on the pile load test setup. As detailed in Figure 46, during testing, the load cell to be used was placed above the compressive weight stack, which rested on the hydraulic jack, so as to only read the net load that was transferred to the pile.

5.3.2 Displacement Transducer and Calibration

On the project, one LVDT was implemented to determine pile head displacements. This LVDT is an Omega Engineering Inc. LD-621-50 transducer with serial number M930465AH08-02. This LVDT features a spring-loaded rod that actuates a set of

excitation and output coils located in the transducer housing as the rod displaces. Excitation voltage is 10 VDC to 30 VDC (with 10 VDC implemented in this study) and readable travel range is from 0 to 50 mm (1.97 in) of displacement. Output voltage is 10 VDC (Omega Engineering Inc., 2020c).

As part of this study, a calibration of the LVDT was performed by holding the LVDT with a dial gauge holder in a fixed position bearing against a fixed countertop. Pratt and Whitney metallic calibration blocks were inserted between the rod and the countertop, of progressing size, until 6.35 cm (2.50 in) of displacement was achieved. The process was then repeated using blocks of decreasing size. The five highest and lowest readings were removed due to inaccuracy in the data at the ends, and the following calibration equation was developed, in U.S. Imperial inches versus volts, as implemented during the project:

$$y = 0.1963x + 0.297 \quad (19)$$

This calibration has an R^2 value of 0.99997. Figure 56 details the calibration performed:

Figure 56: LD-621-50 LVDT calibration performed as part of this study

With a calibration of the LVDT performed, the transducer was implemented to read pile head displacements. The standard LVDT pile head setup is detailed in Figure 46 and Figure 57.

Figure 57: Placement of LVDT on pile head, with the use of a reference beam

This setup involved placing the bearing end of the transducer on to the top of the pile cap. The LVDT was supported by a magnetic dial gauge holder mounted to a 2 meter long steel box-channel section. The box channel was placed perpendicular to test beam setup, to have an independent arrangement from test beam, and was supported on both ends by an adjustable sawhorse, and connected by C-clamps. A complete testing frame and transducer setup is found in Figure 58.

Figure 58: Complete load frame and transducer setup as implemented in this study

5.3.3 Data Acquisition and Power Supply Development

With electronic transducers implemented to record pile head loads and displacements, a power supply system was required to provide excitation voltages to the system, as well as a data acquisition system was required to collect and record output voltages from the transducers. For the data acquisition system, a Personal DAQ 56 acquisition module, produced by IO Tech Systems was implemented. This data acquisition system featured ten differential device input channels, and was supplied power by either a 115 VAC wall adapter, or by computer power with a USB adapter. Detailed in Figure 59 is the power supply and data acquisition system implemented in this study.

Figure 59: Data Acquisition (DAQ) and power supply wiring setup. The DAQ is in the upper right of the picture, the LVDT power supply is in the lower left, and the load cell power supply is in the lower right.

Output voltages from the transducers were wired to the acquisition system, which were recorded through a Personal DAQ Microsoft Excel add-in. Green and white wires from the transducers provided output signal voltages, and were connected to either the high or low channel port to produce a positive signal voltage. The DAQ add-in recorded output voltages with time for post-processing following the pile tests. The voltage range set for device reading was -10 to 10 VDC. The DAQ scan rate was set to occur with a period of 0.5 seconds for the monotonic test, and a 1.0 second period for the cyclic tests.

As for the power supplies, two separate, albeit similar systems were implemented. The power supply powering the LVDT was an Omega Engineering Inc. DP25B-S Strain Gage Panel Meter, which drew power from 115 VAC wires connected to positive, negative, and ground terminals. The power supply generated the 10 VDC excitation force for the LVDT, with a red wire connected to the positive excitation port and a black

wire being connected to the negative excitation port. The negative excitation port from the power supply was also connected to the common low of the Personal DAQ system.

The power supply for the load cell was a Newport Electronics, Inc. INFCS-001B-1.2 Strain Meter/Controller. This power supply has the same setup as the Omega Engineering Inc. Strain Gage Panel Meter for the LVDT. Power is supplied by 115 VAC positive, negative, and ground wires. The power supply then creates a 10 VDC excitation force through positive and negative ports, which are connected to the excitation wires to the load cell (red and black wires, respectively). The negative excitation port from the power supply was also connected to the common low of the Personal DAQ system.

5.3.4 Hydraulic Pump

The hydraulic jacks used to apply loads to the test piles required a pump to pressurize the hydraulic fluid in the cylinder bore. Obtained from a previous set of pile load tests at the university was an OTC Tools model number 2510A Air/Hydraulic foot pump, as shown in Figure 60. The foot pump converts pneumatic pressure provided by an air compressor into hydraulic pressure.

Figure 60: OTC Tools 2510 Air/Hydraulic pump used to supply hydraulic pressure to the jacks. The pneumatic inlet is on the right side of the picture, and the pressurized hydraulic fluid outlet is on the left.

The air/hydraulic foot pump provides a maximum hydraulic pressure of 69000 kPa (10,000 psi) and requires 0.14 to 0.28 m³/minute (5.0-10.0 CFM) at 690 kPa (100 psi) of pneumatic flow and pressure (OTC Tools, 2011). Directly from the manufacturer, the pump is designed to use a foot pedal to increase and decrease pressure and flow into the jack, meaning the hydraulic outlet of the pump acts two-ways. To be described later, the pressure regulator implemented for the project requires one way flow, so another port was drilled into the hydraulic fluid reservoir of the pump to accept a fluid return line.

5.3.5 Hydraulic Pressure Controller

The hydraulic pump detailed in Section 5.2D is able to increase and decrease pressure, however struggles to maintain a consistent pressure as required for the monotonic pile tests, and has no ability to cycle the pressure in any manner as required for the cyclic tests. After consulting with engineers at Northeast Engineering, an instrumentation and controls system supplier, a pressure control system was purchased and implemented within the pump and jack system.

Northeast Engineering recommended a Tescom Model Number: ER5K-ENDB control system manufactured by Emerson Process Management. This system serves as a backpressure regulator to the pump-jack setup and is rated for a maximum hydraulic pressure of 68948 kPa (10,000 psi), the maximum pressure the pump can produce. In a regular pump-jack setup, the pressure created by the hydraulic pump would immediately be applied to the jack. The pump would supply increasing pressure by adding fluid to the line, since the system is closed and hydraulic fluid is relatively incompressible.

The backpressure regulator is designed to fit along the flow line from the pump to the jack, and is subject to the same pressure the jack experiences, as seen in Figure 61.

Figure 61: Complete flow schematic detailing backpressure regulator setup

In the pressure regulated system, the ER5K system is added with a tee connection to the flow line from the pump to the jack. The hydraulic pump constantly runs, adding pressure to the hydraulic line to the jack and regulator. A pressure transducer in the regulator monitors the pressure in the flow line. Once a preset hydraulic pressure is reached, an exhaust vent is opened in the regulator, diverting fluid from the flow line to the jack and releasing it back to the pump reservoir through an outlet line.

Small changes in hydraulic pressure are corrected by the use of a pneumatic actuated diaphragm, where a supply of pneumatic pressure over the diaphragm is adjusted, causing the diaphragm to displace and hydraulic pressure changes to occur

through the adjustment of valves. The complete air-over-hydraulic system is outlined in Figure 62.

Figure 62: ER5000 Backpressure regulator as implemented. Green arrows show inlet hydraulic pressure from the pump and jack, red arrows show the excess fluid return lines, the blue arrow shows pneumatic diaphragm pressure

In Figure 62, the hydraulic inlet from the pump-jack line is in the lower right of the Figure. The pressurized fluid then enters the hydraulic regulator chamber, a cross-section of which is provided in Figure 63. Attached to the regulator chamber is a pressure transducer, which monitors pressure in the inlet side of the flow setup. Once a pressure level has been reached, the pressure transducers notifies the electro-pneumatic controller that the desired hydraulic pressure has been achieved, and the electro-pneumatic controller releases pneumatic pressure over the diaphragm. This release of pneumatic pressure causes the diaphragm to move upwards, as the reaction pressure offered is no longer present. The diaphragm moving up allows the hydraulic

vent/exhaust valve to open, releasing excess pressure and fluid to the outlet line from the regulator back to the pump.

Figure 63: Cross-section of the ER5K Air/Hydraulic regulator. Note that this cross-section is a mirrored left-right flow version of Figure 62 (Emerson Process Management, 2015)

In a monotonic pile load test, where the load, and by association, pressure, has to remain constant for several minutes, the exhaust outlet on the regulator is open the entire load step. This is because the hydraulic pump is constantly running, which would generate excess pressure if the exhaust valve was closed.

As for a cyclic test, the pressure has to vary from low to high pressure and back to low pressure in a matter of seconds. Going from low pressure to high pressure, the exhaust vent remains closed, which allows pressure to build. When the peak load has been reached, the exhaust vent opens and releases excess pressure to the pump reservoir.

As described, this pressure regulation for the cyclic test is rather coarse and would produce a triangular wave with respect to time. The control system, being an electro-pneumatic system with a Proportional Integral Derivative (PID) controller, has a computer interface that allows for creation of a program to “ramp” the current pressure to a new pressure over a time interval. This interface is called ERTune. The controller processes one pressure change per second at its quickest setting, allowing an eight second load cycle period to be developed, incorporating intervals of $\pi/4$ with respect to time. This ERTune control system interface is detailed in the following section.

5.3.6 ERTune Program

As noted in previous sections discussing the use of a compressive weight stack above the jack to overcome minimum system pressures, the control system of the hydraulic regulator has a minimum control hydraulic pressure of 1379 kPa (200 psi). This means that even at the lowest jack load, which should require zero pressure on the jack, 1379 kPa (200 psi) is required to be present. The compressive weight stack is utilized to overcome this pressure to lower the pile load as close to zero as possible. The net load when the compressive weight stack is subtracted from the minimum system pressure becomes the $Q_{\min}$ value the pile is subjected to.

To show this process in greater detail, a plot of target pile and jack loads is provided in Figure 64. To setup a test, the desired pile head loads are defined first, followed by the regulated pressure control points to be set with the ERTune interface. With the pressure set, the jack loads can be determined, which is simply the regulated pressure multiplied by the effective area of the jack piston.

Figure 64: Theoretical pile and jack loads that are controlled by the ER5000 system

For the test setup in Figure 64, the desired cyclic load ratio (the quotient of maximum cyclic load and monotonic pile capacity, $(\frac{Q_c}{Q_s})$) is 0.4. The monotonic capacity of the pile is assumed to be 2.22 kN for this example. Knowing the cyclic load ratio, the desired maximum cyclic pile load is 0.89 kN.

$$Q_{max,pile} = (0.4 * 2.22 \text{ kN}) = 0.89 \text{ kN} \quad (20)$$

For the SPX Flow Power-Team jack, the effective piston area is 14.26 cm², and at a minimum hydraulic pressure of 1379 kPa, the minimum jack load is 1.967 kN.

$$Q_{min,jack} = (1379 \text{ kN/m}^2) * (14.26 \text{ cm}^2) / (10000 \frac{\text{cm}^2}{\text{m}^2}) = 1.967 \text{ kN} \quad (21)$$

Assuming that 1.824 kN of weights is added above the jack to compensate for the minimum load, the minimum cyclic pile load becomes 0.143 kN.

$$Q_{min,pile} = (1.967 \text{ kN}) - (1.824 \text{ kN}) = 0.143 \text{ kN} \quad (22)$$

With these known loads, a sinusoidal equation can be developed to determine the pile loads, regulated pressures, and jack loads present on the system. The amplitude (a) of the function becomes the maximum cyclic pile load minus the minimum cyclic pile load, all divided by two:

$$a = (0.89 \text{ kN} - 0.143 \text{ kN})/2 = 0.3735 \text{ kN} \quad (23)$$

With the amplitude of the sine wave known, the y-axis shift is the minimum cyclic pile load plus the amplitude of the wave:

$$Y - \text{shift} = (0.143 \text{ kN} + 0.3735 \text{ kN}) = 0.5165 \text{ kN} \quad (24)$$

Thus, the cyclic pile load sine equation becomes:

$$y = 0.3735 \text{ kN} * \sin(x) + 0.5165 \text{ kN} \quad (25)$$

Taking the cyclic pile load sine equation, the pile head load with respect to time can be determined for increments of $\pi/4$, knowing that in an eight-second period cycle, the $\pi/4$ values correspond to one-second intervals. This distribution is plotted in Figure 64.

With the pile head load defined for one-second increments of time, the regulated jack pressure can be determined by taking the pile load, adding the load of the weight stack and dividing the net value by the area of the jack piston. For the maximum cyclic pile load, the maximum regulated jack pressure becomes:

$$\text{Max. Pressure} = \frac{0.89 \text{ kN} + 1.824 \text{ kN}}{0.0014258 \text{ m}^2} = 1903.49 \text{ kPa} \quad (26)$$

The full set of regulated pressures for one load cycle by the control system can also be determined and is as follows in Figure 65:

Figure 65: Sinusoidal distribution of regulated jack pressure

With the regulated jack pressure control values determined, the jack loads can also be found by multiplying the hydraulic pressure by the area of the jack. This is detailed as follows for the maximum pressure:

$$Q_{max,jack} = (1903.49 \text{ kN/m}^2) * (0.0014258 \text{ m}^2) = 2.71 \text{ kN} \quad (27)$$

For all of the one second ramp times for the example cyclic load ratio of 0.4 provided, the pile load, regulated jack pressure set-points, and jack load are displayed in Table 7, and are plotted in Figure 64 and Figure 65 :

Table 7: Regulated jack pressures for one cycle of the example cyclic load ratio of 0.4

Required Regulated Jack Pressure for Qc/Qs of 0.4					
Intervals of Pi/4	x (radians)	Time (s)	Pile Load (kN)	Regulated Jack Pressure (kPa)	Jack load (kN)
0	0.00	0	0.52	1641.53	2.34
pi/4	0.79	1	0.78	1826.76	2.60
pi/2	1.57	2	0.89	1903.49	2.71
3pi/4	2.36	3	0.78	1826.76	2.60
pi	3.14	4	0.52	1641.53	2.34
5pi/4	3.93	5	0.25	1456.30	2.08
3pi/2	4.71	6	0.14	1379.57	1.97
7pi/4	5.50	7	0.25	1456.30	2.08
2pi	6.28	8	0.52	1641.53	2.34

The calculations described on the previous pages are tabulated using a Microsoft Excel spreadsheet. Slight changes to the spreadsheet, in terms of cyclic load ratios and monotonic pile capacities, allow the development of a pile loads and regulated jack pressures for any desired combination of loads.

The regulated pressures from Table 7, which are for one cycle, are then placed into the ERTune program to be set-points for the control system to regulate. The ERTune Main Screen is detailed in Figure 66:

Figure 66: ERTune main control screen

As seen in Figure 66, the ERTune main screen features a plot screen in middle of the figure. The plot screen provides a means to track system pressures over time. The left-hand y-axis of this screen displays the pump-jack-regulator line pressure, as determined by the pressure transducer, up to the full system pressure of 69000 kPa (10,000 psi as noted on the plot). It should be noted that this pressure data is sent to the ERTune program as a ratio of the full scale voltage output of the pressure transducer, meaning that the full scale range of actual pressure values that can be read by the

transducer (10,000 psi) need to be set as the y-axis range, so calibrations can be made accurately.

The right-hand y-axis displays pneumatic pressure as presented to the diaphragm. Similarly to the pressure transducer, this scale requires setup to the full scale of expected pneumatic pressure supplied to the system, otherwise the calibration will be incorrect.

On the lower left-hand side of Figure 66, is the PID (Proportional-Integral-Derivative) settings for the control system. These settings determine how the control system will regulate pressure set-point changes, with a series of set-point responses detailed in Figure 67:

Figure 67: ERTune PID set-point pressure change responses (Emerson Process Management, 2015)

If the PID settings are set to process a change too quickly, a pressure overshoot will occur as detailed in Curve A. Pressure “ringing” then ensues as the system tries to correct the pressure to the set-point. If the PID settings are set to process the change too slowly, extended time will be required to reach the set-point, as seen with Curve D. Thus, the PID settings require placement to a range somewhere in between, to minimize

the disadvantages of both setups (Emerson Process Management, 2015). For the purpose of this study, the PID settings were set to the manufacturer supplied settings, and provided sufficient regulation response times and performance.

The upper left-hand section of Figure 66 features the pressure set-point controller for the system. Hydraulic pressures can either be set manually, which proved to be sufficient for the monotonic tests, or set through a user-generated program called a “profile”. The profile setting was implemented for the cyclic tests.

To create a profile, the “Profile” tab at the top portion of the main screen needs to be pressed. This pulls up the profile creation screen, detailing a window where set-point commands can be entered, as shown in Figure 68:

Figure 68: ERTune profile creation screen

The “Profile” window at the top-left portion of the profile screen lists command actions set as pressure set-points. These command set-points are added, modified, or deleted through the “Profile Builder” at the middle-left hand portion of the screen. In the case of the cyclic pile load tests, eight “ramp” commands are included to raise the

system pressure to a set value in a certain time interval. The regulated pressures from Table 7 are input in each ramp function, with a time step value of one second input as well. This creates a full set of pressure set-points for one eight-second cycle. A “loop” command is then added to the end of the program to repeat the cycle for as many cycles as desired. A sample cyclic profile is detailed in Figure 68.

With the cyclic profile created, the hydraulic control regulator will control the pressures to the jack, as long as sufficient pressure is supplied by the pump. A plot of the controlled cyclic pressure during a pile load test, as monitored by the pressure transducer is shown in Figure 69. This profile is the same profile as shown in Figure 68, however with converted units.

Figure 69: Regulated cyclic pressure as produced by the ER5K system during a pile load test

As a comparison, Figure 70 offers a plot of load as determined by the load cell during approximately the same time as the data in Figure 69 is recorded. The difference in time axis values is due to the pressure data acquisition beginning at a different time than the load and displacement data acquisition.

Figure 70: Pile load determined by the load cell over approximately the same time interval as shown Figure 69

Although the minimum pile load was not brought to exactly zero (the minimum load was approximately 0.45 kN), the cyclic load was consistent in application.

5.3.7 Air Compressors

Air compressors are required to power the load application system. As noted earlier, the hydraulic pump required 0.14 to 0.28 m³/minute (5.0-10.0 CFM) at 690 kPa (100 psi) of pneumatic flow and pressure (OTC Tools, 2011) to maintain uniform hydraulic pressure generation. As the hydraulic pump was required to constantly run during testing, a large pneumatic pressure reservoir was necessary to maintain system flow. This requirement proved challenging, as suitable compressors were either too large to be portable, or too small to be powerful enough.

It was decided that a Campbell Hausfeld VT6271 0.11 m³ (30 gallon) air compressor would be suitable to supply pneumatic pressure, as shown in Figure 71. The compressor features a 2759 watt (3.7 hp) electric motor and requires 240 VAC of single phase power. It provides 10.9 m³/minute (10.2 CFM) of flow at 620 kPa (90 psi)

(Campbell Hausfeld, 2016). These specifications met the criteria for the foot pump, and during testing provided sufficient pneumatic power to perform long-term tests.

Figure 71: Campbell Hausfeld VT6271 compressor implemented to power the hydraulic pump

The ER5000 also required pneumatic pressure, although its demands were much lower. During the cyclic tests, the control system was mostly free of excess pneumatic pressure to keep the hydraulic outlet valves open, with very little time in between cycles to rebuild pneumatic pressure.

A Husky Tools model number 3320445 0.017 m³ (4.5 gallon) compressor was obtained to supply this pressure. The compressor is powered by 120 VAC and produces 0.085 m³/minute (3.0 CFM) at 620 kPa (90 psi) (Husky Tools, 2019). Figure 72 details the compressor.

Figure 72: Husky Tools Model No. 3320445 compressor implemented to supply pneumatic power to the hydraulic control system

Standard pneumatic hoses were implemented to carry pneumatic pressure and flow from the compressors to the pump and control system. The compressors were placed as far as possible from the test piles to limit vibrations on the transducers. 120 VAC and 240 VAC power was transmitted from a circuit breaker panel at the site, and thus, no remote power generation was required.

CHAPTER 6- TEST PILES

Details of the test piles implemented as a part of this study were introduced in Chapter 5 with the design of the load frame. Since the performance of the piles are the main focus of this study, further detail into their characteristics is necessary. This chapter will detail physical pile characteristics, installation manner and locations, driving records, and surface roughness calculations.

6.1 Pile Material and Dimensions

The test piles implemented in this study consisted of three sections of HW-series steel casing, which were threaded together. One casing section is detailed Figure 73. Each casing section provided a male and female threaded end, each 76.2 mm in depth, with a total casing length of 1.52 meters. An inner diameter of 101.6 mm and an outer diameter of 114.3 mm are provided (N&N Drilling, 2019). The thread pitch of the male and female ends is 6.35 mm (4 threads per inch).

Figure 73: HW-series casing section properties

The embedded length of the piles (L_{emb}) is approximately 4.27 m, as detailed in the following section. This depth was chosen to produce a slenderness ratio (L_{emb}/D) of 37.4, where D is the outside pile diameter. The piles for offshore jacket structures have been noted to have slenderness ratios ranging between 10 and 40 (Achmus and Mueller, 2010), which this study was designed to evaluate.

6.2 Installation Locations and Methods

The HW-series casing sections were driven in six locations on the site between August 12 and August 23, 2019, with most sections being driven mid-distance between CPT tests locations. The locations of the piles are detailed in Figure 74.

Figure 74: Pile locations at the Allen Harbor testing site. Piles are labelled “P#”

The piles were placed approximately the same distance horizontally from the mean high tide level, in a ring bordering the water’s edge, so as to encounter equal water table

levels with respect to the increasing and decreasing tides. In extreme high tide events, the water's edge reached the pile ring, as witnessed by the tidal beach disturbance in Figure 74.

Spacing of the piles is approximately 3.0 meters, to avoid group effects in the radial soil mass in the surroundings. Depth of embedment is 4.27 meters, to provide 0.3 meters of stickup to ensure the threaded pile cap connection would remain free of soil particles.

A Diedrich D-50 Turbo track rig was implemented in the driving process, operated by Hoffman Environmental Services. Figure 75 provides a view of the track rig.

Figure 75: Diedrich D-50 Turbo track rig implemented to drive the test piles

The rig featured a 1.33 kN hammer, which was dropped from a height of 0.76 meters above the pile head. The hammer was a drop hammer, and implemented a cathead system with three rope loops to release the hammer. The hammer slid around a steel guide pipe, which was threaded into a pile cap, applying the blow to the pile head when bottoming out on the top of the rod. A view of the hammer setup is provided in Figure 76.

The first section of casing to be driven arrived with a driving shoe attached, which was cut off to provide a similar interface and dimension over the pile's full length.

Figure 76: View of pile installation immediately following the application of one hammer blow

The three pile sections per full pile length were driven separately, with the first section driven, followed by the removal of the hammer system and subsequent measurement taken of the soil plug to determine development. Plug measurements were taken in reference to the top of the casing, with a separate measurement taken of the top of casing to outside soil surface, to be able to track the plug surface depth. Figure 77 provides a view of a soil plug after the driving of a casing section.

Figure 77: Soil plug depth as viewed after driving one section of pile

Full driving records for the six piles installed at the site are provided in Appendix C. A record of blow counts over the depth during driving is detailed in Figure 78.

Figure 78: Installation blow counts with depth for each of the piles driven at the Allen Harbor site

As can be noted in the figure, piles P2-P6 encounter similar driving conditions to 3 meters in depth, where pile P2 then encounters less resistance than piles P3-P6 to reach final embedment. Pile P1 noted more initial driving resistance, however, this resistance remained consistent throughout driving, also matching pile P2's resistance from 3

meters to final embedment. This indicates that the western edge of the test site may be weaker than the eastern edge, as larger values of strength at depth causes compounding strength increases for the overall pile. This trend corresponds well with observations made from SPT blow counts in Chapter 3.

As for soil plug development, the final plug length ratio, which is the ratio of the outer soil depth around the pile to the soil level inside of the pile at the final embedment of driving, ranges from 0.73 to 0.81 over all of the piles at the site, as provided in Table 8. This suggests that the piles are primarily coring in action.

Table 8: Pile installation summary and embedment details at final embedment

Pile Number	Total Blow Count	Embedment Depth (m)	Final Plug Depth from Surface (m)	Plug Length Ratio
P1	379	4.267	0.953	0.777
P2	304	4.267	0.978	0.771
P3	370	4.267	0.825	0.807
P4	389	4.267	0.984	0.769
P5	384	4.267	1.137	0.734
P6	362	4.267	0.953	0.777

6.3 Monitoring Well

During the process of driving the test piles at the site, a monitoring well was also installed in the middle of the ring of driven piles, as plotted in Figure 74 and viewed in Figure 79. The well was installed to a depth of approximately 3.57 meters.

The purpose of the well is to track water table elevations during pile testing, so as to be able to determine and correct the level of effect the tides have on the load test results. Water table measurements were made during testing through the use of a flexible tape measure with a weight attached. The measurement was taken of the water table level to the top of the well, and the height of the well from the soil surface was subtracted off this value.

Figure 79: Test piles and monitoring well installed at the site. The view is looking westward across the site, with piles P6, P5, and P4 in the foreground (left to right). The monitoring well, as well as piles P3, P2, and P1 are in the background (right to left)

6.4 Surface Roughness Testing Methodology and Results

It is a well-known concept that the roughness of a pile interface, in combination with the particle size of soil the pile is driven in, will affect capacity and shear band dilation when a pile is under load. This surface roughness of the pile is incorporated in the interface friction angle component of the API RP-2A and UWA-05 pile capacity estimation methods.

It is also known that the surface roughness of a pile is not a static value with time. Gavin et al. (2013) note that pile capacities often increase with time due to many factors, one being chemical reactions occurring with the pile interface, making it change roughness with time.

The roughness of the piles installed at the test site, and more specifically the normalized roughness, is required to be analyzed to determine shear band performance

during loading. As noted previously, the normalized roughness of a pile interface is defined by the following equation (Uesugi and Kishida 1986):

$$R_n = \frac{R_{\max}(L = D_{50})}{D_{50}} \quad (28)$$

Where $R_{\max}$ = the total roughness of the steel interface over the length of D_{50} of the soil.

To evaluate the normalized roughness of the test piles, one section of casing was installed to a depth of approximately 1.45 meters, with a driving shoe attached to the bottom of the casing. The casing was installed between the locations of geotechnical borings URI-2 and URI-6 in Figure 74, on August 12, 2019. The casing remained installed at the site for 39 days, following removal on September 20, 2019. To do this, the soil around the pile was removed, and it was jacked out of the hole. The ground water table was encountered during removal approximately a meter deep.

The removed casing section was cut into four sections for easier transportation and testing, and brought to the URI Metrology Laboratory for surface profile testing. By visual inspection, there were clear increases in surface roughness on the casing deeper than 0.89 meters in embedment depth, where the intertidal zone began.

Surface profile testing began on the casing section lengths using a MarSurf XR 20 metrology device, which implemented a probe to trace surface features over a given length. Before testing occurred, the MarSurf XR 20 device was calibrated using a calibration block of known roughness. The physical calibration process is detailed Figure 80 (a), with the calibration recording in process in Figure 80 (b). During tracing, the roughness program records data for the full range of the length of travel (L_t) and produces what is known as a primary profile (Mitutoyo Corporation, 2014). The beginnings of primary profile of the calibration block are shown in Figure 80 (b).

(a)

(b)

Figure 80: Calibrating the MarSurf XR 20 metrology device (a) with a calibration block of known roughness, with (b) calibration scan in progress shown

For the pile roughness tests, the roughness program then splits the primary profile results into seven divisions of the total traversing length of the scan, and then discards the first (pre-travel length) and last (post-travel length) divisions. This leaves the system with five divisions of evaluation length, which are filtered using a Gaussian low pass filter of $8.0\ \mu\text{m}$ to remove short-wave roughness data scatter. A high pass filter of $2.5\ \text{mm}$ is then applied to remove surface waviness, to generate a roughness profile (Mahr Federal Inc., 2017) (Mitutoyo Corporation, 2014).

Two traces were taken from each embedded pile section length, from representative locations on the top and bottom of the length (producing Readings 1-6). One additional trace was performed on the stickup portion of the pile (Reading 0). All of the readings performed, in approximate locations on the pile surface, are detailed in Figure 81:

Figure 81: Surface roughness reading locations, as well as installation location details

The MarSurf XR 20 device implemented a trace length of 17.5 mm for each scan performed, resulting in an evaluation length of 12.5 mm. Probe traversing speed was 0.5 mm/s. Figure 82 details a view of the profile testing setup for Reading 6 at the bottom of the embedded section.

The casing section was placed in a v-shaped cradle, designed to support cylindrical objects. The probe was then very carefully lowered onto the surface of the pile, upon which tracing began.

Figure 82: Roughness testing of the bottom of the casing section (Reading 6)

The roughness program automatically converts the traversing trace length (17.5 mm) into an evaluation trace (12.5 mm), and then filters the data to produce a roughness profile. The roughness profile generated for Reading 6 is shown in Figure 83, with full sets of roughness profiles produced in Appendix D:

Figure 83: Roughness profile generated for Reading 6 at the bottom of the casing section

As well as producing the roughness profile for the scan, the program calculates and lists more common roughness parameters determined from the evaluation length. The parameters calculated are preset by the user. For the testing performed as a part of this study, the average (R_a), mean root squared (R_q), mean depth (R_z), and total height (R_t)

roughness parameters are recorded. For Reading 6, these parameters are as follows in Table 9:

Table 9: Roughness parameters determined for Reading 6 on the casing section

Profile: R [LC GS: 2.5 mm]		
Ra	8.97	µm
Rq	11.60	µm
Rz	52.44	µm
Rt	76.34	µm

This testing procedure and analysis was performed for all pile sections. Table 10 presents roughness parameters for all sections:

Table 10: Roughness measurements determined for each roughness reading section

Pile Reading Data		Pile Roughness Measurements			
Reading Number	Approx. Depth of Embedment (m)	Ra (µm)	Rq (µm)	Rz (µm)	Rt (µm)
0 (Not Embedded)	-0.075	2.362	3.038	13.804	17.335
1	0.100	1.210	1.941	14.014	19.259
2	0.400	1.248	1.925	9.893	16.244
3	0.500	0.759	1.063	8.044	12.446
4	0.800	0.919	1.668	12.610	17.468
5	1.000	10.550	13.154	53.565	63.573
6	1.300	8.973	11.596	52.438	76.341
Definition:	Soil Surface is at 0 meters	Arithmetic Mean	Root Mean Squared	Mean Depth	Total Height

In computing the normalized roughness of the pile-soil system, the total roughness over an evaluation length equal to D_{50} is required. In the case of data presented in Table 10, the total roughness values are taken over the evaluation length of 12.5 mm, much larger the D_{50} of the site soil. In this case, the mean roughness depth (R_z), which takes the mean vertical distance between the highest peak and the lowest valley for all five of the evaluation length divisions (2.5 mm in length each), provides the most similar roughness value (Mahr Federal Inc., 2017).

Taking the mean roughness depth for each reading, and dividing it by the lower and upper bounds for D_{50} for the site soil from Figure 29 (0.17 mm and 0.60 mm, respectively), provides a high and low value for the normalized roughness for each

reading. Table 11 presents the high and low normalized roughness values for each reading of the casing section.

Table 11: Normalized roughness values calculated using R_z for each of the readings

Pile Reading Data		Roughness-Soil Calculations	
Reading Number	Approximate Depth of Embedment (m)	Rn High Value	Rn Low Value
0 (Not Embedded)	-0.075	0.081	0.023
1	0.100	0.082	0.023
2	0.400	0.058	0.016
3	0.500	0.047	0.013
4	0.800	0.074	0.021
5	1.000	0.315	0.089
6	1.300	0.308	0.087
Definition:		Normalized Roughness ($R_z/D50$)	
Soil Surface at 0 meters		D50 Minimum (mm)	0.17
		D50 Maximum (mm)	0.6

As can be seen visually in Figure 81 and noted numerically in Table 11, the normalized roughness was much larger in the intertidal zone (Readings 5 and 6). This definition of intertidal zone was established since the geotechnical borings in the area noted the high groundwater table at approximately 0.91 m deep, which was confirmed upon removing the casing section for testing. The U.S. National Oceanic and Atmospheric Administration (NOAA) maintains tidal data buoy QPTR1-8454049 approximately five kilometers away from the test site, which recorded water level changes of more than 1.3 meters over tidal cycles (NOAA, 2019). Taking into account the 0.91 m deep high water table in the area, and the 1.3 meters or more of tidal drawdown change, it can be said that there was no area on the casing section that was fully submerged at all times. This, in combination with the roughness determinations from the intertidal zone, suggest that the possibility for chemical changes (such as rust) occurring to the pile interface are greater in the intertidal zone.

In terms of comparisons on normalized roughness values, it was noted in previous study by Mortara et al. that a “smooth” interface presented a normalized roughness of 0.030 and a “rough” interface presented a normalized roughness of 0.157 (2007). Comparing with the data in Table 11, the section of the pile above the water table would be considered “smooth”, while the portion within the intertidal zone would be considered “rough”.

Since this change in roughness was noted after 39 days of installation, it can be surmised that the piles tested during one week after installation may still incur “smooth” cyclic performance over the full interface, as the chemical reactions creating roughness may be just initializing. This leads to the inference that the piles one-week after installation may encounter a reduction of normal stress during cycling, which is sustained upon monotonic loading after cycling, leading to lower post-cyclic capacities.

Analyzing the data in reference to the longer term ten-week tests, sufficient chemical reactions may have occurred along the pile interface in the intertidal zone to induce greater roughness, while similar roughness is predicted below the water table. This suggests that the piles may experience some recovery of radial normal stresses during cycling in the intertidal zone, leading post-cyclic monotonic strengths to be higher than those at the one-week testing range.

CHAPTER 7- TEST MATRIX AND RESULTS

This chapter will first outline the pile load tests performed by pile number, load type, and date. The proceeding section will then outline the pile load step methodology. The following subsections of the chapter will then detail the results from individual piles. It is to be noted that Pile P2 was tested multiple times, once monotonically to failure, and then twice afterwards cyclically. Thus, the pile P2 subsection contains multiple sets of results. Tidal displacement corrections will also be discussed, as applicable to the individual tests.

Results will first be presented in a set of plots detailing the full test results (full loads and displacements), with more detailed results sets presented as necessary.

7.1 Test Matrix

One of the main goals of the study is to determine static and cyclic pile load responses on the test piles, while accounting for aging effects. To accomplish this, a series of monotonic and cyclic load tests were performed approximately one week (7-8 days) and ten week (71-87 days) intervals after piles were installed. Table 12 details the test type, dates of installation and testing, as well as aging time at time of testing for each of the piles.

Table 12: Pile test matrix detailing test type, date of installation and testing, and aging time

Pile Number	Test Type	Date of Installation	Date of Test	Aging Time (days)
P1	Monotonic	8/12/2019	8/20/2019	8
P2	Monotonic	8/15/2019	8/22/2019	7
P2 (pre-failed)	Cyclic-Monotonic	8/15/2019	8/23/2019	8
P2 (pre-failed)	Cyclic-Monotonic	8/15/2019	8/29/2019	14
P3	Monotonic	8/15/2019	10/25/2019	71
P4	Cyclic-Monotonic	8/23/2019	8/31/2019	8
P5	Monotonic-Cyclic-Monotonic	8/23/2019	11/8/2019	77
P6	Cyclic-Monotonic-Cyclic-Monotonic	8/23/2019	11/18/2019	87

In terms of one-week tests, piles P1 and P2 were tested monotonically, at seven and eight days of aging. Pile P4 was then tested cyclically at eight days of aging, followed by a post-cyclic monotonic test to determine capacity. Pre-failed pile P2 was also tested cyclically and then post-cyclic monotonically at eight and thirteen days after installation.

As for ten-week tests, pile P3 was tested seventy days after installation monotonically, to determine a baseline ten-week aged capacity for all of the remaining piles. Piles P5 and P6 were then tested in a monotonic-cyclic-monotonic method, seventy-seven and eighty-seven days after installation, respectively. The test arrangement, providing a pre-cyclic monotonic portion, was implemented with analysis of pile head creep rates. This was in an attempt to determine a load level where the piles became metastable or unstable, so that cyclic load ratios could be applied near those levels.

7.2 Load Step Methodology

In a monotonic pile load test, the magnitude of load steps and the time the loads are held for are crucial parameters to set. A general set of rules for the creation of load steps for a single pile load test are to ensure the load increments are equal in magnitude change, applied at a consistent rate, and are applied for similar lengths of time (Fellenius, 2020). If these conditions were not met, data from the tests would become difficult to analyze for failure. ASTM D3689 recommends for a tensile “Quick Test” that load steps are held for a period of 4 to 15 minutes, the time period chosen remaining constant (ASTM, 2013).

In the case of the monotonic load tests at the Allen Harbor site, on piles P1, P2, and P3, the loads were held for 5 minute intervals, to ensure drained loading conditions, and were distributed into a series of consistent magnitude load steps.

As for the cyclic tests, cyclic load ratios during testing were estimated from the similarly aged monotonic capacities of the surrounding piles. As noted in Chapter 5, the loads were applied with a period of eight seconds (0.125 Hz), similar to the 0.1 Hz expected from ocean waves (Jardine et al., 2012). The goal of testing was to apply cyclic loads for 10,000 cycles. Cyclic load ratios started generally lower for all of the tests, around a value of 0.4, and were gradually increased if displacements were negligible.

One distinction in this chapter is that the cyclic load ratios, in terms of results, are defined as a given cyclic load, with respect to the ultimate post-cyclic monotonic capacity of the pile. During testing, cyclic load ratios were estimated based on the estimated ultimate capacity of the nearby monotonically tested piles.

7.3 Pile P1 Eight-Day Monotonic Load Test

Pile P1 was tested monotonically on August 20, 2019. Initial pile capacity was estimated using the UWA-05 estimation provided in Chapter 4, and was approximated to be 50 kN for full-scale piles. In the case of this expected capacity, the OTC Tools Power-Twin 30 Ton ram and the Omega Engineering LC8250-875-30K load cell were implemented. A series of 4.2 kN load steps were created for the test, with a five minute hold time between steps.

While setting the first tensile load step, the hydraulic regulator briefly overshot the load control and failed the pile.

Figure 84 details the load-time, displacement-time, and load-displacement plots for the test. The maximum load determined from this test was 5.84 kN at approximately 0.92 mm of displacement, with the failure overshoot shown at approximately 0.0025 h.

Figure 84: Pile P1 8-day monotonic test results detailing the full test results

This result was unexpected, and was an order of magnitude lower than capacity estimated through the UWA-05 method. Since the hydraulic ram and load cell implemented for this test were rated for capacities much larger than the capacity of the pile, a monotonic test on pile P2 was warranted to determine if this result was typical.

7.4 Pile P2 Seven-Day Monotonic Load Test

Test pile P2 was monotonically loaded on August 22, 2019, seven days after it was installed. In response to the lower capacity determined in pile P1, the SPX Flow Power Team RH102 cylinder was implemented, as well as the Omega Engineering Inc. LC8151-750-10K load cell. The test pile was loaded in approximately 0.4 kN load steps,

which were held for five minutes. All of the tests following also implemented the same equipment.

Pile P2 reached a peak load of 7.01 kN at 2.75 mm of displacement and had a load of 5.83 kN at a displacement of 1% of the pile diameter. The displacement of one percent of the pile diameter was implemented so as to determine a capacity near the fully-mobilized pile capacity, which is especially important where a strain-hardening pile performance results.

The full set of data including load-time, displacement-time, and load-displacement results are found in Figure 85:

Figure 85: Pile P2 load-time, displacement-time, and load-displacement data for the seven-day monotonic test

The ultimate capacity of pile P2, being 7.01 kN, is 20.0% higher than the eight-day capacity of pile P1. However, the result indicates that the seven-day capacities of the test site are significantly lower than expected. A comparison plot of the monotonic load-

displacement plots for piles P1 and P2 is provided in Figure 86. The stiffness of both test piles before yielding are similar.

Figure 86: Monotonic load-displacement results for piles P1 and P2, enlarged to show detail

7.5 Pile P2 Eight-Day Cyclic-Monotonic Load Test

In order to better prepare for the load tests that were to require much larger numbers of cycles, a series of tests under lower cycle numbers were performed on pile P2 after the seven-day monotonic test had occurred. The first one of these tests was performed one-day after the monotonic test, thus becoming an eight day test. The date of this test was August 23, 2019.

Approximately 300 cycles were applied for this test, with a monotonic portion included after the test to gauge capacity. Figure 87 presents the full set of test results.

Figure 87: Pile P2 eight-day cyclic test full set of results

The pile displaced very little during the cyclic portion of the test, with a maximum displacement of 0.1 mm achieved. The post-cyclic monotonic test presented a slight strain-hardening response, with a capacity determined at 1% of the pile diameter (or 1.14 mm) of 5.36 kN. The ultimate post-cyclic monotonic pile capacity was 5.83 kN at 22.84 mm of total displacement (22.74 mm of net displacement). Thus, the approximate cyclic load ratio was 0.46, based on the post-cyclic capacity and maximum cyclic load of 2.62 kN. Figure 88 presents the test data to show the monotonic capacity and cyclic displacement in a more enlarged view.

Figure 88: Pile P2 eight-day cyclic test at a scaled-in view

The post-cyclic capacity of the pile at 1% pile diameter displacement is 76% of the monotonic capacity tested the day before, indicating some loss of strength had occurred due to mobilization of the pile or cyclic effects.

7.6 Pile P2 Fourteen-Day Cyclic-Monotonic Load Test

A second cyclic test was performed on pile P2 on August 29, 2019, fourteen days after the pile was installed. The previous compressive weight stack was stolen from the test site, so another stack of weights, similar in total weight, was obtained. Approximately 1250 cycles were applied to the pile over the course of five separate cyclic load ratios. At the highest cyclic load ratio, the pile failed, with a load at 1% pile diameter displacement of 6.70 kN. The pile displaced cyclically for 26.45 mm, and was halted. A post-cyclic monotonic load test was then performed, with a load at an additional 1% pile diameter of displacement being 5.90 kN. The pile was slightly strain-

hardening over the monotonic portion, with a large strain capacity produced of 6.94 kN at a total displacement of 47.3 mm (net 20.80 mm). A full plot of test results is detailed in Figure 89:

Figure 89: Pile P2 cyclic and post-cyclic monotonic results showing cyclic failure

Based on the data presented in Figure 89, the cyclic load ratios that were placed on the pile with time, with respect to the large strain monotonic pile capacity, were 0.53, 0.64, 0.75, 0.84, and 0.97, with the cyclic failure occurring at the 0.97 cyclic load ratio.

The large strain capacity of the pile, being 6.94 kN is 99% of the seven-day monotonic capacity (7.01 kN), suggested the pile had regained most, but not all of its capacity after allowing six days of aging and through cyclic action.

7.7 Pile P3 Seventy-One Day Monotonic Load Test

Test pile P3 was monotonically tested on October 25, 2019, seventy-one days after installation, so as to determine the magnitude of strength increase over time at the test site.

The test pile was loaded in a series of 0.95 kN load steps that were held for five minutes each. One unload-reload cycle occurred due to an inadvertent loss of system control pressure. The pile reached a load of 27.04 kN at a displacement of 1.14 mm (1% pile diameter), and reached an ultimate capacity of 27.66 kN at a displacement of 4.0 mm (3.50% pile diameter). A full set of test results is provided in Figure 90:

Figure 90: Pile P3 seventy-one day monotonic testing results

In comparing the seventy-one day monotonic results with those from piles P1 and P2 at displacements of one percent of the pile diameter, the capacity of pile P3 is 463% of the values recorded for the other piles. Piles P1 and P2 were determined to have

similar capacity to each other at this level of displacement. A plot of all three monotonic load tests is found in Figure 91, detailing results before pile “plunging”, and after slightly after plunging.

Figure 91: Load-displacement plot of the monotonic capacities of piles P1 and P2, aged one-week, and pile P3, aged seventy-one days

As can be seen in Figure 91, the stiffness before yield of the pile P3 is much higher, at 40 kN/mm. The stiffness of piles P1 and P2 is 20 kN/mm, as found in Figure 86.

7.8 Pile P4 Seven-Eight Day Cyclic-Monotonic Load Test

Test pile P4 was loaded cyclically on August 30 into August 31, 2019, with a post-cyclic monotonic test performed immediately after the cyclic portion. This makes the pile aged seven days upon commencement of the first cyclic loads, and eight days at completion of testing.

A series of six load steps were performed for the test with an approximate number of 10,150 cycles being placed over all six load steps. Several tidal cycles occurred over the course of the test, with 0.5 mm of loss of displacement occurring at the peak high

tides. This large period cyclic loss of displacement is theorized to be due to surrounding soil and load frame buoyancy, and was recovered when the tides decreased.

The raw displacement data for the test appears in Figure 92, with several dips in displacement noted from the tidal changes. During the test, the water table level was also recorded from the monitoring well located about two meters away from the pile. The water table readings were noted with time, with the levels presented in Figure 92.

Figure 92: Pile P4 cyclic test raw displacement and water table measurements

As is clear in Figure 92, the significant increases in the water table correlate well with the drops in displacement. To correct the displacement data, Dr. Aaron Bradshaw created a Matlab script that implements the following consolidation settlement equation to solve for the settlement occurring at the test site:

$$\Delta H = H_0 * C_{re} * \log\left(\frac{\sigma'_f}{\sigma'_i}\right) \quad (29)$$

Where ΔH = the settlement of one individual sublayer, H_0 = the initial sublayer height, C_{re} = the consolidation index iterated for to correct the displacements, σ'_f = the final effective stress at the midpoint of each layer, and σ'_i = the initial effective stress at the midpoint of each layer.

Input parameters are a list of layer midpoint depths, the assumed moist unit weight of the soil, the assumed saturated unit weight, the unit weight of water, the chosen sublayer thickness, and the assumed consolidation index (to be iterated for).

The program uses a for loop, with “if” and “else” nested conditional statements to determine the initial effective stress over the list of midpoint depths. The program then determines the final effective stress at every interpolated water table measurement in Figure 92 by using a nested for loop and nested “if” and “else” conditional statements to determine where the water table lies over the list of sublayer midpoints. With the determination of the water table level, the final effective stress was computed. The initial and final stresses were input into the consolidation settlement equation noted on the previous page and summed, to determine the settlement of the soil profile with time. This correction is detailed in the middle plot of Figure 93:

Figure 93: Pile P4 cyclic test water table correction, where the middle plot details the estimated ground heave, and the lower plot details the corrected displacement

The C_{re} term in the consolidation equation is then iterated for until the settlement correction closely matches the loss of displacement from the pile load test results. The settlement calculated for was then added to the raw displacement values from the test data to produce a corrected displacement plot as seen in the lower plot of Figure 93. Some displacement peaks were produced by the correction, mainly in areas where the tide was above the soil surface.

With this correction, and the data from the relatively short monotonic test added in, the full plot for the load test for pile P4 is presented in Figure 94:

Figure 94: Full monotonic and corrected cyclic test results for the seven and eight day pile P4 load test

Combining the results from Figure 93 and Figure 94, approximately 0.70 mm of average displacement was induced on the pile through all 10,150 cycles of loading. During the post-cyclic monotonic load test, a peak load of 23.03 kN was recorded, at 15.30 mm of displacement. At a displacement of 1.14 mm (1% pile diameter) past the plastic displacement created by the cyclic test (0.84 mm), the load was 17.26 kN. This was approximately the yield point for the pile. A plot of the load-displacement data for the test up to the peak load is found in Figure 95:

Figure 95: Pile P4 corrected cyclic and post-cyclic monotonic load-displacement results up to the peak load

The post-cyclic ultimate capacity of 23.03 kN was unexpected, being 329% of the pile P2 monotonic ultimate capacity, and 394% of pile P1 ultimate capacity. Explanations for these results are found in the following chapter.

As for cyclic load levels, based on the ultimate pile capacity determined, ratios of 0.18, 0.21, 0.24, 0.30, 0.32, 0.36, and 0.41 were induced on the pile. These cyclic load ratios created minimal levels of displacement, with 0.05 mm of displacement occurring during the 0.41 load ratio portion and 0.84 mm of displacement occurring over all of the cycles.

7.9 Pile P5 Seventy-Seven Day Cyclic-Monotonic Load Test

Two series of cyclic load tests occurred on untested piles aged ten weeks after installation. The first of these tests occurred on November 8, 2019 on test pile P5. The test format had evolved slightly at the commencement of the ten-week cyclic tests. As part of the goal of the research was to evaluate the cyclic degradation exhibited on the

test piles, the previous tests under lower cyclic load ratios were presenting “stable” displacement rates. As cyclic failure was sought, a monotonic load portion of testing was added before the cyclic portion, so as to evaluate the displacement rates with additional load steps, in an attempt to determine the pile’s monotonic capacity.

Air temperatures also presented a concern for the test. At the beginning of testing, air temperature was recorded at the NOAA instrumented buoy QPTR1-8454049, approximately five kilometers from the test site, and recorded a high temperature of approximately 3.0 degrees Celsius and steadily decreasing to 0.0 degrees Celsius at the end of the test (NOAA, 2019). With these cold temperatures causing the air compressors to produce a large flow of cold air, and with the compressed air removing further heat from the surroundings, the hydraulic fluid became more viscous. It was theorized the pressurized seals on the pump contracted, causing periods of uncontrolled pressure to the system. A heated blanket around the pump mitigated some of these concerns, providing sufficient heat to overcome compressed air heat losses. A tarp was required to cover the test equipment, as the 8.0 m/s north-westerly wind speed (NOAA, 2019) caused the loose sand at the surface of the site to become airborne and cover the equipment.

The pre-cyclic monotonic loads at the test site were held for approximately 5 minutes each step, reaching a load of 23.4 kN before the displacement rate reached a yielding point suggesting failure was approaching, as detailed in Figure 96:

Figure 96: Displacement rate of pile P5 pre-cyclic monotonic compared to the displacement rate of pile P3

Displacement rates were previously determined for the pile P3 monotonic test, which allowed comparison to pile P5 data, to determine the yield point. On both the plots of displacement rate in Figure 96, a clear yielding point is found at a load of 23 kN, and displacement rate of approximately 0.6 mm/h. The pre-cyclic monotonic loading on pile P5 caused a maximum displacement of 0.92 mm, of which 0.45 mm was plastic when the loading was terminated.

A series of three cyclic load steps were performed, with two sets of similar cyclic load ratios, and one higher portion where the load was less controlled. These portions are visible in Figure 97, and a total of approximately 2790 cycles were placed on the pile.

Figure 97: Full test data for the pile P5 seventy-seven day monotonic-cyclic-monotonic load test

Much of the appreciable cyclic displacement for the test was during the less controlled loading portion (where cycles with higher loads were created), with 1.6 mm of displacement occurring during that ratio. The ultimate cyclic load in that portion was 33.3 kN (for one cycle), with an approximate average of 30.0 kN for the other cycles in that ratio. A total of 2.3 mm of plastic displacement occurred over the full cyclic portion, meeting the criteria for pile failure. Using the post-cyclic monotonic load as a reference, cyclic load ratios the pile experienced ranged from 0.67 prior to the uncontrolled portion, and 0.84 after the uncontrolled portion. The uncontrolled portion, which lasted for several cycles, applied a cyclic load ratio of 1.07, meaning some cyclic degradation occurred in the cyclic portion, which was displayed post-cyclic monotonic capacity.

In terms of post-cyclic monotonic capacity results, an ultimate load of 29.2 kN at 27.0 mm of net displacement. 27.0 kN of load were placed to create an additional 1.14

mm of displacement past the displacement generated during the cyclic portion, with the loading stiffness of the monotonic curve matching the loading stiffness of the cyclic curves.

Figure 98: Pile P5 seventy-seven day monotonic-cyclic-monotonic test results up to the peak monotonic load

It was determined that tidal buoyancy corrections were not necessary for the data, as the tide never reached the load frame setup, and the water table readings deviated between 1.2 m and 0.81 m below the ground surface for the entire test, causing minimal soil consolidation effects. A view of the high tide mark at the site, in comparison to the load frame is provided in Figure 99:

Figure 99: High tide mark (darker colored soil near waterline) as seen during the pile P5 cyclic test

7.10 Pile P6 Eighty-Seven Day Cyclic-Monotonic Load Test

Test pile P6 was tested for cyclic loads eighty-seven days after installation on November 18, 2019. Similar procedures were followed as were implemented in the seventy-seven day cyclic load test on pile P5, with a pre-cyclic monotonic loading proceeding the cyclic portion of the test. One additional component was added, which consisted of cyclic loading before the pre-cyclic monotonic portion, so as to evaluate cyclic effects on capacity of the untested pile before monotonic loading at levels similar to the one-week tests. The main high-level cyclic portion of the test will be referred to as the “secondary cyclic portion”.

Temperature conditions were slightly more moderate at the site, with a low temperature of 3.0 degrees Celsius at the start of testing, and becoming warming to about 5.0 degrees Celsius overnight and remaining that temperature for the rest of the test (NOAA, 2019). As the air compressors were still producing cold air that was

removing heat from the hydraulic pump, a heated blanket was implemented to try to mitigate the seals from contracting and hydraulic fluid from becoming viscous. Winds were 6.0 m/s at the start of the test, becoming faster to 10 m/s by the end, with periods of rain and snow, requiring a tent to keep the equipment dry (NOAA, 2019). Tidal fluctuations were also significant, with the high tide surpassing the test frame and pile, and buoyancy calculations were required to correct displacement results. Figure 100 provides a view of the test setup and high tide level:

Figure 100: Pile P6 eighty-seven day monotonic-cyclic-monotonic test setup

As tidal fluctuations were notable during the test, considerable buoyancy effects on pile displacement were produced during the cyclic portion, as displayed in Figure 101:

Figure 101: Pile P6 displacement-time and water table-time data for the initial cyclic, pre-secondary cyclic monotonic, and secondary cyclic test portions

Using the displacement correction Matlab code created by Dr. Aaron Bradshaw described in Section 7.8 and Equation 29, the soil profile surrounding the pile was discretized into several sublayers of 0.5 m thickness. The initial and final stresses were calculated with water level changes for each observed water table data point, and input into the consolidation settlement equation outlined in Section 7.8. The consolidation index parameter term, C_{re} , was iterated for to correct the displacement, and was found to provide the correction at a value of 0.009, similar to the correction in Section 7.8. Figure 102 displays the corrected displacement data for the test:

Figure 102: Pile P6 load and corrected displacement results

As displayed in Figure 102, the tidal buoyancy correction removed most of the negative displacement data. Over the test, there were eight separate cyclic load segments, counting both the initial and secondary cyclic load portions. Through all of the segments, approximately 7650 cycles were created on the pile.

The initial cyclic load segment, with a maximum load of 8.2 kN, and a maximum displacement of 0.40 mm, had a plastic displacement of 0.15 mm. 530 cycles were placed on the pile during this segment, with the initial cycle creating 88% of the total displacement, and further cycles becoming more stable.

With stable pile results determined for the initial cyclic portion, the monotonic portion was commenced until a displacement rate was reached, which would indicate the pile is close to failure. A maximum load of 25.61 kN was placed on the pile before the target displacement rate was reached. This monotonic portion induced 0.60 mm of

net displacement past the plastic amount achieved through the initial cyclic portion. 0.08 mm of the net monotonic displacement was plastic.

During the secondary cyclic load portion, in which approximately 7120 cycles occurred, maximum loads ranged from 26.2 kN to 40.0 kN (the 40.0 kN occurred during an uncontrolled load spike, most of the maximum loads were 29.9 kN in magnitude). Over all of the cycles, 0.90 mm of net cyclic displacement occurred past the pre-cyclic monotonic plastic displacement, of which 0.52 mm was plastic. The uncontrolled load spike caused a 0.16 mm net increase in displacement, of which 0.04 mm was retained upon correction of the load. Data was lost from 11.83 to 12.84 hours into the test (approximately 230 net cycles) when the data acquisition system crashed. The pile still encountered cyclic loading in this time period, and displacements were similar to those at the beginning and end of the segment.

Upon post-cyclic monotonic loading of the pile, the control system would not allow the load to be placed above 22.2 kN, most likely due to clogged exhaust valves, and the hydraulic regulator was removed. The pile was then tested until failure by steady increases in jack pressure applied from the foot pump, achieving a maximum load of 36.45 kN at 2.15 mm of total displacement (0.95 mm of net displacement past the maximum cyclic displacement). This was also approximately the load where the net monotonic displacement of one percent of the pile diameter (1.14 mm) was encountered.

Cyclic load ratios for the test were 0.22 for the initial cyclic portion, and 0.82 for the secondary portion, except where the load spike occurred. It is worthy to note that the load spike induced a load of 40.0 kN on the pile, and was corrected quickly so that the plastic displacement was 0.04 mm. This load is 3.55 kN larger than the maximum

load encountered during post-cyclic monotonic section. Figure 103 details the load-time, displacement-time, and load-displacement data for the full test:

Figure 103: Full load-time, displacement-time, and load-displacement plots for the pile P6 cyclic test

The load displacement data, in an expanded view, is detailed in Figure 104:

Figure 104: Pile P6 load-displacement plot, showing the maximum uncontrolled cyclic load reached (40 kN), as well as the post-cyclic monotonic peak capacity

7.11 Load Test Results Summary

Figure 105 and Figure 106 provide summaries of load-displacement data for all tests:

Figure 105: Full load-displacement data of all one-week load tests

Figure 106: Full load-displacement plots of all ten-week load tests

Table 13 presents capacity and displacement results for pile load tests:

Table 13: Summary of capacity and displacement results for all pile load tests

Summary of Results for all Pile Load Tests					
Pile No.	Test Description	Aging Time (days)	Ult. Post-Cyclic/Mono. Capacity (kN)	Net Disp. at Ult. Capacity (mm) (1)	Mono. Capacity at 1% Net Pile Diameter Disp. (kN) (1)
P1	Monotonic	8	5.84	0.92	5.64
P2	Monotonic	7	7.01	2.75	5.83
P2	Cyclic-Monotonic	8	5.83	22.74	5.36
P2	Cyclic-Monotonic	14	6.94	20.80	5.90
P3	Monotonic	71	27.66	4.00	27.04
P4	Cyclic-Monotonic	8	23.03	15.30	17.26
P5	Monotonic-Cyclic-Monotonic	77	29.2	27.00	27.00
P6	Cyclic-Monotonic-Cyclic-Monotonic	87	36.45	0.95	35.64
Pile No.	Test Description	No. of Cycles	Max. Cyclic Load Ratio (2)	Cyclic Disp. (mm)	
P1	Monotonic	N/A	N/A	N/A	
P2	Monotonic	N/A	N/A	N/A	
P2	Cyclic-Monotonic	300	0.46	0.1	
P2	Cyclic-Monotonic	1250	0.97	26.45	
P3	Monotonic	N/A	N/A	N/A	
P4	Cyclic-Monotonic	10150	0.41	0.84	
P5	Monotonic-Cyclic-Monotonic	2790	1.07	2.26	
P6	Cyclic-Monotonic-Cyclic-Monotonic	7650	0.82	1.30*	
Notes:	(1) Net disp. is calculated based on displacements of 1.14 mm solely in the monotonic portion (1% pile diameter = 1.14 mm = criteria for pile failure)				
	(2) Cyclic load ratio is defined as Q_{cyc}/Q_T , where Q_{cyc} = maximum cyclic load sustained for more than one cycle, Q_T = Ultimate post-cyclic monotonic capacity				
	* Includes initial cyclic loading and secondary cyclic loading displacement (0.40 mm and 0.90 mm, respectively)				

CHAPTER 8- DISCUSSION OF RESULTS

The results presented in Chapter 7 provide a data set with many variables to consider in terms of evaluating results. These factors include: site variability, aging time, load type applied, numbers of cycles applied, whether the pile was retested or not, and drainage conditions. The goal of this chapter is to provide a full discussion of the results based on lateral drainage response, displacement rates encountered, failure and response with aging.

8.1 Lateral Drainage Analysis

8.1.1 Lateral Drainage Analysis Methodology

One of the main concerns in analyzing the soil response to the cyclic loading created in this study is the drainage conditions in the near pile soil. If undrained cyclic loading occurred, the results may be subject to analysis using an undrained shear strength “alpha” type manner. If drained loading occurred, analysis using a “beta” type framework would be applicable.

The main concern presented from the soil at the URI Allen Harbor site is the presence of silty-sand mixes in the substrata. Sandy soil itself is known to produce drained loading conditions during CPT cone advancement and pile loading, whereas soil containing silt is known to produce partially drained responses (Campanella et al., 1983) (Bradshaw et al., 2012).

In order to evaluate the drainage conditions for the piles at the URI test site, comparisons can be made with lateral drainage analyses determined from scaled CPT pore pressure dissipation tests, one of which was performed during site investigations in the area.

In a CPT pore-pressure dissipation test, in which the cone advancement of 2.0 cm/s is halted and pore pressure dissipation with time is monitored, the following relation has been proposed by Finnie and Randolph to determine drainage rates (1994):

$$V = vd_c/c_h \quad (30)$$

Where V = dimensionless penetration rate, v = CPT cone penetration rate (2.0 cm/s), d_c = CPT cone diameter (4.4 cm), and c_h = horizontal consolidation rate.

From the CPT pore pressure dissipation tests, pore pressure with time is recorded, from which the time for fifty percent dissipation can be determined from a logarithmic-time analysis approach. With the time for fifty percent of dissipation known (t_{50}), the following equation can be used to approximate the horizontal consolidation rate using a 15 cm² cone area that was implemented during the URI Allen Harbor site investigations (Robertson, 2010a):

$$c_h = 1.5(1.67 \times 10^{-6})10^{(1-\log(t_{50}))} \quad (\text{m}^2/\text{s}) \quad (31)$$

The horizontal consolidation rate can then be inserted into Equation 30 to calculate the dimensionless penetration rate, which for values less than 0.05 are fully drained, and above 10 are fully undrained, with values in between classifying as partially drained (Kim et al., 2008).

This process is sufficient for CPT testing, where the cone is being rapidly advanced at 2.0 cm/s, and a more undrained response may be achieved, however is also useful in determining if rapidly altering pile loading was causing undrained loading. In this manner, Equation 30 can be scaled to the pile geometry and load test displacement velocity, with the horizontal drainage rate remaining constant. Since the piles at the URI

Allen Harbor site were open-ended, an equivalent diameter approximation to a close-ended pile horizontal consolidation relation was proposed by Randolph as (2003):

$$d_{eq} \approx 2\sqrt{(dt)} \quad (32)$$

Where d_{eq} = is the equivalent close-ended pile diameter for an open-ended pile, d = is the open-ended pile diameter, and t = is the wall thickness. As such, this process allows the determination if the load test results induced drained, partially drained, or undrained responses.

8.1.2 CPT Pore Pressure Dissipation Results

During the CPT site investigations at the URI Allen Harbor test site, Conetec Inc. performed one pore pressure dissipation test during cone advancement of test SCPT19-01 at a depth of 3.0 m, the raw results of which are found in Appendix A.

Immediately upon commencement of the pore pressure dissipation test at time t_0 , the pore pressure was 1.26 m (4.15 ft). Based on a logarithmic time analysis approach detailed in Figure 107, the fully drained pressure was determined to be 2.04 m.

Figure 107: SCPT19-01 pore pressure dissipation plot with t_{50} determined

Knowing the initial pressure and the fully drained pressure, the pressure at fifty percent dissipation can be calculated as 1.65 m, at an extrapolated time of 1.8 seconds. Inserting t_{50} into Equation 31, the horizontal consolidation rate becomes 8.325×10^{-4} m²/s. When the horizontal consolidation rate is inserted into Equation 30, with a CPT cone penetration rate of 2.0 cm/s and cone diameter of 4.4 cm, the dimensionless penetration rate becomes 1.05, which is determined as partially drained according to the drainage ranges proposed by Kim et al. (2008).

8.1.3 Pile Loading Pore Pressure Dissipation Results

To relate Equation 30 to the pile loading results from this study, the open-ended pile diameter is approximated to a close-ended pile through the use of Equation 32, with an open-ended outside diameter of 0.114 m and a wall thickness of 0.006 m. Thus, the equivalent close-ended pile diameter becomes 0.054 m.

For each pile load test, the derivative of the corrected displacement-time data for the cyclic portions of the tests were taken. The maximum derivative, also known as the maximum displacement velocity, was determined for each test to see if undrained displacement conditions may have occurred. From the maximum displacement velocity, multiplied by the equivalent close-ended pile diameter of 0.054 m and divided by the determined horizontal drainage rate of 8.325×10^{-4} m²/s, the dimensionless penetration rate (V) for the tests was determined. Table 14: details the results of this analysis for each cyclic test:

Table 14: Cyclic load test maximum dimensionless penetration rate results

Maximum Cyclic Dimensionless Penetration Rate for Allen Harbor Pile Load Tests				
Pile P2 8/23/2019	Pile P2 8/29/2019	Pile P4 8/30/2019	Pile P5 11/8/2019	Pile P6 11/18/2019
0.00159	0.04432	0.01707	0.01656	0.01053

As is evident in Table 14: Cyclic load test maximum dimensionless penetration rate results, all of the dimensional penetration rates are less than 0.05 for at a maximum, meaning that all of the pile load tests proceeded in a fully drained loading manner and should be analyzed in a “beta” type framework.

8.2 Displacement Rate Analysis

8.2.1 Displacement Rate Analysis Methodology

As noted in Chapter 2, one way of analyzing cyclic load performance on pile capacity is through determining the displacement rate of each monotonic or cyclic load step the piles experienced, so as to evaluate a threshold load where the pile becomes unstable.

Jardine et al. presents a compiled plot of displacement rate with load for a pile load test in Plancoet, France (data published by Puech et al., 1982) detailed in Figure 108:

Figure 108: Plot of pile displacement rate results for the Plancoet test pile (Jardine et al. 2012)

Cyclic displacements rates in Figure 108 by Puech et al. were determined by taking the maximum pile head displacement with respect to time after the displacement rate began to stabilize. The minimum pile head displacement was also considered, however was subject to initial instability in the first ten cycles, as detailed in Figure 109:

Figure 109: Maximum and minimum displacements (z_{max} and z_{min}), with number of cycles, for the Plancoet pile (Puech and Garnier, 2017)

Under a certain load level, the displacement rate remains relatively stable with higher loads producing a slightly quicker displacement rate. After a certain threshold load has been reached, the displacement rate increases at a much larger slope, ultimately becoming unstable and approaching failure. This general analysis can be implemented for the Allen Harbor monotonic and cyclic load tests, with adaptations.

During the cyclic tests, it was noted that under more stable lower cyclic load ratios, displacement initially accumulated in the first several cycles in an unstable manner as also found by Puech et al., before more slowly accumulating afterwards, and ultimately stabilizing. This performance is detailed in Figure 110 for cyclic test data from pile P2:

Figure 110: Stable cyclic load displacement-time performance for pile P2 under a low cyclic load ratio

In Figure 110, the cyclic displacement performance under a lower load ratio stabilizes before 300 seconds, and enters a more stable linear displacement rate afterwards. This 300 second accumulated displacement interval at the commencement of a load step was reproducible for all piles presented with lower cyclic and monotonic load ratios. Thus, the slope of the displacement and time in a region two cycles before and after the 300 second interval is defined as the “tangent slope”, and is representative of the stable accumulated displacement for the entire load step.

Under higher load steps, where failure is presented, displacement accumulates much more quickly and is defined as being unstable. Figure 111 presents an unstable displacement-time performance, as noted at the end of the second cyclic test on pile P2.

Figure 111: Unstable cyclic load displacement-time performance for pile P2 under a high cyclic load ratio (approaching 1.0)

In the case of Figure 111, upon the commencement of cyclic loading, displacements begin to rapidly accumulate and do not stabilize. The tangent slope after the initial 300 second unstable interval is significantly larger in magnitude than the stable performance presented in Figure 110. Thus, the tangent slope of the displacement rate can be analyzed for all pile load steps to determine where the pile approaches failure.

The Jardine et al. plot in Figure 108 uses load as the independent variable and is sufficient where only one pile is tested. When several piles are being analyzed, and site variability, aging, and cyclic degradation concerns are introduced, the loads tested are dependent on these factors and will present discrepancies based on the effects.

Load ratio provides another means to present the independent variable for analysis, and removes aging discrepancies and site variability as the developed loads are in respect to each individual pile's capacity at the same age. With this said, load ratio is based on the ultimate pile capacity, (Q_u). The ultimate pile capacity is known for pure monotonic tests, and the ultimate post-cyclic monotonic capacity is known for cyclic

tests. The ultimate post-cyclic capacity is subject to the cyclic degradation of the pile and is built into the load ratio displacement rate analysis.

8.2.2 Displacement Rate Analysis Results

For each pile monotonic and cyclic load test, all recorded load steps that were longer than 300 seconds in duration were subjected to tangent displacement rate calculations. Plots were then produced of the displacement rate with respect to the load ratio of that step, with the divisor of the load ratio being the ultimate capacity for the monotonic tests, and for the cyclic tests, the ultimate post-cyclic monotonic capacity.

Presented in Figure 112 is the tangent displacement rates for the one-week load tests:

Figure 112: Tangent displacement rates of one-week test piles

Analyzing Figure 112, a clear change in tangent displacement rate slope is apparent for the pile P2 cyclic test, where cyclic failure occurred. This slope change is not

apparent for the monotonic tests due since the immediate failure load step did not last for 300 seconds. The break in displacement rate slope occurs after a load ratio of 0.85.

Figure 113 displays the tangent displacement rate data for the ten-week load tests:

Figure 113: Tangent displacement rates of ten-week tested piles

The tangent displacement rates are stable for the monotonic and cyclic load steps for the ten week tests up until a load ratio of approximately 0.9 and displacement rate of 1.8 mm/h is reached. The failure point for the pile P5 cyclic test, where cyclic degradation occurred, was not considered as some of the immediate cycles at the end of the 300 second interval were relatively unstable in terms of cyclic displacement.

Some negative tangent slopes were determined for the two figures, from a combination of analysis window and cyclic load application deviations (where the load would be allowed to minimally overshoot the control and cause further displacements, with the recovery causing negative displacements).

As a summary, for the tangent displacement rate plots of all of the pile load tests, stable behavior is determined up to load ratios of 0.85 and displacement rates of 1.8 mm/h.

In comparing the results of the Allen Harbor test piles with those from the Plancoet pile in Figure 108, the displacement rate where the Plancoet pile becomes unstable is approximately 0.1 mm/h. This rate is significantly lower than the encountered at the Allen Harbor site, and may be due to differences in site soil conditions and pile type. The ultimate capacity of the Plancoet pile is unknown, and thus comparisons of the load ratio cannot be made.

The data in Figure 113 can also be compared with a general cyclic stability diagram offered by Tsuha et al. in Figure 114 (2012):

Figure 114: A typical cyclic stability diagram, presenting stable, meta-stable, and unstable zones (Tsuha et al., 2012).

For the Allen Harbor tests, the stable $Q_{\max}/Q_T$ value extends to a value of 0.85. Q_{cyclic} is defined as one-half of the difference of the maximum and minimum cyclic load levels. The minimum cyclic load level for all of the Allen Harbor tests is approximately zero, leading Q_{cyclic}/Q_T to be 0.425 for the piles. Q_{mean} is defined as one-half of the sum

of the maximum and minimum cyclic load ratios, leading Q_{mean}/Q_T to be approximately 0.425, as well. Based on the cyclic stability diagram presented by Tsuha et al., (2012) in Figure 114, a cyclic stability diagram can be made for the Allen Harbor monotonic and cyclic load steps, as shown in Figure 115 for all load ratios, based on load ratios in Table 15 and Appendix E, which are shown using accurate average values of the maximum and minimum cyclic loads for each step:

Table 15: Load ratios for all cyclic load steps of all cyclic load tests performed at the Allen Harbor site with clear load steps

Cyclic Stability Diagram Cyclic Load Ratios								
Cyclic Load Step	Pile P2 8/29/2019		Pile P4 8/30/2019		Pile P5 11/8/2019		Pile P6 11/18/2019	
	Qcvc/Qt	Qmean/Qt	Qcvc/Qt	Qmean/Qt	Qcvc/Qt	Qmean/Qt	Qcvc/Qt	Qmean/Qt
1	0.218	0.272	0.081	0.092	0.370	0.408	0.108	0.117
2	0.282	0.338	0.081	0.092	0.504	0.523	0.331	0.352
3	0.327	0.393	0.087	0.087	0.383	0.408	0.343	0.367
4	0.398	0.466	0.104	0.113	0.385	0.430	0.340	0.365
5	0.436	0.508	0.113	0.122			0.338	0.367
6			0.134	0.162			0.338	0.359
7			0.164	0.175			0.336	0.358
8			0.182	0.182			0.321	0.348
9			0.195	0.217				

Allen Harbor Cyclic Stability Diagram (after Tsuha et al., 2012)

Figure 115: Allen Harbor cyclic stability diagram (after Tsuha et al., 2012)

Based on the results presented in Figure 115, the second cyclic test for pile P2 and cyclic test for P5 clearly present data into the unstable zone, helping indicate cyclic failure was reached at those points. These piles also have load steps above the stable displacement rate points from Figure 112 and Figure 113. The pile P4 cyclic test occurred in the stable region, and as noted during the tests, minimal cyclic displacements occurred. The pile P6 data presents load ratios towards the upper limit of the stable zone, and cyclic failure was not noted for these piles. It was noted that meta-stable conditions were noted significantly noted during the tests, as once the failure cyclic load ratio of 0.85 was reached, displacement and instability occurred rapidly.

It should be noted that the boundaries in Figure 115, as taken from Figure 114, are pile and soil specific, and many adaptations have been made with slightly different envelopes of stability. Thus, the Allen Harbor load ratios performing in a stable manner into an unstable zone is dependent on the individual soil-pile interaction.

8.3 UWA-05 Pile Capacity Corrections

8.3.1 UWA-05 Pile Capacity Correction Methodology

Ultimate post-cyclic monotonic capacity is heavily influenced on cyclic degradation. A way to evaluate the presence of cyclic degradation is through the calibration of the capacity estimation methods against known pure monotonic capacities. This provides a means of estimating expected pure monotonic capacities for the cyclically tested piles.

The UWA-05 pile capacity estimation method was implemented in Chapter 4 to provide an estimation of the magnitude of loads expected at the test site. Overall, it was determined that the method over-predicted the one-week capacity by an order of

magnitude. That being said, the mechanics of the UWA-05 estimation method are well-developed, and a correction of the method for actual monotonic capacities determined will allow estimation of the monotonic capacities of the cyclic test piles. Typically, monotonic capacity methods such as the UWA-05 method are intended to predict the short term (i.e. 7-day) capacity of a pile, and will be applied as such to evaluate the pile P4 cyclic test.

The UWA-05 method outlines the total skin friction (tensile resistance) a pile can withstand by the following equation (Lehane et al. 2007):

$$TSF = \left(\frac{f}{f_c}\right) (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan(\delta_{cv}) \quad (33)$$

Where the $\left(\frac{f}{f_c}\right)$ = empirical constant that relates tensile resistance to compressive (0.75 in the case of tensile loading), σ'_{rc} = radial effective stress after pile installation and equalization, $\Delta\sigma'_{rd}$ = change in radial effective stress due to soil dilation during application of pile loads, and δ_{cv} = constant volume (critical state) interface friction angle of the soil-pile system.

Of the stress terms presented in the total skin friction equation, $\Delta\sigma'_{rd}$, the stress due to constrained dilation of the shear band, is oftentimes over-represented in small diameter piles. This is due to the overall shear band volume being larger relative to the embedded surface area of the pile. In the case of many cyclically loaded piles, the soil in the shear band usually contracts, causing a loss of strength instead of a gain (Puech and Garnier, 2017). Thus, the assumed gain in strength due to constrained dilation is a large component of the initial estimated strength of a pile (in the case of the Allen

Harbor soil, it is consistently upwards of 30% of the strength). If the constrained dilation term is present, large overestimations of capacity can occur.

Furthermore, if piles are being cyclically loaded, contraction (densification) of the shear band that does occur will cause a post-cyclic monotonic dilation and provide an increase in capacity. For the short term cyclic tests, a “smooth” pile allows the interface to mobilize more easily against the shear band, will prevent most increases in capacity due to dilation. It then becomes reasonable to ignore the constrained dilation term.

To account for the lack of dilation under cyclic loading for the test piles, the constrained dilation term is removed from the UWA-05 equation, leaving the following skin friction equation:

$$TSF = \left(\frac{f}{f_c}\right) (\sigma'_{rc}) \tan(\delta_{cv}) \quad (34)$$

Where the strength after equalization, σ'_{rc} , is defined by the following equation (Lehane et al. 2007):

$$\sigma'_{rc} = a q_c (A_{rs}^*)^{0.3} \max \left[\left(\frac{h}{D} \right), 2 \right]^c \quad (35)$$

Where a = the reduction in magnitude of the CPT cone tip resistance near the pile tip during driving (assumed to be 0.03) (Randolph and Gourvenec, 2011), q_c = the cone tip resistance from CPT results, A_{rs}^* = is the effective area ratio of the pile, which accounts for soil plugging and pile geometry, $\frac{h}{D}$ = the height of the pile in the soil mass above the pile tip, relative to the pile diameter, and c = empirical constant for friction fatigue during installation (assumed to be -0.5) (Randolph and Gourvenec, 2011). In the case of the Allen Harbor pile capacity estimation, all of the above terms were known before driving, except for the effective area ratio, A_{rs}^* and the friction fatigue constant, c .

The effective area ratio relates the pile inside (D_i) and outside diameters (D_o) and incremental filling ratio to become a strength factor. The incremental filling ratio (IFR) is the change in soil plug length per increment of driven pile length (Jeong et al., 2015). Encompassing these terms, the effective area ratio takes the following form (Lehane et al, 2007):

$$A_{rs}^* = 1 - IFR \left(\frac{D_i^2}{D_o^2} \right) \quad (36)$$

In the case of the pile installation data, the soil plug development was not measured for each hammer blow, and was only measured during the connection of additional casing sections (every 1.5 m). Thus, the incremental filling ratio was calculated over the driven length of each pile section, with the value over the final installation section being applied to the full pile length in the UWA-05 correction. Table 16 presents the incremental filling ratio for each pile during the installation of the final section of each.

Table 16: Incremental Filling Ratio for each pile section during driving of the final casing section

Incremental Filling Ratio Data During Installation of Last Casing Section for Each Pile			
Pile Number	Final Pile Section Driven Length (m)	Final Section Change in Plug Length (m)	Final Section IFR
P1	1.219	0.749	0.615
P2	1.524	1.048	0.688
P3	1.524	1.270	0.833
P4	1.524	1.340	0.879
P5	1.524	1.232	0.808
P6	1.524	1.308	0.858

These incremental filling ratios over the final installation section were implemented for the UWA-05 capacity correction for each pile.

The friction fatigue correction term, c , is not as well-defined for each pile. As discussed in Chapter 2, all driven piles incur a degree of friction fatigue during driving. In the case of the piles driven at Allen Harbor, a smaller sized hammer imparted 300 to 400 blows to install the piles, and thus, large amounts friction fatigue occurred due to

the large number of driving cycles. This degree of friction fatigue will be described later in the chapter.

Since the UWA-05 method assumes the friction fatigue term is -0.5, empirically, it can be adjusted for each of the piles as a correction on the estimated monotonic capacity.

8.3.2 UWA-05 Pile Capacity Correction Results

In terms of the correcting the UWA-05 monotonic pile capacity method to represent values at the Allen Harbor test site, the known ultimate monotonic capacities at the one-week interval needs to be corrected against. The eight-day ultimate capacity of pile P1 is 5.84 kN, the seven-day ultimate capacity of pile P2 is 7.01 kN.

As CPT tests were performed in between the pile locations at the test site, two sets of CPT test results can be corrected against the known ultimate capacities. For pile P1, the closest CPT tests were SCPT19-04 and SCPT19-03 and for pile P2, the closest tests were SCPT19-03 and SCPT19-02. Table 17 presents the friction fatigue corrections based on the pure monotonic capacities:

Table 17: UWA-5 friction fatigue corrections based in final section incremental filling ratio and ultimate monotonic capacity results

UWA-05 Friction Fatigue Term Correction Based on Monotonic Capacities						
Pile No.	Mono. Ult. Capacity (kN)	CPT Test Left	Corrected "c" Term	CPT Test Right	Corrected "c" Term	IFR
P1	5.84	SCPT 19-04	-1.238	SCPT 19-03	-1.2639	0.615
P2	7.01	SCPT 19-03	-1.165	SCPT19-02	-1.333	0.688

Based on the corrected friction fatigue terms for piles P1 and P2 one-week monotonic capacity estimate bounds can be made for the other test piles at the site by inserting the minimum and maximum friction fatigue factor into the modified UWA-05 equation, as well as inserting each respective pile's final installation section incremental filling ratio.

For the one-week tests, the minimum friction fatigue factor is -1.333, and the maximum is 1.165, as found in Table 17. The seven-day monotonic capacity estimates based on the corrected UWA-05 calculations are presented in Table 18 for test piles P3, P4, P5, and P6:

Table 18: Seven-day estimated monotonic capacities of test piles P3, P4, P5, and P6

7-day Monotonic Capacity Estimates from Corrected UWA-05 Method (kN)							
IFR	Pile No.	CPT Test Left	-1.333	-1.165	CPT Test Right	-1.333	-1.165
0.833	P3	SCPT19-02	6.42	8.95	CPT19-05	7.26	9.84
0.879	P4	SCPT19-02	6.24	8.65	CPT19-05	7.02	9.51
0.808	P5	SCPT19-02	6.53	9.11	CPT19-05	7.39	10.01
0.858	P6	SCPT19-02	6.34	8.79	CPT19-05	7.13	9.66
			Minimum	Maximum			

Analyzing the values in Table 18, the expected seven day capacity of test pile P4, for which a seven-day test was performed, would range from 6.24 kN to 9.51 kN. The post-cyclic monotonic capacity of 23.03 kN, which is 242% of the upper bound from Table 18, and indicates cyclic loading of the pile presented some positive gain in capacity that cannot be contributed to site variability or roughness concerns.

8.4 Aging Analysis

8.4.1 Aging Analysis Results

Immediately after installation, piles encounter low capacities due to friction fatigue disrupting and essentially failing the soil surrounding the pile. Piles are then known to increase in capacity over time, as long as service loads are not presented, otherwise known as being a “virgin” pile.

As the load tests at the URI Allen Harbor site occurred at one-week and ten-week intervals, with the piles being in the virgin condition before testing, a plot of the ultimate capacity with time can be produced. With this plot, changes in capacity outside of site

variability can be determined. Figure 116 presents a plot of ultimate monotonic capacity with time after installation based on data from Table 13 and UWA-05 corrections:

Figure 116: Plot of ultimate monotonic pile capacities with respect to time from installation for all Allen Harbor pile load tests

Analyzing the trends presenting in Figure 116, the virgin pure monotonic tests for piles P1, P2, and, P3 create a clear linear distribution with respect to logarithmic time after installation. The slope of the aging line taken in respect to ultimate capacity of pile P3 at 71 days of aging, and the average ultimate monotonic capacity of pile P1 and P2, is 21.2 kN in one log cycle. Piles P1 and P2, differ by 20.0 percent in capacity. This discrepancy, which contradicts the trend in Figure 116, is within the bounds of site variability determined using the UWA-05 capacity estimation results in Chapter 4 of 20.7 percent.

The mechanism for monotonic capacity increases due to aging is shown in Figure 117. As a pile is installed, the soil being displaced by the pile is compressed outwards,

increasing radial stress along line A-O, with dissipation of stress as the soil further away from the pile is less compressed. The soil in the interface shear band immediately surrounding the pile is then known to contract and collapse, in a process known as “cavity collapse”. This mechanism causes line B-O, where the interface zone collapse lowers stress against the pile face by unloading the “soil spring” radially passed the shear band. With time, the high stresses imposed by the “soil spring” transfer back to the interface shear band and the shear band and more distant radial soil spring begin to equalize to a higher strength at line C-O (White and Bolton, 2004).

Figure 117: Plot for radial monotonic capacity increase (σ_r) with time (known as aging)(White and Bolton, 2004).

Analyzing the pile P4 cyclic test, the post-cyclic monotonic capacity at eight-days was 23.03 kN, and the corrected UWA-05 estimated monotonic capacity at seven-days ranges from 6.24 to 9.51 kN. The post-cyclic monotonic capacity is 242% of the upper UWA-05 estimated bounds, indicating that the lower cyclic load ratio cycles provided capacity increases over and above expected pure monotonic loading capacity, and closer in line with the long term monotonic capacity affected by aging and setup.

This type of cyclic increase in capacity was noted by Bowman and Soga in laboratory creep tests on model piles. It was noted that more aggressive disturbances (higher load ratios and quicker loading) of test piles displayed greater capacity increases

(2005). It was also determined that low-level cyclic loading (as is the case with Allen Harbor test pile P4) increased capacity through the creation of a more dilatant shear band response (2005).

This creation of a more dilatant (denser) shear band is shown through a general interface shear band void ratio plot with respect to the interface normal stress produced by DeJong et al. after Particle Image Velocimetry (PIV) analysis of interface CNS tests, as detailed in Figure 118 (2006):

Figure 118: Interface shear band void ratio plot comparing cyclic and monotonic shear band volume changes with normal stress (DeJong et al., 2006)

The void ratio performance when a shear band soil element when subjected to a cyclic load from an initial state of (σ_{n0}, e_0) decreases towards e_{min} , the densest arrangement of soil particles possible. This is in contrast to volume changes due to pure monotonic loading, where the void ratio and normal stress increases along the stiffness of the surrounding radial soil (shown as one divided by the shear band thickness times the radial soil stiffness) to the interface critical state line (the loosest soil-interface arrangement possible).

As for the pile P2 cyclic tests in the pre-failed condition, the post-cyclic monotonic capacity of the eight-day cyclic test is 83 percent of the seven-day monotonic capacity, suggesting the monotonic test weakened the shear band surrounding the pile by dilating the soil to a “looser” configuration, which the 300 cycles of low level cyclic loading subsequently densifying the soil could not make up for.

The 14-day cyclic test, in which the pile did not experience any service loads for six days, was determined to have an ultimate monotonic capacity of 6.94 kN (99 percent of the eight-day capacity). It is to be noted that this test failed cyclically, and can be said that aging or cyclic action produced this strength recovery, with a cyclic displacement of one percent of the pile diameter (serviceability failure) being 6.70 kN, or 96 percent of the eight-day monotonic capacity.

Reasoning for the slight increase in capacity for the second pre-failed cyclic test due to cyclic action may lie in that after the first pre-failed cyclic test did not provide significant shear band densification, the second pre-failed cyclic test did provide that recovery through 1250 cycles of high load level cycling, producing a more dilative response.

With the pile P5 cyclic test, which occurred at seventy-seven days of aging, the pile was noted to fail cyclically, with an uncontrolled peak cyclic load of 33.33, and a sustained load of approximately 30.0 kN. The post-cyclic ultimate capacity, being 29.2 kN in magnitude, and in-line with the Allen Harbor Monotonic aging line, suggests the failing high magnitude cyclic loading did not benefit, and even degraded post-cyclic capacity. A pre-cyclic monotonic portion was performed before the cyclic portion, with a peak capacity of 23.4 kN produced before reaching a displacement rate that predicted

failure was approaching. This load is 80.1% of the post-cyclic capacity determined, and 70.3% of the peak cyclic load produced.

In terms of shear band volume change behavior, the lower high level cyclic loading before the uncontrolled portion would induce a densification of the shear band, upon which the failing uncontrolled cyclic loads would cause rapid dilation with some recovery contraction as the pile became mobilized and then becoming more dilatant again under the lower controlled cyclic loading. Upon termination of cyclic loading, the already mobilized shear band would not have the same density or strength as presented by the naturally aged shear band, causing a reduction in capacity relative to the peak cyclic load.

For the eighty-seven day pile P6 cyclic test, a portion of 0.22 cyclic load ratio testing occurred, followed by a monotonic load test to a load of 25.61 kN, and was halted before failure. This load is below the Allen Harbor Monotonic Aging Characterization Line. Then, cyclic testing began of 0.82 cyclic load ratio loading for 7120 cycles. The post-cyclic ultimate monotonic capacity after was 36.45 kN, 142% of the pre-cyclic monotonic load. It should also be noted that pre-secondary cyclic ultimate load of 25.61 kN is 70.3% of the ultimate post-secondary cyclic pile capacity, exactly the same difference between the peak cyclic load and pre-cyclic monotonic load of pile P5.

These findings for the pile P5 and P6 tests suggest the lower-level cyclic loading of pile P5 strengthened the pile, up until a failure cyclic load of 33.3 kN was reached. The same can be said for the 0.82 cyclic load ratio loading of pile P6, however the noted strength increase occurred during a post-cyclic monotonic portion. The pile P6 strength

increase is most likely due to the cyclic loading causing a densification of the shear band around the pile, with a “rough” interface causing a dilative post-cyclic monotonic response with a clear peak in capacity as the dense particles rearrange around a rougher interface. This clear post-cyclic dilative response is detailed in Figure 106.

8.5 Friction Fatigue

8.5.1 Discussion of Friction Fatigue Effects and Methodology of Analysis

In Section 8.3, it was noted that the cyclic load tests on the piles at the one-week and ten-week time frames after installation noted capacity increases over expected pure monotonic results. The gain in capacity is, in part, due to friction fatigue allowing further recovery of capacity from aging responses.

During pile driving, the soil near the tip of the pile encounters significant increases in stress, with the magnitude increasing as the pile type advances. Once the pile passes a given soil element in depth, and after the soil either flows around the pile or enters the compacted soil plug, the stresses the pile is subjected to begin to decrease.

In denser soils or when lighter driving hammers are required to install the pile, the number of driving cycles required to reach a certain depth of embedment increases. With additional driving cycles, the number of large stress cycles the soil encounters increases by the same amount, reducing the overall strength of the soil post-installation.

This performance has been noted by Anusic et al., which in a study of monotonic pile load tests from different regions of the world noted a significant decrease in average shear strength of a pile with increasing numbers of driving cycles required per unit area of total pile embedment (2018). The average shear strength reported in the study was obtained as the quotient of the ultimate tensile capacity and the surface area of the pile.

The shear strength is considered to be the “average” as local stress responses of the piles may not be the same throughout the embedded shaft.

The monotonic test piles analyzed in the study were from well-characterized sites such as the University of Western Australia’s Shenton Park site (Schneider, 2007), (Lim and Lehane, 2014), (Lehane, 2008), and (Anusic et al, 2018); the Imperial College London’s Dunkirk site (Jardine et al. 2006), (Chow et al., 1997), and (Carroll et al., 2017); and the University College of Dublin’s Blessington, site (Gavin and Lehane, 2003), (Gavin et al., 2013), (Gavin and O’Kelly, 2007), and (Carroll et al., 2017).

The data compiled and published by Anusic et al., as found in Appendix E, provide a database of piles to compare the University of Rhode Island Allen Harbor pile performance with in determining the effect of installation induced friction fatigue on capacity. In the case of the Anusic et al. study, sixty-four pile load tests were analyzed, with large variances in diameter and embedment depth presented. In order to compare all piles of differing sizes together, a driving resistance term was developed correlating blow counts with total embedded shaft area (referred to as “blows per square meter”, or N_{bA}).

Similarly, due to the large differences in determined shear (tensile) resistances encountered by each analyzed pile, a strength value normalizing the determined ultimate average monotonic pile strength to the estimated UWA-05 monotonic pile capacity becomes a useful means of comparing a pile’s strength (degraded or otherwise) against expected capacities (Anusic et al., 2018).

In terms of Allen Harbor pile performance, as noted in Chapter 6, a small 1.33kN hammer with a drop height of 0.76 m (1.01 kN-m of hammer energy per cycle) was

implemented to drive the piles. Blow counts to full embedment ranged from 304 to 389, inducing a maximum energy of 393 kN-m over the 4.27 m embedment. It is thought that this energy spread over the number of driving cycles aided in the reduced one-week capacities determined.

8.5.2 Allen Harbor Friction Fatigue

It is understood that high levels of friction fatigue during pile installation will cause short term capacity losses (before subsequent aging and setup occurs). In the monotonic load tests compiled by Anusic et al., the different results presented were separated into two performance categories, “short-term”, and “long-term” (2018). The short term tests were on piles up to one-week after installation (1 to 7 days). The long-term tests were performed much later ranging from seventy to nearly two thousand days after installation. As these time bounds fit with the bounds from the Allen Harbor tests with one and ten-week tests, short-term and long-term analyses can be made.

Short-term normalized shear strength values with respect to total installation resistance are plotted in Figure 119 for the Allen Harbor pure monotonic and post-cyclic monotonic test piles, as well as compiled Shenton Park, Dunkirk, and Blessington monotonic test piles (published by Anusic et al. 2018).

Figure 119: Short-term comparison of normalized pile shear strength with estimated UWA-05 strength (non-Allen Harbor data published by Anusic et al., 2018)

As is evident in Figure 119, as more energy is required to drive the test piles to embedment, the lower the average shear strength is with respect to the estimated amount. Pile P2 encountered approximately 200 blows per square meter of embedment while piles P1 and P4 were approximately 25 blows per square meter. Under pure monotonic and pre-failed post-cyclic monotonic testing, piles P1 and P2 achieved an ultimate capacity of approximately an order of magnitude lower than the UWA-05 estimate. With the compiled data from the other test sites, this level of short-term friction fatigue was only seen in a Shenton Park pile subjected to four thousand blows per square meter of shaft area. This finding implies that the Allen Harbor test piles encountered much larger amounts of friction fatigue than at the other test sites.

As for the pile P4 cyclic test, the cyclic loading induced an ultimate capacity of 40 percent of the UWA-05 estimates, which is significantly higher than the other test piles at the site, and more in-line with the performance found at the Shenton Park and

Blessington test sites. This affirms the findings from Section 8.3 that cyclic action on pile P4 strengthened the pile over and above capacity solely due to aging.

Adding in the effects of aging with the addition of long-term test results from all four test sites, a clear increase in normalized shear strength is found across all test sites, with slopes of different rates of setup determined. Figure 120 presents this comparison, with long-term tests being considered at a timing of seventy days or more after installation and short term at a timing of 14-days or earlier.

Figure 120: Comparison of short term (unfilled points) and long term (filled points) normalized pile capacities from the Allen Harbor test site and other test sites (non-Allen Harbor data published by Anusic et al., 2018)

Between all of the four sets of data presented between test sites, the average rate of setup at the Allen Harbor test site is similar to that at the Blessington test site, with the Shenton Park and Dunkirk sites presenting higher rates of setup, respective to a logarithmic scale. As noted with Figure 119, the Allen Harbor piles' resistances were

significantly lower than predicted, and maintaining this offset even with setup taken into account.

As all of the pile load test results have been on tests with varying size and embedment length, some discrepancies can occur between differing slenderness ratios presented. In order to reduce this discrepancy, a comparison can be made between piles of similar embedment depth and diameter. One set of the test piles in the full set compiled by Anusic et al., Shenton Park's pile SP4, was of similar dimensions as the Allen Harbor piles (4 m of embedment and 114 mm in diameter) (2018).

Due to the similarities in pile dimensions, the driving resistance of blows per square meter can be replaced by blows per meter of embedment depth. Two pile tests were performed for the Shenton Park SP4 piles, one at 3 days after installation and one at 374 days of installation. Non-normalized average shear strengths for the piles were 20 and 60 kPa for the short term and long term tests, with strength increasing by a factor of three over the timeframe (Schneider, 2007), (Anusic et al., 2018). The comparison of average normalized shear strength to driving resistance is detailed in Figure 121:

Figure 121: Short term (unfilled points) and long term (filled points) normalized pile capacity results for the Allen Harbor piles compared to Shenton Park pile SP4 (Shenton Park data from Schneider, 2007 and compiled by Anusic et al., 2018)

In comparison to the rate of strength increase of the SP4 pile, the Allen Harbor piles, which were presented with a much lower normalized short term shear strength and lower driving resistance, increased in capacity by 4.7 times from the one-week to the ten week tests (comparing the pure monotonic tests), which is 17% of the time the Shenton Park site increased in capacity by a factor of 3. It is suggested by this finding that greater amounts of friction fatigue (and ensuing reductions in estimated monotonic strength) leads to a quicker rate of strengthening.

CHAPTER 9- CONCLUSIONS

As offshore wind structures become more common in the waters off of the United States, understanding of the effects environmental loads place on the structures, and the response of the supporting foundational elements are critical to determine. In the case of wind jacket structures, where turbines are supported by a series of piles, the tensile loads caused by environmental moments on the supporting piles creates the critical design element, as it is the only resistive element.

Marine environmental loads are often presented in cyclic form, and cyclic action on piles is often known to cause a gradual loss in pile strength, known as cyclic degradation. Furthermore, during pile installation, friction fatigue is known to cause a weakened initial capacity, with continued setup and aging as time progresses and the radial stiffness provides more strength.

The goals of the investigation were to detail the implementation of model pile load tests in a coastal environment with granular soils, as well as evaluate the effects of cyclic degradation, aging, and friction fatigue in the performance of piles subjected to 10^4 numbers of 0.125 Hz frequency cycles. This amount was chosen as hurricanes present in the offshore areas of the United States are known to generate cycles on a similar order.

Conclusions of this study include:

- Piles initially driven at the test site were “smooth” relative to the surrounding soil and were measured to be “rough” at 39 days after installation in the intertidal zone.

- Short term initial capacity was surprisingly an order of magnitude lower than CPT estimated capacity, with relatively higher numbers of driving cycles presumed and a smooth pile interface combining to generate more friction fatigue and loss of capacity.
- Short term (one-week after installation) low-level tensile cycling provided a significant increase in ultimate monotonic capacity after cycling. For the Allen Harbor test site, this was noted to be 3.3 times the one-week pure monotonic capacity, and outside the realm of site variability.
- Rates of long-term (ten weeks after installation) cyclic capacity increase are noticeably lower than short term cycling, with the maximum capacity increase being the same whether the ultimate load is cyclic or post-cyclic monotonic in nature. Aging and cyclic pile stabilization are expected to work at strengthening the piles, albeit to different degrees.
- Maximum cyclic load ratios of less than 0.85 were determined to generate pile head displacement rates of less than 1.8 mm/h and provide stable responses, whereas piles above a ratio of 0.85 were generally unstable, with tangent displacement rates rapidly advancing to higher displacement rates and ultimately, failure.
- Pure monotonic and cyclic capacity increases linearly with logarithmic setup time (where no service loads are present of the piles).
- Compared to the results of short term and long term monotonic tests at the University of Western Australia (UWA) on piles with a similar slenderness ratio as the Allen Harbor piles, the Allen Harbor piles encountered less driving cycles

until embedment, however the Allen Harbor piles also encountered lower short term normalized capacities and hence a higher rate of friction fatigue from driving.

- The rate of capacity increase from the short term to the long term tests was both larger and quicker than the similarly sized UWA pile increases, indicating lower friction fatigue induced capacities lead to quicker and larger increases in pile setup.

FUTURE TESTING

In terms of future studies in regards to this investigation, the determination of the in-situ shear wave velocity and small strain shear modulus results at the test site are relatively uncertain, and work can be performed characterizing those soils to a larger degree to help characterize shear band stiffness performance.

In terms of suggestions for similar testing types in the future, the development and implementation of a data acquisition system capable of retaining large amounts of data over the 24 hours of the cyclic tests proved difficult. In these tests, readings could only be taken in one second intervals, relatively coarse for the eight second period cycles produced. The data acquisition system recorded data with a Microsoft Excel add-in, and any changes in selected cells during data reading would cause the system to crash and lose results. Implementation of a transducer output voltage amplifier and development of a more user-friendly interface with live load-displacement readouts would have aided field testing significantly.

Another component to take into account is the harsh environmental conditions all of the mechanical testing components were subjected to. Most of the equipment in the

load-testing system was designed to operate under well-regulated temperature and moisture conditions in a clean environment (especially the pressure regulator). Implementation of pneumatic and hydraulic filters would have removed regulator valve clogging, and consideration of wide temperature changes in equipment design would have prevented equipment performance degradation.

One final note of advice for future testing is that almost all of the results determined in this study were not expected during field testing, and even with plenty of consideration and prediction of load test outcomes, capacities and displacements are often unexpected, and small testing errors often create compounding load-displacement results. These complications make the post-test data analysis more interesting, and led to some of the findings of this study which may not have been possible under perfect testing conditions (for instance, the loss of pressure regulation in the Pile P5 cyclic load test caused a cyclic failure to occur, with some degradation of capacity noted in the post-cyclic monotonic response). Summed up in a sentence: at first glance, a test which failed in meeting objectives still produces valuable data.

APPENDICES

APPENDIX A – SITE INVESTIGATION RESULTS

Presented in the following pages are a set of site plans showing geotechnical investigation locations. Boring logs produced during the advancement of geotechnical borings appear next, with Standard Penetration Tests (SPT) performed to evaluate relative density at the test site. The boring logs were compiled by Dr. Aaron Bradshaw.

Following the boring logs are a list of soil samples and volumes obtained from the split-spoon sampler during the SPT test, as well as USCS classification and moisture content results from index tests.

The results of the Seismic and Standard Cone Penetration Test (SCPT and CPT, respectively) performance, as performed and compiled by ConeTec Inc. are also presented.

APPENDIX A.1 – SITE PLANS

Figure 122: Overall site plan of the Allen Harbor Educational Facility with inset of geotechnical investigation bounds

Figure 123: Site plan of site investigations performed as a part of this study, along with locations of test piles

APPENDIX A.2 – INTERPRETED SOIL PROFILE

Figure 124: Section A-A' Interpreted soil profile from SPT and CPT results

APPENDIX A.3 – BORING LOGS

		PROJECT		REPORT OF BORING NO.		URI-1			
		BSEE Cyclic Axial Pile		SHEET		1/1			
				FILE NO.					
				CHKD BY					
BORING CO. Hoffman Environmental		BORING LOCATION Allen Harbor Lat: 41.6224 Long: -71.4104							
FOREMAN Kyle Hoffman				GROUND SURFACE ELEV. _____ DATUM _____					
ENGINEER Aaron Bradshaw (URI)				DATE START 12/4/2019 DATE END 12/4/2019					
DRILLING METHOD: Hollow Stem Auger				GROUNDWATER READINGS					
CASING SIZE:				DATE	TIME	WATER	CASING	STABILIZATION TIME	
SAMPLER: Standard Split Spoon				12/4/2019	10:30 am	3.5 ft			
HAMMER: Automatic									
DEPTH	FIELD	SAMPLE				SAMPLE DESCRIPTION		R	STRATUM
TESTING		NO	PEN./REC (in)	DEPTH (ft)	BLOWS/ft	BURMISTER CLASSIFICATION		K	DESCRIPTION
5		S1	24/11	0-2	1/18", 3	Very loose, brown, medium SAND, trace Gravel, moist			
10		S2	24/14	5-7	6,5,4,5	Loose, dark gray, medium SAND, trace gravel, wet			
15		S3	24/14	10-12	1,5,11,16	Medium dense, dark gray, fine SAND, wet			
20		S4	24/15	15-17	2,7,11,12	Medium dense, dark gray, fine to medium SAND, little Silt, wet			
25						Bottom of Boring at 20 ft			
30									
GRANULAR SOILS		COHESIVE SOILS		REMARKS:					
BLOWS/FT DENSITY		BLOWS/FT DENSITY		Driller indicated very silty soil at the bottom of the borehole. Samples were not taken at this depth.					
0-4	VERY LOOSE	<2	VERY SOFT						
4-10	LOOSE	2-4	SOFT						
10-30	MEDIUM DENSE	4-8	M. STIFF						
30-50	DENSE	8-15	STIFF						
>50	VERY DENSE	15-30	V. STIFF						
		>30	HARD						
NOTES:									
1) STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.									
2) WATER LEVEL READINGS HAVE BEEN MADE AT TIMES AND UNDER CONDITIONS STATED, FLUCTUATIONS OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE									
BORING NO.									

Figure 125: Boring log for boring URI-1 (Bradshaw, 2018)

		PROJECT		REPORT OF BORING NO. URI-2				
		BSEE Cyclic Axial Pile		SHEET 1/1				
				FILE NO.				
				CHKD BY				
BORING CO. Hoffman Environmental		BORING LOCATION Allen Harbor Lat: 41.6224 Long: -71.4105						
FOREMAN Kyle Hoffman		GROUND SURFACE ELEV. DATUM						
ENGINEER Aaron Bradshaw (URI)		DATE START 12/4/2019		DATE END 12/4/2019				
DRILLING METHOD: Hollow Stem Auger		GROUNDWATER READINGS						
CASING SIZE:		DATE	TIME	WATER	CASING			
SAMPLER: Standard Split Spoon		12/4/2019	11:15 am	4.0 ft				
HAMMER: Automatic								
DEPTH	FIELD TESTING	SAMPLE				SAMPLE DESCRIPTION BURMISTER CLASSIFICATION	R K	STRATUM DESCRIPTION
		NO	PEN./REC (in)	DEPTH (ft)	BLOWS/6"			
5		S1	24/5	0-2	2,3,3,3	Loose, brown, medium SAND, little Gravel, moist		
10		S2	24/13	5-7	4,5,4,6	Loose, dark gray, coarse SAND, some Silt, little Gravel, wet		
15		S3	24/13	10-12	1,3,5,7	Loose, dark gray, coarse SAND, some Silt, little gravel, wet		
20		S4	24/12	15-17	2,3,5,7	Loose, dark gray, medium SAND, some Silt, trace Gravel, wet		
25						Bottom of Boring at 15 ft		
30								
GRANULAR SOILS BLOWS/FT DENSITY		COHESIVE SOILS BLOWS/FT DENSITY		REMARKS:				
0-4	VERY LOOSE	<2	VERY SOFT					
4-10	LOOSE	2-4	SOFT					
10-30	MEDIUM DENSE	4-8	M. STIFF					
30-50	DENSE	8-15	STIFF					
>50	VERY DENSE	15-30	V. STIFF					
		>30		HARD				
NOTES:		1) STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL. 2) WATER LEVEL READINGS HAVE BEEN MADE AT TIMES AND UNDER CONDITIONS STATED, FLUCTUATIONS OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE						
		BORING NO.						

Figure 126: Boring log for boring URI-2 (Bradshaw, 2018)

		PROJECT		REPORT OF BORING NO. URI-3				
		BSEE Cyclic Axial Pile		SHEET 1/1				
				FILE NO.				
				CHKD BY				
BORING CO. Hoffman Environmental		BORING LOCATION Allen Harbor Lat. 41.6225 Long. -71.4104						
FOREMAN Kyle Hoffman		GROUND SURFACE ELEV. DATUM						
ENGINEER Aaron Bradshaw (URI)		DATE START 12/4/2019		DATE END 12/4/2019				
DRILLING METHOD: Hollow Stem Auger		GROUNDWATER READINGS						
CASING SIZE:		DATE	TIME	WATER	CASING			
SAMPLER: Standard Split Spoon		12/4/2019	12:15 am	2.0 ft				
HAMMER: Automatic								
DEPTH	FIELD TESTING	SAMPLE				SAMPLE DESCRIPTION BURMISTER CLASSIFICATION	R K	STRATUM DESCRIPTION
		NO	PEN./REC (in)	DEPTH (ft)	BLOWS/6"			
5		S1	24/16	0-2	2,3,3,4	Loose, brown, coarse SAND, some Gravel, trace Silt, moist		
10		S2	24/13	5-7	1,1,2,2	Very loose, dark gray, coarse SAND, some Silt, trace shell fragments, wet		
15		S3	24/17	10-12	4,4,5,6	Loose, dark gray, coarse SAND, some Gravel, trace Silt, wet		
20		S4	24/17	15-17	9,13,13,12	Medium dense, dark gray, medium SAND, some Gravel, trace Silt, wet Bottom of Boring at 15 ft		
25								
30								
GRANULAR SOILS BLOWS/FT DENSITY		COHESIVE SOILS BLOWS/FT DENSITY		REMARKS:				
0-4	VERY LOOSE	<2	VERY SOFT					
4-10	LOOSE	2-4	SOFT					
10-30	MEDIUM DENSE	4-8	M. STIFF					
30-50	DENSE	8-15	STIFF					
>50	VERY DENSE	15-30	V. STIFF					
		>30	HARD					
NOTES:		1) STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL. 2) WATER LEVEL READINGS HAVE BEEN MADE AT TIMES AND UNDER CONDITIONS STATED. FLUCTUATIONS OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE						
		BORING NO.						

Figure 127: Boring log for URI-3 (Bradshaw, 2018)

		PROJECT BSEE Cyclic Axial Pile		REPORT OF BORING NO. URI-4		
				SHEET 1/1		
				FILE NO.		
				CHKD BY		
BORING CO. Hoffman Environmental		BORING LOCATION Allen Harbor Lat: 41.6224 Long: -71.4103				
FOREMAN Kyle Hoffman		GROUND SURFACE ELEV. DATUM				
ENGINEER Aaron Bradshaw (URI)		DATE START 12/4/2019		DATE END 12/4/2019		
DRILLING METHOD: Hollow Stem Auger		GROUNDWATER READINGS				
CASING SIZE:		DATE	TIME	WATER	CASING	
SAMPLER: Standard Split Spoon		12/4/2019	1:05 pm	2.4 ft		
HAMMER: Automatic						
DEPTH	FIELD TESTING	SAMPLE			SAMPLE DESCRIPTION	
		NO	PEN./REC (in)	DEPTH (ft)	BURMISTER CLASSIFICATION	
5		S1	24/11	0-2	1,2,2,4	Very loose to loose, brown, medium SAND, trace shells, moist
10		S2	24/20	5-7	2,4,4,5	Loose, dark gray, medium to coarse SAND, trace Gravel, wet Grading to Silt and Sand at bottom of sample
15		S3	24/17	10-12	7,11,11,13	Medium dense, dark gray, SILT and SAND, wet
20		S4	24/18	15-17	14,17,22,18	Dense, dark gray, SILT and SAND, wet Bottom of Boring at 15 ft
25						
30						
GRANULAR SOILS BLOWS/FT DENSITY		COHESIVE SOILS BLOWS/FT DENSITY		REMARKS:		
0-4	VERY LOOSE	<2	VERY SOFT	Cobble/boulder encountered at 8 feet but pushed out of way with auger.		
4-10	LOOSE	2-4	SOFT			
10-30	MEDIUM DENSE	4-8	M. STIFF			
30-50	DENSE	8-15	STIFF			
>50	VERY DENSE	15-30	V. STIFF			
		>30	HARD			
NOTES: 1) STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL. 2) WATER LEVEL READINGS HAVE BEEN MADE AT TIMES AND UNDER CONDITIONS STATED, FLUCTUATIONS OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE						
					BORING NO.	

Figure 128: Boring log for boring URI-4 (Bradshaw, 2018)

Figure 129: Boring log for boring URI-5 (Bradshaw, 2018)

		PROJECT		REPORT OF BORING NO.				
		BSEE Cyclic Axial Pile		URI-6				
				SHEET 1/1				
				FILE NO.				
				CHKD BY				
BORING CO.	Hoffman Environmental	BORING LOCATION		Allen Harbor Lat: 41.6224 Long: -71.4104				
FOREMAN	Kyle Hoffman	GROUND SURFACE ELEV.		DATUM				
ENGINEER	Aaron Bradshaw (URI)	DATE START		DATE END				
DRILLING METHOD: Hollow Stem Auger		GROUNDWATER READINGS						
CASING SIZE:		DATE	TIME	WATER	CASING			
SAMPLER: Standard Split Spoon								
HAMMER: Automatic								
DEPTH	FIELD TESTING	SAMPLE				SAMPLE DESCRIPTION BURMISTER CLASSIFICATION	R K	STRATUM DESCRIPTION
		NO	PEN/ /REC (in)	DEPTH (ft)	BLOWS/ft			
5		S1	24/16	0-2	1,1,1,2	Very loose, brown, medium SAND, little gravel, moist		
						Boulder encountered at 3 feet. Drilling terminated.		
10								
15								
20								
25								
30								
GRANULAR SOILS BLOWS/FT DENSITY		COHESIVE SOILS BLOWS/FT DENSITY		REMARKS:				
0-4	VERY LOOSE	<2	VERY SOFT					
4-10	LOOSE	2-4	SOFT					
10-30	MEDIUM DENSE	4-8	M. STIFF					
30-50	DENSE	8-15	STIFF					
>50	VERY DENSE	15-30	V. STIFF					
		>30	HARD					
NOTES: 1) STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL. 2) WATER LEVEL READINGS HAVE BEEN MADE AT TIMES AND UNDER CONDITIONS STATED, FLUCTUATIONS OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE								
BORING NO.								

APPENDIX A.4 – LIST OF SOIL SAMPLES RETRIEVED FROM BORINGS

Table 19: Soil samples obtained from geotechnical borings at the Allen Harbor test site

SPT Soil Samples Recovered: URI Allen Harbor Site				
Borehole:	Sample:	Depth (meters below surface)	Recovery (m)	Sample Volume (mL):
URI-1	S1	0 to 0.61	0.43	250
	S2	1.52 to 2.13	0.55	250
	S3	3.05 to 3.66	0.55	250
	S4	4.57 to 5.18	0.59	250
URI-2	S1	0 to 0.61	0.20	125
	S2	1.52 to 2.13	0.51	250
	S3	3.05 to 3.66	0.51	200
	S4	4.57 to 5.18	0.47	250
URI-3	S1	0 to 0.61	0.63	270
	S2	1.52 to 2.13	0.51	250
	S3	3.05 to 3.66	0.67	200
	S4	4.57 to 5.18	0.67	250
URI-4	S1	0 to 0.61	0.43	250
	S2	1.52 to 2.13	0.79	500
	S3	3.05 to 3.66	0.67	500
	S4	4.57 to 5.18	0.71	250
URI-5	No Sample, SPT hit boulder	0 to 0.61	0.59	N/A
URI-6	S1, hit boulder at 0.9 m	0 to 0.61	0.63	250

APPENDIX A.5 – SOIL SAMPLE USCS CLASSIFICATION

Table 20: Soil Sample USCS Classification from grain size analysis

Soil Sample	D10 (mm)	D30 (mm)	D60 (mm)	Cu	Cc	USCS Group Symbol	USCS Group Name
URI-1 S1	0.19	0.33	0.64	3.37	0.90	SP	Poorly Graded Sand
URI-1 S2	0.17	0.3	0.6	3.53	0.88	SP	Poorly Graded Sand
URI-1 S3	0.076	0.12	0.21	2.76	0.90	SM	Silty Sand
URI-1 S4	0.076	0.17	0.55	7.24	0.69	SM	Silty Sand
URI-2 S1	0.17	0.24	0.36	2.12	0.94	SP	Poorly Graded Sand
URI-2 S2	0.076	0.21	0.92	12.11	0.63	SM	Silty Sand
URI-2 S3	0.1	0.28	0.74	7.40	1.06	SW-SM	Well-Graded Sand with Silt
URI-2 S4	0.086	0.21	0.37	4.30	1.39	SP-SM	Poorly Graded Sand with Silt

APPENDIX A.6 – GRADATION PLOT

Figure 131: Soil gradation curve from grain size analysis on soil samples from boreholes URI-1 and URI-2

APPENDIX A.7 – MOISTURE CONTENT WITH DEPTH

Figure 132: Moisture content with depth approximately six months after retrieval and storage in sealed glass jars. Samples were investigated from boreholes URI-1 and URI-2

APPENDIX A.8 – CPT RESULTS FROM CONETEC INC.

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

Job No: 19-53048
Client: University of Rhode Island
Project: Bruce Boyer Road, North Kingstown, RI
Start Date: 02-May-2019
End Date: 02-May-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (ft)	Final Depth (ft)	Shear Wave Velocity Tests	Northing ² (m)	Easting (m)	Refer to Notation Number
SCPT19-01	19-53048_SP01	5/2/2019	524:T375F10U500	3.0	30.02	9	4610667	299197	
SCPT19-02	19-53048_SP02	5/2/2019	524:T375F10U500	3.0	28.54	11	4610663	299199	3
SCPT19-03	19-53048_SP03	5/2/2019	524:T375F10U500	3.0	27.23	10	4610659	299201	3
SCPT19-04	19-53048_SP04	5/2/2019	524:T375F10U500	3.0	30.02	10	4610655	299202	3
CPT19-05	19-53048_CP05	5/2/2019	524:T375F10U500	3.0	30.02		4610664	299193	3
Totals	5 soundings				145.83	40			

1. Assumed phreatic surface depths were determined from the pore pressure data unless otherwise noted. Hydrostatic data were used for calculated parameters.
2. Coordinates are WGS 84 / UTM Zone 19 and were collected using a MR-350 GlobalSat GPS Receiver.
3. Assumed phreatic surface estimated from the dynamic pore pressure response.

Max Depth: 9.150 m / 30.02 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-53048_SP01.COR

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19N N: 4610667 E: 299197

Hydrostatic Line ● Ueq ● Assumed Ueq ▲ PPD, Ueq achieved ▲ PPD, Ueq not achieved
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Figure 133: CPT results for test SCPT19-01 (inadvertently performed in the location of a geotechnical boring) (ConeTec, 2019)

Figure 134: CPT results for test SCPT19-02 (ConeTec Inc., 2019)

Figure 135: CPT results for test SCPT19-03 (ConeTec Inc., 2019)

Max Depth: 9.150 m / 30.02 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-53048_SP04.COR

SBT: Robertson, 2009 and 2010

Coords: UTM Zone 19N N: 4610655 E: 299202

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Figure 136: CPT results for test SCPT19-04 (ConeTec Inc., 2019)

Max Depth: 9.150 m / 30.02 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-53048_CP05.COR

SBT: Robertson, 2009 and 2010

Coords: UTM Zone 19N N: 4610664 E: 299193

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Figure 137: CPT results for test CPT19-05 (ConeTec Inc., 2019)

Seismic Cone Penetration Wave Traces

Figure 138: SCPT19-01 shear wave trace (inadvertently performed in a geotechnical boring) (ConeTec Inc., 2019)

Figure 139: SCPT19-02 shear wave trace (ConeTec Inc., 2019)

Figure 140: SCPT19-03 shear wave trace (ConeTec Inc., 2019)

Figure 141: SCPT19-04 shear wave trace (ConeTec Inc., 2019)

Seismic Cone Penetration Test Tabular Results (Vs)

Job No: 19-53048
Client: University of Rhode Island
Project: Bruce Boyer Road, North Kingstown, RI
Sounding ID: SCPT19-01
Date: 02-May-2019

Seismic Source: Beam
Source Offset (ft): 6.56
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
3.28	2.62	7.07			
4.92	4.27	7.82	0.76	2.65	286
6.56	5.91	8.83	1.00	3.69	272
8.20	7.55	10.00	1.17	3.80	309
9.84	9.19	11.29	1.29	1.22	1059
13.12	12.47	14.09			
14.76	14.11	15.56	1.47	1.04	1419
16.40	15.75	17.06	1.50	1.81	830
18.14	17.49	18.68	1.62	1.34	1209
19.69	19.03	20.13	1.45	2.08	698

Job No: 19-53048
Client: University of Rhode Island
Project: Bruce Boyer Road, North Kingstown, RI
Sounding ID: SCPT19-02
Date: 02-May-2019

Seismic Source: Beam
Source Offset (ft): 6.56
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
1.64	0.98	6.63			
3.28	2.62	7.07	0.43	1.32	327
4.92	4.27	7.82	0.76	2.15	353
6.56	5.91	8.83	1.00	0.94	1070
8.20	7.55	10.00	1.17	0.72	1636
9.84	9.19	11.29	1.29	1.55	833
11.48	10.83	12.66			
13.12	12.47	14.09	1.43	1.76	810
14.76	14.11	15.56	1.47	2.20	667
16.40	15.75	17.06	1.50	2.03	738
18.04	17.39	18.58	1.52	1.92	795
19.69	19.03	20.13	1.54	2.10	735

Job No: 19-53048
Client: University of Rhode Island
Project: Bruce Boyer Road, North Kingstown, RI
Sounding ID: SCPT19-03
Date: 02-May-2019

Seismic Source: Beam
Source Offset (ft): 6.56
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
3.28	2.62	7.07			
4.92	4.27	7.82	0.76	0.72	1060
6.56	5.91	8.83	1.00	0.94	1070
8.20	7.55	10.00	1.17	3.64	322
9.84	9.19	11.29	1.29	1.43	900
13.12	12.47	14.09	2.80	1.93	1452
14.76	14.11	15.56	1.47	2.82	522
16.40	15.75	17.06	1.50	2.19	684
18.04	17.39	18.58	1.52	1.65	925
19.69	19.03	20.13	1.54	1.89	817

Job No: 19-53048
Client: University of Rhode Island
Project: Bruce Boyer Road, North Kingstown, RI
Sounding ID: SCPT19-04
Date: 02-May-2019

Seismic Source: Beam
Source Offset (ft): 6.56
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
3.28	2.62	7.07			
4.92	4.27	7.82	0.76	1.92	395
6.56	5.91	8.83	1.00	3.03	331
8.20	7.55	10.00	1.17	0.77	1520
9.84	9.19	11.29	1.29	1.31	983
11.48	10.83	12.66	1.37	1.51	906
13.12	12.47	14.09	1.43	2.36	606
14.76	14.11	15.56	1.47	2.75	535
16.40	15.75	17.06			
18.04	17.39	18.58	1.52	1.15	1324
19.69	19.03	20.13	1.54	1.70	906

Soil Behavior Type (SBT) Scatter Plots

Figure 142: SBT plot for SCPT19-01 (inadvertently performed in a geotechnical boring) (ConeTec Inc., 2019)

Figure 143: SBT plot for SCPT19-02 (ConeTec Inc., 2019)

Figure 144: SBT plot for SCPT19-03 (ConeTec Inc., 2019)

Figure 145: SBT plot for SCPT19-04 (ConeTec Inc., 2019)

Figure 146: SBT plot for CPT19-05 (ConeTec Inc., 2019)

Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

Job No: 19-53048
Client: University of Rhode Island
Project: Bruce Boyer Road, North Kingstown, RI
Start Date: 02-May-2019
End Date: 02-May-2019

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U _{eq} (ft)	Calculated Phreatic Surface (ft)	Estimated Phreatic Surface (ft)	t ₅₀ ^a (s)	Assumed Rigidity Index (I _r)	c _v ^b (cm ² /min)
SCPT19-01	19-53048_SP01	15	120	9.84	6.83	3.01				
Totals	1 dissipations		2.0 min							

a. Time is relative to where umax occurred

b. Housley and Teh, 1991

University of Rhode Island

Job No: 19-53048

Date: 05/02/2019 09:58

Site: Bruce Boyer Road, North Kingstown, RI

Sounding: SCPT19-01

Cone: 524:T375F10U500 Area=15 cm²

Trace Summary: Filename: 19-53048_SP01.PPD U Min: 4.1 ft WT: 0.919 m / 3.014 ft
Depth: 3.000 m / 9.842 ft U Max: 6.9 ft Ueq: 6.8 ft
Duration: 120.0 s

Figure 147: Pore water dissipation data from performed during SCPT19-01 (ConeTec Inc., 2019)

APPENDIX B – UWA-05 PILE CALCULATION METHODOLOGY

Contained in the following pages is the UWA-05 Method calculation explanation as implemented in Chapter 4. Objectives of this section are to discuss the methodology of these calculations when applied towards soil capacity calculations from Cone Penetration Testing (CPT) Results.

METHOD AS APPLIED

The UWA-05 Method, as applied in this investigation, is based off publications describing the method: (Lehane et al., 2007) and (Randolph and Gourvenec, 2011). This method has been adapted for use on short open-ended piles as follows.

There are four main calculated components to the UWA-05 method which, in order, allow the calculation of axial tensile resistance of open-ended piles:

- Radial Effective Stress after Pile Installation, σ'_{rc}
- Change in Radial Effective Stress due to soil dilation during loading, $\Delta\sigma'_{rd}$
- Skin Friction
- Tensile Resistance

These components are derived from the following series of equations:

RADIAL EFFECTIVE STRESS AFTER PILE INSTALLATION

Radial Effective Stress after pile installation is found by the following equation:

$$\sigma'_{rc} = a * q_c * (A_{rs}^*)^{0.3} * \max\left[\left(\frac{h}{D}\right), 2\right]^{-0.5} \quad (37)$$

Where q_c = the CPT uncorrected cone tip resistance, a = the stress drop from q_c around the pile tip (Randolph and Gourvenec, 2011), A_{rs}^* = the effective area ratio described below, and $\frac{h}{D}$ is a relation of the pile's slenderness ratio, otherwise defined as a ratio of height above the pile tip to outside pile diameter. The value, a , is taken as

0.03 for open ended piles in compression (to be corrected for tension further in the description). Effective area ratio (A_{rs}^*) is defined by the following equation:

$$A_{rs}^* = 1 - IFR \left(\frac{D_i^2}{D_o^2} \right) \quad (38)$$

Where IFR = the incremental filling ratio, D_i = the inner pile diameter, and D_o = the outer pile diameter. Values of A_{rs}^* closer to zero mean the pile acts unplugged (coring), whereas values closer to unity mean the pile plugs with soil upon the commencement of driving, creating a type of “close-ended” condition. The incremental filling ratio is a correction factor for plugging conditions during installation as approximated in the equation below, when not known from driving.

$$IFR_{mean} \approx \min \left[1, \left(\frac{D_i}{1.5} \text{ meters} \right)^{0.2} \right] \quad (39)$$

For the incremental filling ratio, values closer to zero would mean the pile is plugged, while closer to unity would mean the pile is fully coring.

CHANGE IN RADIAL STRESS DUE TO DILATION, ($\Delta\sigma'_{rd}$)

Soil dilation is known to cause changes in stress acting on the pile due to the confined expansion of the “shear band” against the pile wall during loading. In terms of capacity, dilation would cause a net increase of soil tensile resistance, while contraction would cause a net decrease of tensile resistance. In the UWA-05 Method, stress changes due to shear band volume changes are expressed by the following equation:

$$\Delta\sigma'_{rd} = 4G \left(\frac{\Delta r}{D_o} \right) \quad (40)$$

Where $\Delta\sigma'_{rd}$ = the change in radial effective stress due to shear band expansion or contraction, G = the small-strain shear modulus (otherwise known as G_{max}) and Δr is the value for dilation (+) or contraction (-) of the shear band under loading. The UWA-05

Method assumes a Δr value of 0.02 mm when tests of in situ soils have not been performed (as applied in this study). The small-strain shear modulus is determined by the following equation:

$$\frac{G}{q_c} = 185 * (q_{c1N})^{-0.75} \quad (41)$$

In this case, the shear modulus is set as a ratio to the uncorrected cone tip resistance, q_c . Here, q_{c1N} is the normalized cone tip resistance, as defined by the following equation:

$$q_{c1N} = \frac{\frac{q_c}{p_a}}{\left(\frac{\sigma'_{v0}}{p_a}\right)^{0.5}} \quad (42)$$

Where p_a = atmospheric pressure (2116 psf or 101.33 kPa), and σ'_{v0} is the initial vertical effective stress of the soil.

SKIN FRICTION

Skin friction (TSF), or the frictional resistance of the adjacent soil on the pile walls, is expressed in the following equation combining the strength terms indicated earlier:

$$TSF = \left(\frac{f}{f_c}\right) * (\sigma'_{rc} + \Delta\sigma'_{rd}) * \tan(\delta_{cv}) \quad (43)$$

In this case, TSF is the total skin friction acting per unit area of pile wall, $\frac{f}{f_c}$ = an axial load direction correction factor, with compressive loads being unity, and tensile loads being 0.75, and δ_{cv} = the constant volume friction angle of the soil. This value can be determined from the following plot if not known (Lehane et al., 2007).

Figure 148: Estimation of the interface friction angle in a pile-soil system (Lehane et al., 2007)

TENSILE RESISTANCE

The skin friction along the unit length and perimeter of the pile is summed (and for use in this project, averaged) using the following equation:

$$Q_s = TSF_{avg.} * pile\ perimeter * (L_{embedding}) \quad (44)$$

Here Q_s is the ultimate tensile resistance and $L_{embedding}$ is the length of pile embedment in the soil.

APPENDIX C – PILE DRIVING LOGS

The following section will detail the driving logs recorded and compiled by the Author during installation (in SI units). Hoffman Environmental Services performed the installation over multiple days.

Table 21: Casing driving record for pile P1

URI Allen Harbor Casing Installation Record Sheet: Casing P1					
Date:	8/12/2019	Time of Finish:	4:50 PM	Hammer Weight:	300 lbs/1.33 kN
Time of Start:	3:52 PM	Do	4.5"/0.1143 m	Notes:	3" Cap connected to Casing While Driving
Driller:	Hoffman Env. Services	Wall Thickness:	0.5"/0.0127m		Plug Measured to Soil Surface
Stroke:	30"/0.76m	Hammer Make/Model:	Safe-T Hammer with Cathead		After 2.13 m of driving, driller switched to 3 wraps from 4
Field Engineer:	TEK, CDPB, ASB				Driving shoe cut off bottom of casing
Depth (m)	Blows	Top of Casing to Outside Soil Surface (m)	Top of Casing to Inside Soil Surface (m)	Plug Depth from Surface (m)	Notes:
0.30	3	0	0	0	Some small plug left in bottom from retry
0.61	7				
0.91	16				
1.22	31				
1.52	26	0	0	0	
1.83	29				Plug Measured to Soil Surface 0-2.13 m 4 wraps on cathead 2.44-4.27 m rope burnt off cathead-used 3 wraps
2.13	29				
2.44	32				
2.74	34				
3.05	35	0	0.48	0.48	
3.35	35				Casing left 0.3 m sticking out of ground-marked in orange
3.66	34				
3.96	33				
4.27	35	0.3	1.26	0.96	
Sum of Blows:	379				

Table 22: Casing driving record for pile P2

URI Allen Harbor Casing Installation Record Sheet: Casing P2					
Date:	8/15/2019	Time of Finish:	4:25 PM	Hammer Weight:	300 lbs/1.33 kN
Time of Start:	3:45 PM	Do	4.5"/0.1143 m	Notes:	3" Cap connected to Casing While Driving
Driller:	Hoffman Env. Services	Wall Thickness:	0.5"/0.0127m		Plug Measured to Soil Surface
Stroke:	30"/0.76m	Hammer Make/Model:	Safe-T Hammer with Cathead		Driving shoe threaded off bottom of casing
Field Engineer:	TEK, ASB, JSF				0.1 m of threads left exposed at bottom
Depth (m)	Blows	Top of Casing to Outside Soil Surface (m)	Top of Casing to Inside Soil Surface (m)	Plug Depth from Surface (m)	Notes:
0.30	2	0	0	0	2 wraps used for entire run
0.61	4				
0.91	4				
1.22	9	Missed Measurement	Missed Measurement	N/A	
1.52	12				
1.83	13				
2.13	20				
2.44	25				
2.74	35	0.30	0.81	0.51	
3.05	34				
3.35	35				
3.66	37				
3.96	39				
4.27	35	0.32	1.30	0.98	Hook closing hinge broke, had to directly tie rope to hammer Casing left 0.32 m sticking out of ground-marked in orange
Sum of Blows:	304				

Table 23: Casing driving record for pile P3

URI Allen Harbor Casing Installation Record Sheet: Casing P3					
Date:	8/15/2019	Time of Finish:	5:15 PM	Hammer Weight:	300 lbs/1.33 kN
Time of Start:	4:35 PM	Do	4.5"/0.1143 m	Notes:	3" Cap connected to Casing While Driving
Driller:	Hoffman Env. Services	Wall Thickness:	0.5"/0.0127m		Plug Measured to Soil Surface
Stroke:	30"/0.76m	Hammer Make/Model:	Safe-T Hammer with Cathead		Driving shoe threaded off bottom of casing
Field Engineer:	TEK, JSF				0.1 m of threads left exposed at bottom
					Hook closing hinge broke, had to directly tie rope to hammer
Depth (m)	Blows	Top of Casing to Outside Soil Surface (m)	Top of Casing to Inside Soil Surface (m)	Plug Depth from Surface (m)	Notes:
0.30	2	0	0	0	2 wraps used for entire run
0.61	3				
0.91	5				
1.22	13	0.23	0.43	0.2	
1.52	13				
1.83	14				
2.13	14				
2.44	22				
2.74	33	0.30	0.88	0.58	
3.05	43				
3.35	54				
3.66	55				
3.96	47				
4.27	52	0.33	1.16	0.83	Casing left 0.33 m sticking out of ground-marked in orange
Sum of Blows:	370				

Table 24: Casing driving record for pile P4

URI Allen Harbor Casing Installation Record Sheet: Casing P4					
Date:	8/23/2019	Time of Finish:	6:15 PM	Hammer Weight:	300 lbs/1.33 kN
Time of Start:	5:34 PM	Do	4.5"/0.1143 m	Notes:	3" Cap connected to Casing While Driving
Driller:	Hoffman Env. Services	Wall Thickness:	0.5"/0.0127m		Plug Measured to Soil Surface
Stroke:	30"/0.76m	Hammer Make/Model:	Diedrich D50 Turbo Safe-T Hammer with Cathead		Driving shoe and threads cut off bottom of casing
Field Engineer:	TEK, AMH		2 rope wraps on cathead		Hook closing hinge broke, had to directly tie rope to hammer
Depth (m)	Blows	Top of Casing to Outside Soil Surface (m)	Top of Casing to Inside Soil Surface (m)	Plug Depth from Surface (m)	Notes:
0.30	4				
0.61	6				
0.91	5				
1.22	13	0.47	0.23	0.24	
1.52	12				
1.83	15				
2.13	14				
2.44	16				
2.74	28	1.10	0.30	0.8	
3.05	44				
3.35	49				
3.66	56				Driller noted hitting rock around 3.66 m
3.96	58				
4.27	69	1.28	0.3	0.98	
Sum of Blows:	389				

Table 25: Casing driving record for pile P5

URI Allen Harbor Casing Installation Record Sheet: Casing P5					
Date:	8/23/2019	Time of Finish:	6:44 PM	Hammer Weight:	300 lbs/1.33 kN
Time of Start:	6:21 PM	Do	4.5"/0.1143 m	Notes:	3" Cap connected to Casing While Driving
Driller:	Hoffman Env. Services	Wall Thickness:	0.5"/0.0127m		Plug Measured to Soil Surface
Stroke:	30"/0.76m	Hammer Make/Model:	Diedrich D50 Turbo Safe-T Hammer with Cathead		Driving shoe and threads cut off bottom of casing
Field Engineer:	TEK, AMH		2 rope wraps on cathead		Hook closing hinge broke, had to directly tie rope to hammer
Depth (m)	Blows	Top of Casing to Outside Soil Surface (m)	Top of Casing to Inside Soil Surface (m)	Plug Depth from Surface (m)	Notes:
0.30	3				
0.61	4				
0.91	9				
1.22	11	0.43	0.20	0.23	
1.52	15				
1.83	14				
2.13	13				
2.44	30				
2.74	34	1.07	0.23	0.84	
3.05	30				
3.35	41				
3.66	63				
3.96	57				
4.27	60	1.37	0.23	1.14	
Sum of Blows:	384				

Table 26: Casing driving record for pile P6

URI Allen Harbor Casing Installation Record Sheet: Casing P6					
Date:	8/23/2019	Time of Finish:	7:18 PM	Hammer Weight:	300 lbs/1.33 kN
Time of Start:	6:54 PM	Do	4.5"/0.1143 m	Notes:	3" Cap connected to Casing While Driving
Driller:	Hoffman Env. Services	Wall Thickness:	0.5"/0.0127m		Plug Measured to Soil Surface
Stroke:	30"/0.76m	Hammer Make/Model:	Diedrich D50 Turbo Safe-T Hammer with Cathead		Driving shoe and threads cut off bottom of casing
Field Engineer:	TEK, AH		2 rope wraps on cathead		Hook closing hinge broke, had to directly tie rope to hammer
Depth (m)	Blows	Top of Casing to Outside Soil Surface (m)	Top of Casing to Inside Soil Surface (m)	Plug Depth from Surface (m)	Notes:
0.30	3				
0.61	6				
0.91	10				
1.22	13	0.5	0.28	0.22	
1.52	14				
1.83	20				
2.13	11				
2.44	11				
2.74	18	1.04	0.30	0.74	
3.05	32				
3.35	59				
3.66	54				
3.96	58				
4.27	53	1.26	0.30	0.96	
Sum of Blows:	362				

APPENDIX D – PILE ROUGHNESS

The following pages will detail the casing roughness drawings and results of a single casing section after installation at the Allen Harbor test site, and removal after 39 days. The roughness testing was performed with a MarSurf XR20 metrology device located at the University of Rhode Island's mechanical engineering metrology laboratory.

Figure 149: View of surface profile tested pile after being installed at the Allen Harbor test site for 39 day. Approximate locations of roughness traces are shown

MarSurf XR 20 V1.30-5	University of Rhode Island Industrial and Systems Engineering Kirk Hall	UNIVERSITY OF Rhode Island 02/07/20
	XR20-Admin New Measurement	

Object: Number: Comment:

Meas. Instrum.: Mahr Data Acquisition Board Drive Unit: PZK Pick-up: MFW-250 [-0.8 %]	Lt: 17.5 mm [N=5] Ls: 8.0 μm VB: $\pm 250 \mu\text{m}$ Vt: 0.50 mm/s Points: 11667
---	---

Profile: R [LC GS: 2.5 mm]	
Ra	2.36 μm
Rq	3.04 μm
Rz	13.80 μm
Rt	17.33 μm

Figure 150: Surface roughness Reading 0 on the section of the casing not embedded

Figure 151: Surface roughness Reading 1 results

MarSurf XR 20 V1.30-5	University of Rhode Island Industrial and Systems Engineering Kirk Hall XR20-Admin New Measurement	02/07/20
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Object: Number: Comment:

Meas. Instrum.: Mahr Data Acquisition Board Drive Unit: PZK Pick-up: MFW-250 [-0.8 %]	Lt: 17.5 mm [N=5] Ls: 8.0 μm VB: $\pm 250 \mu\text{m}$ Vt: 0.50 mm/s Points: 11667
---	---

Profile: R [LC GS: 2.5 mm]	
Ra	1.25 μm
Rq	1.92 μm
Rz	9.89 μm
Rt	16.24 μm

Figure 152: Surface roughness Reading 2 results

MarSurf XR 20 V1.30-5	University of Rhode Island Industrial and Systems Engineering Kirk Hall	UNIVERSITY OF Rhode Island
	XR20-Admin New Measurement	

Object: Number: Comment:

Meas. Instrum.: Mahr Data Acquisition Board Drive Unit: PZK Pick-up: MFW-250 [-0.8 %]	Lt: 17.5 mm [N=5] Ls: 8.0 μm VB: $\pm 250 \mu\text{m}$ Vt: 0.50 mm/s Points: 11667
---	---

Profile: R [LC GS: 2.5 mm]	
Ra	0.76 μm
Rq	1.06 μm
Rz	8.04 μm
Rt	12.45 μm

Figure 153: Surface Roughness Reading 3 results

MarSurf XR 20 V1.30-5	University of Rhode Island Industrial and Systems Engineering Kirk Hall	02/07/20
	XR20-Admin New Measurement	

Object: Number: Comment:

Meas. Instrum.: Mahr Data Acquisition Board Drive Unit: PZK Pick-up: MFW-250 [-0.8 %]	Lt: 17.5 mm [N=5] Ls: 8.0 μm VB: $\pm 250 \mu\text{m}$ Vt: 0.50 mm/s Points: 11667
---	---

Profile: R [LC GS: 2.5 mm]	
Ra	0.92 μm
Rq	1.67 μm
Rz	12.61 μm
Rt	17.47 μm

Figure 154: Surface roughness Reading 4 results

Table 27: Surface roughness Reading 5 results (the Marsurf printout was not retained)

MarSurf XR 20	University of Rhode Island	2/7/2020	
V1.30-5	Industrial and Systems Engineering		
	Kirk Hall		
Meas. Instrum.:	Mahr Data Acquisition Board	Lt:	17.5 mm [N=5]
Drive Unit:	PZK	Ls:	8.0 um
Pick-up:	MFW-250 [-0.8%]	VB:	500 um
		Vt:	0.50 mm/s
		Points	11667
	No Profile- Printout not saved		
Profile: R	[LC GS: 2.5 mm]		
Ra:	10.55 um		
Rq:	13.15 um		
Rz:	53.56 um		
Rt:	62.57 um

MarSurf XR 20 V1.30-5	University of Rhode Island Industrial and Systems Engineering Kirk Hall	UNIVERSITY OF Rhode Island 02/07/20
	XR20-Admin New Measurement	

Object: Number: Comment:

Meas. Instrum.: Mahr Data Acquisition Board Drive Unit: PZK Pick-up: MFW-250 [-0.8 %]	Lt: 17.5 mm [N=5] Ls: 8.0 μm VB: $\pm 250 \mu\text{m}$ Vt: 0.50 mm/s Points: 11667
---	---

Profile: R [LC GS: 2.5 mm]	
Ra	8.97 μm
Rq	11.60 μm
Rz	52.44 μm
Rt	76.34 μm

Figure 155: Surface roughness Reading 6 results

APPENDIX E –CYCLIC LOAD TEST LOAD STEPS

The following figures in Appendix E display the cyclic load steps with respect to time for the creation of cyclic stability diagrams of Figure 115. The ultimate post-cyclic monotonic capacities are shown at the end of the tests.

Figure 156: Pile P2 8/29/2020 cyclic load retest load steps

Figure 157: Pile P4 cyclic load test cyclic load steps

Figure 158: Pile P5 cyclic load test cyclic load steps

Figure 159: Pile P6 cyclic load test cyclic load ratio steps

APPENDIX F – PUBLISHED REFERENCE PILE DATA

Table 28: Compiled reference pile data for friction fatigue analysis with URI Allen Harbor Results provided (after Anusic et al., 2018)

Table of Comparison Test Pile Results (After Anusic et al. 2018)													
Test Site	Pile I.D.	Age (days)	Driving Cycles	NbA (Blows/m2)	τ Avg. (kPa)	τ UWA-05 (kPa)	Install. Method	PLR	Lemb (m)	Do (mm)	End Cond.	τ/τ UWA-05	Reference
UWA-Shenton Park	SP1	4	1642	1470	16	46	D	0.76	4	88.9	O	0.35	(Schneider, 2007)
		376	1642	1470	59	46	D	0.76	4	88.9	O	1.28	
	SP2	5	305	437	16	33	D	0.64	2.5	88.9	O	0.48	
		259	305	437	58	33	D	0.64	2.5	88.9	O	1.76	
	SP3	3	1840	1647	23	45	D	0.76	4	88.9	O	0.51	
		374	1840	1647	77	45	D	0.76	4	88.9	O	1.71	
	SP4	3	1830	1274	20	39	D	0.88	4	114.3	O	0.51	
		374	1830	1274	60	39	D	0.88	4	114.3	O	1.54	
	SP5	3	1486	1330	26	44	D	0.75	4	88.9	O	0.59	
		374	1486	1330	77	44	D	0.75	4	88.9	O	1.75	
	SP6	4	330	619	15	68	D	0.44	4	42.4	O	0.22	
		376	330	619	98	68	D	0.44	4	42.4	O	1.44	
	SP7	5	90	270	17	46	D	0.43	2.5	42.4	O	0.37	
		259	90	270	74	46	D	0.43	2.5	42.4	O	1.61	
	SP8	4	4344	3888	7	55	D	0	4	88.9	C	0.13	
		375	4344	3888	79	55	D	0	4	88.9	C	1.44	
	SP9	5	451	646	7	34	D	0	2.5	88.9	C	0.21	
		259	451	646	59	34	D	0	2.5	88.9	C	1.74	
	SP10	1	15	13	46	45	J	0	3.72	101.5	C	1.02	
	SP11	29	15	13	48	45	J	0	3.75	101.5	C	1.07	
	SP12	72	31	13	48	44	J	0	3.1	101.5	C	1.09	
	SP13	1	31	48	46	46	J	0	3.14	65	C	1.00	
	SP14	27	32	49	43	47	J	0	3.2	65	C	0.91	
	SP15	29	32	49	50	47	J	0	3.2	65	C	1.06	
	SP16	70	31	49	56	53	J	0	3.1	65	C	1.06	
	SP17	1	32	24	37	34	J	0	3.16	134.6	C	1.09	
	SP18	29	32	24	39	34	J	0	3.16	134.6	C	1.15	
	SP19	71	31	24	43	41	J	0	3.1	134.6	C	1.05	
	SP20	1	35	49	52	71	J	0	3.49	65	C	0.73	
	SP21	38	36	49	53	71	J	0	3.57	65	C	0.75	
	SP22	1	27	23	43	50	J	0	2.73	134.6	C	0.86	
	SP23	41	28	24	43	51	J	0	2.77	134.6	C	0.84	
	SP24	14	N/A	1	60	N/A	B	0	4	225	N/A	N/A	(Lehane, 2008)
	SP25	14	N/A	1	57	N/A	B	0	4	340	N/A	N/A	
	SP26	6	374	288	18	26	D	0.555	4	165.1	O	0.69	(Anusic et al., 2018)
	SP27	114	280	216	20	26	D	0.537	4	165.1	O	0.77	
	SP28	N/A	246	190	N/A	26	D	0.575	4	165.1	O	N/A	
	SP29	7	661	364	23	27	D	0.575	4	165.1	O	0.85	
	SP30	119	469	258	26	27	D	0.605	4	165.1	O	0.96	
	SP31	N/A	423	233	N/A	26	D	0.597	4	165.1	O	N/A	
	SP32	7	N/A	N/A	26	26	V	0.5	4	165.1	O	1.00	
	SP33	117	N/A	N/A	22	25	V	0.617	4	165.1	O	0.88	
ICP-Dunkirk	DK1	9	2507	90	52	53	D	0.604	19.31	457	O	0.98	(Jardine et al., 2006)
	DK2	81	2405	89	89	46	D	0.479	18.9	457	O	1.93	
	DK3	235	2268	84	119	50	D	0.466	18.85	457	O	2.38	
	DK4	188	721	64	34	50	D	0.49	11.1	324	O	0.68	(Chow et al., 1997)
		1991	721	64	64	50	D	0.49	11.1	324	O	1.28	
	DK5	176	721	64	40	48	D	0.45	11.1	324	O	0.83	
		288	721	64	46	48	D	0.45	11.1	324	O	0.96	
	DK6	188	3040	136	77	52	D	0.55	22	324	O	1.48	
		1988	3040	136	142	52	D	0.55	22	324	O	2.73	
	DK7	1	220	687	157	93	D	0.22	2	51	O	1.69	(Carroll et al., 2017)
DK8	85	220	687	273	93	D	0.22	2	51	O	2.94		
UCB-Blessington	BL1	1	19	29	85	99	J	0.35	1.9	111	O	0.86	(Gavin and Lehane, 2003)
	BL2	1	480	745	50	65	D	0.85	1.8	114	O	0.77	
	BL3	1	325	575	48	60	D	0.9	1.8	100	O	0.80	
	BL4	1	376	1140	41	60	D	0.93	1.4	75	O	0.68	(Gavin et al., 2013)
	BL5	1	695	93	46	64	D	0.693	7	340	O	0.72	
	BL6	12	739	99	89	66	D	0.693	7	340	O	1.35	
	BL7	30	529	71	51	66	D	0.719	7	340	O	0.77	
	BL8	220	630	84	132	68	D	0.693	7	340	O	1.94	
	BL9	1	54	87	109	124	J	0	2.7	73	C	0.88	(Gavin and O'Kelly, 2007)
	BL10	1	3	5	136	127	J	0	2.49	73	C	1.07	
	BL11	1	15	64	165	84	D	0.36	1.75	60	O	1.96	
	BL12	20	15	64	172	84	D	0.36	1.75	60	O	2.05	(Carroll et al., 2017)
	BL13	80	15	64	154	84	D	0.36	1.75	60	O	1.83	
URI-Allen Harbor	P1	8	379	247	4	31	D	0.777	4.27	114.3	O	0.12	This Study
	P2	7	304	198	5	35	D	0.771	4.27	114.3	O	0.13	
	P2	8	304	198	4	35	D	0.771	4.27	114.3	O	0.11	
	P2	14	304	198	5	35	D	0.771	4.27	114.3	O	0.13	
	P3	71	370	241	18	38	D	0.807	4.27	114.3	O	0.48	
	P4	8	389	254	15	38	D	0.769	4.27	114.3	O	0.40	
	P5	77	384	251	22	38	D	0.734	4.27	114.3	O	0.58	
P6	87	362	236	24	38	D	0.777	4.27	114.3	O	0.63		
Legend:													
D=Driven													
O=Open-Ended													
C=Close-Ended													
J=Jacked													
B=Bored													
CFA=Cont. Flight Auger													
V=Vibrated													
PLR=Plug Length Ratio													
Lemb= Embedment Length													
Do=Outside Diameter													

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