ABSTRACT

SHANAHAN, CASEY DANE. Improving the Resiliency of Coastal Sand Dunes using Microbial Induced Calcite Precipitation. (Under the direction of Dr. Brina Montoya.)

In the field of soil improvement research, microbial induced calcite precipitation (MICP) has been at the forefront. Bio-mediated soil improvement couples sustainability and cost effectiveness with drastic improvements to the mechanical properties of sandy soils. While MICP in unsaturated conditions is in the beginning stages of study, the objective herein is to evaluate the suitability of MICP for reducing erosion of coastal sand dunes. In this thesis, the issue was approached from three physical scales: elemental-scale, model-scale, and system scale.

At the elemental-scale, an optimization of the saltwater-based treatment solution was conducted with respect to unconfined strength characteristics. Used in parallel with the chemical-mechanical optimization, a pore fluid batch test was performed with varying nutrient concentrations to ascertain the influence of the chemical components of the treatment solution on the MICP reaction time. After the optimized chemical concentration was determined, the strength-cementation relationship was studied by creating more heavily cemented sands and subjecting them to unconfined compressive strength testing. The optimization was concluded by a micro-scale investigation via scanning electron microscopy to observe changes in cementation pattern, distribution, and chemical composition.

At the model-scale, the assessment of the critical shear stress as a function of cementation level was assessed experimentally. Based on impinging jet scour tests, four soil
columns at varying levels of cementation were scour tested to evaluate the increase in critical shear stress as a function of cementation. Critical shear stress is commonly used as a parameter in numerical modeling of dune erosion, bridging the gap between cementation level and expected erosion.

At the system scale, MICP cemented sands were subjected to wave actions using a large-scale wave tank. Ranging from un-cemented to heavily cemented, five wave tank tests were completed and erosion patterns were measured using 2-D laser morphology. Based on the morphology before and after the application of wave loading, the erosion was determined and trends were assessed for expected erosion at a particular cementation level.
Improving the Resiliency of Coastal Sand Dunes using Microbial Induced Calcite Precipitation

by

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A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Master of Science

Civil Engineering

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APPROVED BY:

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Committee Chair

_______________________________
Dr. M. A. Gabr, P.E.
DEDICATION

I dedicate my thesis to my Family for their never-ending love and support throughout the last two years. Without having such a support system, I couldn’t possibly be where I am today. In particular, a special feeling of gratitude to my parents, Michael and Kelly Shanahan, whose daily encouragement allowed me to fight through the hard times and who taught me the importance of consistent hard work. My siblings, Michael and Lindsay Shanahan, for their support and for always having an open ear for my complaints. My grandmother, Carol Shanahan, for always being by my side and for believing in me, even when I had my doubts.

In addition, I also dedicate this thesis to my wonderful girlfriend, Kara Shervanick for accompanying me in the lab for countless hours and for inspiring me to push outside of my comfort zone. I give special thanks to my best friend, Shawn Anderson for all of his help in and out of the lab throughout the last couple of years.

Finally, I dedicate this thesis to Dr. Brina Montoya. I’m incredibly grateful to have had the opportunity to work beneath you for the last four years. It was you that sparked and grew my interest in Geotechnical Engineering through your contagious passion. You’ve been the best mentor I could ask for and I’m honored to have worked with you as long as I have.
BIOGRAPHY

Casey Shanahan was born in Wichita, Kansas, to Michael and Kelly Shanahan in 1992. He is one of three children, with an older brother, Michael, and a younger sister, Lindsay. Shortly after being born, his family moved to Iowa, where they would stay until 1999. In 1999, his family moved to Jacksonville, North Carolina, where they still live today. Casey attended Jacksonville High School, where he made the decision to pursue a B.S. in Civil Engineering at North Carolina State University. This decision stemmed initially from his interest in his father’s line of work, bridge construction. Although his original intent was to study Structural Engineering, after doing a couple of internships, taking soil mechanics courses, and beginning to do Geotechnical Engineering research, Casey developed a passion for Geotechnical Engineering and his MICP research. The logical decision was to continue on and complete his MSCE at North Carolina State University under the guidance of Dr. Brina Montoya. Upon graduation, Casey intends to begin his career with Schnabel Engineering as a Geostructural Engineer in Rockville, MD.
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1 OVERVIEW OF THESIS

1.1 Introduction

Microbial induced calcite is the most prominent bio-mediated soil improvement technique in current study. In a new sustainability focused era, the desire to enhance natural processes rather than providing man-made solutions to subsurface issues, has become the motivation for many research projects to date. Previous contamination issues associated with the improvement of subsurface soil have led to the development of a cost-effective soil improvement method with minimum negative environmental repercussions (Karol, 2003). The appeal of soil improvement methods arrive from economic solutions to improving the soil rather than designing more tolerance structures. In-situ improvement using microbial induced calcite precipitation (MICP) has the potential to improve the strength and stiffness of the soil, enhancing the engineering performance.

Bio-mediated soil improvement can be readily defined as a biochemical reaction occurring within a soil matrix that is controlled by a microbial process to induce mechanical or hydrological changes to the soil (Martinez, 2012). The MICP reaction occurs through microbial ureolysis, which provides urea to the ureolytic bacteria to convert ammonia and carbon dioxide gas. The conversion into ammonia and carbon dioxide gas causes a diffusion of gas through the bacterial cell membrane and consequently, speciation in the pore fluid. This causes a spike in local pH and provides carbonate bonds for calcium carbonate precipitation in
the presence of calcium. Typically in laboratory settings, *Sporosarcina pasteurii* is used as the microbe that harness the urease enzyme.

Unlike most of the industry standard soil improvement methods, the proposed application of this thesis is to improve the resiliency of coastal sand dunes. Coastal sand dunes are geological features in need of surficial improvement. Typically on most geotechnical engineering projects, surficial soil improvement can be completed relatively cost-effectively through compaction. However, compaction is not feasible on these natural structures, sparking interest in other improvement methods. Through surficial treatment, microbial induced calcite precipitation has the potential to cement the sandy soil together at the particle contacts, leading to a drastic reduction in eroded volume when subjected to wave action.

### 1.2 Thesis Organization

The thesis consists of three primary chapters, each of which spark investigations into different testing scales. Chapter 2 consists of an elemental-level study, during which an optimization of the saltwater based treatment solution with respect to effluent chemistry and mechanical strength is performed and compared with freshwater. Chapter 3 is at the model scale, as the critical shear stress of as a function of cementation level is examined. Chapter 4 contains system-scale wave tank testing and evaluates the reduction in erosion on the basis of cementation.

Chapter 2 contains a series of unconfined compressive strengths at varying chemical concentrations to assess the sensitivity of the treatment components on the corresponding unconfined compressive strengths. To further investigate the pore-space bio-geo-chemistry,
pore fluid batch tests were conducted and the reaction times with the inoculated bacterium are used during the chemical optimization. SEM images and x-ray diffraction with elemental analysis is used to observe changes in cementation patterns at the sand grain scale and to provide insight into the chemical changes caused by the addition of saltwater into the treatment solution.

Chapter 3 is used as bridge between the elemental-scale chemical optimization and the system-level testing. This chapter focuses on evaluating the increase critical shear stress as a function of cementation level, with an intention of connecting the critical shear stress to the expected erosion reduction. The susceptibility to scour is likely related to the erodibility of the soil, which led to the inclusion of the critical shear stress into this thesis.

Chapter 4 consists of system-scale wave tank tests used to evaluate changes in erosion due to wave loading for soils cemented to various levels. Five wave tank tests are performed and the eroded volumes are assessed using 2-D laser morphology. Erosive behavior with respect to cementation is observed experimentally.
2 TREATMENT OPTIMIZATION FOR UNSATURATED COASTAL SANDS

2.1 Background

Extensive research has been conducted to further the understanding of the MICP induced mechanical properties obtained through saturated treatment; limited attention has been given to unsaturated treatment regimes. Through eliminating the necessity of transporting freshwater to the treatment sites, the cost of in-situ MICP treatment of coastal sand dunes can be dramatically reduced. In conjunction with reduced chemical costs (through treatment optimization) and pumping seawater, MICP becomes a much more economically feasible option for improving the surficial mechanical properties of sand deposits. Unlike most of the MICP research to date, the sand dune application is predominately unsaturated, which eliminates the need for injection wells and allows for surficial application of the treatment media.

To determine whether or not MICP treatments are a viable option for improving sand dune resiliency, a series of unconfined compressive strength tests were conducted with the intention of optimizing the chemical treatment recipe to achieve maximum strength while minimizing effluent ammonium, which is a known groundwater contaminant. The EPA regulated maximum ammonia concentration for health advisory is 30,000 ppb \( \left( \frac{\text{mg}}{L} \right) \) (Macler, 2007). It becomes evident by the common urea dosages (0.333M, which translates to 20,200
ppb per treatment) that the effluent concentrations of ammonia over time are substantially larger than the EPA regulated health advisory levels, which magnifies the need for optimization of the treatment recipe. Generally, in the case of MICP treatments, extraction wells would be drilled and the effluent would be pumped out to avoid ground water contamination. The ammonium generated can be absorbed by the sand grain particles, flow through the soil, or be assimilated into indigenous bacteria or plants (Whiffin, 2004).

2.1.1 Literature Review on Unsaturated Treatment

Unsaturated soil environments, such as partially saturated dune sand, have great potential for MICP treatments. MICP binds the sand grains together with calcium carbonate cement, causing the spatial cementation distribution to have two extremes. The first extreme is sand grain encapsulation, during which a thin coating of calcium carbonate surrounds the entire sand particle. The other, opposite spectrum distribution is that calcium carbonate only precipitates at the particle-particle contacts. It has been shown through visual analysis of SEM and X-ray computed tomography images that the actual cementation distribution is a combination of these two extremes. The calcium carbonate cement exhibits preferential precipitation at the particle contacts (DeJong et al 2010), which has much more significant implications on increasing the strength and stiffness of the soil when compared cementation coating the soil grain outside of the contact area, as shown in Figure 2.1.

In unsaturated environments, the retained treatment fluid will be retained within the menisci at the particle contacts as a result of inherent matric suction. This creates a more conducive...
environment for calcium carbonate to precipitate at the particle contacts while minimizing the encapsulation effect and allowing the cementation to have an optimal effect on mechanical soil behavior.

Degree of saturation also plays a substantial role in the strength and stiffness of MICP cemented sands (Cheng et al., 2013). Cementation distribution is varied by the degree of saturation; increasing saturation levels lead to more pronounced particle encapsulation, as shown in Figure 2.2a. Conversely, as saturation levels decrease, the tendency of the cementation begins to form preferentially at the particle-particle contacts improves.
dramatically. Figure 2.2 displays a more global image of the cementation distribution as a function of degree of saturation.

![Figure 2.2. Effect of saturation on spatial cementation distribution: a) 100% saturation, b) 20% saturation. (Modified from Cheng et al., 2013).](image)

As cementation level increases, mechanical properties of cemented sands increase. For both fine and coarse silica sand, it has been shown that as the degree of saturation decreases, the effect of cementation level on the increase in friction angle and the magnitude of the cohesion are amplified (Cheng et al. 2013). Intuitively, the change in friction angle is more pronounced for the coarse sand, and the cohesion is most pronounced for the finer sand (Cheng et al. 2013). The increases in these shear strength parameters, which are most magnified at lower degrees of saturation, lead to an increase in unconfined compressive strength for lower degrees of saturation at a constant cementation level. Additionally, as a result of cementation
(true cohesion), the stiffness increases as a function of cementation, an effect that is magnified at lower cementation levels (Cheng et al. 2013). In contrast to the drastic changes in mechanical properties, the hydraulic properties of the MICP cemented sands vary relatively uniformly with respect to degree of saturation.

Using unsaturated Ottawa 50-70 sand treated with a freshwater based treatment solution, Shanahan and Montoya (2014) presented a series of unconfined compressive strength tests with the corresponding average mass of calcites, as shown in Table 2.1

Table 2.1: Summarized Results from Shanahan and Montoya (2014)

<table>
<thead>
<tr>
<th>Column</th>
<th>Number of Treatments</th>
<th>Mass of Calcite (%)</th>
<th>Unconfined Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>101</td>
<td>7.0</td>
<td>2047</td>
</tr>
<tr>
<td>2</td>
<td>81</td>
<td>4.4</td>
<td>967</td>
</tr>
<tr>
<td>3</td>
<td>119</td>
<td>11.2</td>
<td>3736</td>
</tr>
<tr>
<td>4</td>
<td>62</td>
<td>1.2</td>
<td>70</td>
</tr>
</tbody>
</table>

**2.1.2 Literature Review on Saltwater Chemistry**

Prior to making a truly ethical recommendation for MICP treatment on coastal sand dunes, the chemistry in the pore space must be understand and the negative impacts minimized.
Due to the incessant need for a calcium containing ionic compound, salt water appears to be a viable alternative to fresh water as there is potential for reducing chemical costs. Based on a study by Mortensen et al. in 2011, the effect of varying amounts of seawater were used during MICP treatment and the effect on cementation level were observed. Although the treatment during this study was conducted under saturated conditions, the interaction between the saltwater and the induced MICP chemistry is discussed.

The saltwater was created at varying levels of salinity using the commercially available ‘Instant Ocean’, which is manufactured by Aquarium Systems, Inc. in Mentor, OH, USA. It had also been shown that the generation time of bacteria growth in the freshwater medium (with DI water) was shorter than the growth rate with salt water. Generation time is defined as the time it takes to for bacteria to double in number (Powell, 1956). Although the generation time is prolonged with the addition of salt into the nutrient medium, the final yield remained constant, which is indicative of the potential for MICP using saltwater as a feasible option. However, it would most likely require longer in order to reach similar levels. Using saltwater, Mortensen found that increasing salinity (up to 100% seawater) leads to an increased rate of calcium carbonate precipitation and much larger shear wave velocities.

Cheng et al. 2014 displayed some similar finding with regards to saltwater chemistry during the MICP reaction. One of the major findings of the study was that high strengths can be obtained using a saltwater-based treatment solution, though, in order to obtain similar strength levels as a freshwater solution, the number of treatments must be greater. This phenomena is most likely related to the generation time of the bacteria during fresh and saltwater conditions.
2.1.2 Review on MICP Chemistry

MICP is bio-geochemical process that leads to calcium carbonate precipitation within a soil matrix and is intended to induce changes in the mechanical properties of the soil. This reaction is driven through microbial urea hydrolysis which provides urea to the ureolytic bacteria before converting urea to ammonia and carbon dioxide gas. These gases diffuse through the bacterial cell membrane and undergo speciation (translation of ammonia to ammonium given that there is chemical equilibrium and urea concentrations are significantly higher than available calcium concentrations), which raises the local pH in the pore fluid and provides carbonate ions (Martinez, 2012), as shown in the chemical expression below.

\[
NH_2 CO \rightarrow NH_2 + 2H_2O \rightarrow 2NH_4^+ + CO_3^{2-}
\]

\[
Ca^{2+} + CO_3^{2-} \rightarrow CaCO_3(s) \downarrow
\]

In the presence of calcium, the carbonate ions precipitate with the calcium to form calcium carbonate. Typically, *Sporosarcina pasteurii* (ATCC 11859) is used as the ureolytic bacteria for laboratory testing due to it being readily available and most significantly, it has a high ureolytic activity (Bachmeier et al., 2002). By increasing subsurface alkalinity, subsurface microbes can promote calcium carbonate precipitation (Kohnhauser, 2007). For MICP implementation, there are two distinct methods for inducing urea hydrolysis, bio-stimulation and bio-augmentation (Fujita et al., 2008; DeJong et al., 2009).
During bio-stimulation, the objective is to stimulate indigenous bacteria to the desired population density, allowing for the MICP process to occur. This biological process eliminates the necessity of transporting ureolytic bacteria. When the ureolytic bacteria are not present, bio-augmentation is necessary, during which ureolytic bacteria are injected (or percolated in the case of surficial unsaturated treatment such as coastal sand dunes) into the subsurface prior to initiation of the MICP reaction.

2.2 Saltwater Treatment Testing Methods

With the addition of salt into the treatment solution, it is hypothesized that the presence of the salts will act partially as the calcium source, and the amount of calcium chloride needed in each treatment could potentially be reduced. It has been shown that under constant treatment conditions and when compared to freshwater, the calcium carbonate concentrations in saturated environments are higher for saltwater MICP treatments for Ottawa 50-70 sand (Mortensen et al. 2011). It has also been shown that more saltwater based treatments are needed to achieve similar strengths than when compared to freshwater treatments (Cheng et al. 2014).

2.2.1 Overview

In order to determine the effect of differing influent nutrient concentrations on unconfined compressive strength and cementation level, a series of soil column tests were
conducted. To reduce the amount of ammonium generated, the urea concentration should be as low as possible while still allowing for optimal strength levels. As coastal sand dunes are near the marine environment, it would be advantageous to have the ability to conduct the treatments using seawater, eliminating the need for transporting water. The overarching goal in this study is to evaluate the interaction of saltwater with the known MICP process and to determine the effect the presence of the salts has on the cementation level, distribution, and unconfined compressive strength.

To evaluate the interaction of saltwater and the effect of varying the chemical concentrations in the treatment solution, a series of unconfined compressive strength tests are to be performed to investigate the effects of the treatment solution on the mechanical strengths. As the aim is to optimize the solution with respect to induced chemistry and mechanical strength, the results of each set of soil columns will be used to design the chemical components of the treatments used in the next round of soil column tests. To further understand the saltwater chemistry, a pore fluid batch test will be conducted to observe the deviation of the equilibrium time (time for the MICP reaction to occur) based on several different treatment solutions. The results of this pore fluid batch test will be used in conjunction with the soil column tests results to optimize the treatment solution. Finally, after the saltwater treatment solution is optimized, several soil columns will be cemented to larger cementation ranges to understand the strength-cementation relationship at the optimized treatment solution. Following the formation of a strength-cementation relationship, SEM imagery is planned to observe the difference in cementation patterns and chemical composition.
2.2.2 Soil Column Materials and Methods

A series of unsaturated soil column tests were conducted with an objective of furthering the understanding of the pore space chemistry when subjected to saltwater based treatments. Six soil columns were used during the first two soil column tests and two columns were tested during the final two soil column tests. The soil column specimens were prepared using dry pluviation to a target void ratio of 0.70. North Carolinian dune sand and Ottawa 50-70 were used during this investigation, and a summary of the characteristics is listed in Table 2.2.

Table 2.2: Sand Characteristics

<table>
<thead>
<tr>
<th>Sand</th>
<th>D$_{50}$ (mm)</th>
<th>C$_{u}$</th>
<th>C$_{c}$</th>
<th>Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Carolinian Dune Sand</td>
<td>0.23</td>
<td>2.0</td>
<td>0.96</td>
<td>Round</td>
</tr>
<tr>
<td>Ottawa 50-70</td>
<td>0.22</td>
<td>1.4</td>
<td>0.9</td>
<td>Round</td>
</tr>
</tbody>
</table>

The chemical constituents of the Instant Ocean used to simulate seawater is shown in Table 2.3. The concentrations in Instant Ocean are remarkably close to the mean concentrations in natural seawater. According to the Principle of Constant Composition, the salinity and the chemical composition of major ions in sea water is constant from one ocean to another and from the surface to extreme depths (Pilson, 2008). This principle widens the applicability of this research to oceans world-wide without any major geographical bias.
associated with localized variations in ocean chemistry. Although Table 2.3 does not include sodium chloride or magnesium chloride, the reason for that is these compounds simply are not present in natural seawater in their compound form. Instead, they have dissolved or dissociated into cations and anions (Orr, 2008).

The soil column specimens have a 2 in. diameter and an aspect ratio of 2:1. The soil columns were designed to be free draining and have approximately 6 inches of crushed gravel placed above the sand to apply a low overburden stress, as shown in Figure 2.3. To prevent erosion loss of soil throughout testing, a 0.125 in. Porex filter was used above and below the specimen. Between the crushed gravel and Porex filter, a one in. thick and two in. diameter acrylic piece is used to dampen the gravitational effects of pouring the solution onto the sand sample. In order to allow for free drainage, sufficient holes were drilled through the acrylic. The 2 in. acrylic cell walls are placed onto a bottom cap and waterproofed by an O-ring seal. Plumbing in the bottom cap allows for the chemical effluent to leave the sample at a rate approximately equal to the permeability of the soil. Valves and PVC tubing are connected to the bottom cap, effluent pH measurements are recorded as the first drops to leave the tubing.
Table 2.3: Mean Concentrations of the Major Ions of Constituents of Natural Seawater and Instant Ocean Synthetic Sea Salt (Adapted from Orr, 2008)

<table>
<thead>
<tr>
<th>Ion</th>
<th>Natural Seawater (g/L)</th>
<th>Instant Ocean (g/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sodium (Na⁺)</td>
<td>10.781</td>
<td>10.780</td>
</tr>
<tr>
<td>Potassium (K⁺)</td>
<td>0.399</td>
<td>0.420</td>
</tr>
<tr>
<td>Magnesium (Mg^{++})</td>
<td>1.284</td>
<td>1.320</td>
</tr>
<tr>
<td>Calcium (Ca^{++})</td>
<td>0.4119</td>
<td>0.400</td>
</tr>
<tr>
<td>Strontium (Sr^{++})</td>
<td>0.00794</td>
<td>0.0088</td>
</tr>
<tr>
<td>Chloride (Cl⁻)</td>
<td>19.353</td>
<td>19.290</td>
</tr>
<tr>
<td>Sulfate (SO₄⁻)</td>
<td>2.712</td>
<td>2.660</td>
</tr>
<tr>
<td>Bicarbonate (HCO₃⁻)</td>
<td>0.126</td>
<td>0.22</td>
</tr>
<tr>
<td>Bromide (Br⁻)</td>
<td>0.0673</td>
<td>0.056</td>
</tr>
<tr>
<td>Boric Acid (B(OH)₃)</td>
<td>0.0257</td>
<td>-</td>
</tr>
<tr>
<td>Fluoride (F⁻)</td>
<td>0.00130</td>
<td>0.001</td>
</tr>
</tbody>
</table>
2.2.3 Testing Methods

Different treatments were used for each soil column test, with an ultimate goal to maximize unconfined compressive strength while minimizing influent nutrient molarities, more specifically, the amount of ammonia/ammonium generated. Sixty-two cementation treatments were conducted on each column for all soil column tests. These treatments have a 200 mL volume, which is approximately 2 pore volumes, with chemical constituents summarized in Tables 2.5, 2.6, 2.9, and 2.11. A treatment is conducted by pouring the 200 mL solution onto the gravel in the soil column and letting the solution medium percolate through the pore spaces. The pH of the influent and effluent are measured during each treatment and
used as a relative indication of the microbial activity. Although the variation of chemical components of each cementation flush vary by column, the other flush types referenced throughout this thesis are shown in Table 2.4.

Table 2.4: Chemical Constituents of the Biological and Urea Treatments

<table>
<thead>
<tr>
<th>Component Name</th>
<th>Biological Flush</th>
<th>Urea Flush</th>
<th>Biodose Flush</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urea</td>
<td>0.333M</td>
<td>0.333M</td>
<td>0.1M</td>
</tr>
<tr>
<td>ATCC11589</td>
<td>30mL</td>
<td>0</td>
<td>15mL</td>
</tr>
<tr>
<td>Calcium Chloride</td>
<td>0</td>
<td>0</td>
<td>0.05M</td>
</tr>
<tr>
<td>Instant Ocean</td>
<td>0</td>
<td>0</td>
<td>26.8 $\frac{g}{L}$</td>
</tr>
</tbody>
</table>

Upon completion of testing, the soil specimens were extruded using a Shelby Tube extruder, modified to extrude a two inch diameter specimen. The lightly cemented soil specimens were carefully measured and moved during the sample preparation. Unconfined compressive strength testing was conducted in accordance with ASTM D2166. The displacement strain rate used for all testing was $0.5 \% \frac{\text{min}}{\text{min}}$; testing was conducted until failure. After failure was reached, the load cell was lifted and samples were obtained from the top, middle, and bottom of the initial specimen to be used during acid washing testing.
The mass of calcium carbonate was determined post-testing using a gravimetric acid washing technique. The oven-dried mass of the soil sample from the rigid cell was measured before and after an acid wash (1M HCl). The dissolved calcium carbonate – acid wash solution was rinsed multiple times allowing the dissolved salts to be rinsed from the soil. The difference in the two measured masses was taken as the mass of calcium carbonate and the percentage of mass of calcium carbonate is expressed as the mass of calcium carbonate divided by the mass of soil. In order to form a composite correction for the North Carolinian dune sand, a series of acid washing tests were conducted on non-cemented North Carolinian dune sand. Factors that lead to this correction include the presence of fines and the degradation of calcium carbonate based shells. After the calcium carbonate degrades the acid is poured out of the beaker. The presence of fines in the soil make this stage difficult as the fines have a tendency to float and are difficult to see, allowing for fines to leave the beaker and disrupt the mass of calcite determination. Also, the chemical degradation of the non-soil elements, such as seashells had considerable influence on the dissolved mass.

### 2.2.4 Soil Column Test 1

**Overview**

The first soil column test consisted of six columns each with their own chemical constituents. As displayed in Table 2.5, three of the columns used constant urea but had differing concentrations of calcium chloride, due to the presence of salt components in the Instant Ocean. The other three columns had constant calcium chloride with differing levels of
urea, due to the alkalinity of the saltwater. As shown in Table 2.5, column 1 does not contain the saltwater simulating Instant Ocean. Column 1 was pluviated with Ottawa 50-70 sand instead of North Carolinian dune sand. The remaining five columns used both Instant Ocean and North Carolinian dune sand. The aim of holding one nutrient constant is to determine the resulting compressive and cementation behavior with increased or decreased quantities of that particular nutrient. Prior to performing the unconfined compressive strength testing, the six extracted soil specimens are displayed in Figure 2.4.
Figure 2.4: Specimens as Extracted for Soil Column Test 1: a) Column 1, b) Column 2, c) Column 3, d) Column 4, e) Column 5, f) Column 6

Table 2.5: Chemical Recipe for Soil Column Test 1

<table>
<thead>
<tr>
<th>Column</th>
<th>Void Ratio</th>
<th>Instant Ocean ($\frac{g}{L}$)</th>
<th>Urea (M)</th>
<th>CaCl₂ (M)</th>
<th>Cementation Flushes</th>
<th>Biodose Flushes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.70</td>
<td>0</td>
<td>0.333</td>
<td>0.10</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.71</td>
<td>26.8</td>
<td>0.333</td>
<td>0.05</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0.70</td>
<td>26.8</td>
<td>0.333</td>
<td>0.00</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0.70</td>
<td>26.8</td>
<td>0.200</td>
<td>0.05</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.70</td>
<td>26.8</td>
<td>0.100</td>
<td>0.05</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>0.69</td>
<td>26.8</td>
<td>0.050</td>
<td>0.05</td>
<td>62</td>
<td>0</td>
</tr>
</tbody>
</table>
Soil Column Test 1 Results

The results of the unconfined compressive and acid washing tests are displayed in Table 2.6 and the post-failure specimens are shown in Figure 2.5. As noted by the results of soil column 1 (Table 2.6), the unconfined compressive strengths are relatively low when compared to the same chemical treatments as Shanahan and Montoya (2014), which exhibited a 70 kPa unconfined compressive strength. In general, for the columns with constant urea (columns 1-3), the unconfined compressive strengths are relatively low. Some slumping due to self-weight of the sample occurred, which likely had negative repercussions on the unconfined compressive strengths.

Table 2.6: Strength and Cementation Levels for Soil Column Test 1

<table>
<thead>
<tr>
<th>Column</th>
<th>qu (kpa)</th>
<th>Mass of Calcite</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Top (%)</td>
<td>Middle (%)</td>
<td>Bottom (%)</td>
</tr>
<tr>
<td>1</td>
<td>2.9</td>
<td></td>
<td>0.37</td>
<td>0.22</td>
<td>0.16</td>
</tr>
<tr>
<td>2</td>
<td>1.4</td>
<td></td>
<td>0.49</td>
<td>0.23</td>
<td>0.02</td>
</tr>
<tr>
<td>3</td>
<td>3.7</td>
<td></td>
<td>0.27</td>
<td>0.28</td>
<td>0.26</td>
</tr>
<tr>
<td>4</td>
<td>7.4</td>
<td></td>
<td>0.69</td>
<td>0.41</td>
<td>0.28</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
<td></td>
<td>0.66</td>
<td>0.40</td>
<td>0.15</td>
</tr>
<tr>
<td>6</td>
<td>5.5</td>
<td></td>
<td>0.53</td>
<td>0.21</td>
<td>0.10</td>
</tr>
</tbody>
</table>
Due to these extremely low strength levels, disturbance from extrusion and moving the samples into the testing position is much more significant than the typical sample disturbance for unconfined compressive strengths on cohesive soils. A key indicator of an increased unconfined compressive strength however, is that the North Carolinian dune sand specimens maintained the ability of being self-supported, a feat typically not possible for non-cohesive soils. Although not intuitive, the unconfined compressive strength increases as the concentration of calcium chloride decreases (with constant urea), which may be associated with the availability of other salt components that are present in the Instant Ocean.

For the constant calcium chloride columns (columns 4-6), the strengths tend to decrease as the urea concentration decreases. This trend is not fully explained by this round of testing however, due to some difficulties during the unconfined compressive testing of column 5 where, no meaningful data was obtained. Although the unconfined compressive strength is unknown for this column, the cementation information is available, and on the basis of the trends of cementation, the strength is most likely in between the values obtained for columns 4 and 6. As expected based on the strength values, the cementation levels are much higher for the higher strength, constant calcium chloride columns.
Soil Column Tests 1 – Conclusion/Reassessment

In order to further understand the saltwater chemistry, the chemical solutions for a second round of soil column tests were designed based on the unconfined compressive strengths and cementation levels of the six specimens in soil column test 1. To add a more
microbial approach of understanding the saltwater chemistry, a series of pore fluid batch tests were to be conducted, as discussed in the section Pore Fluid Batch Tests. The highest strengths in the first round are associated with the lower urea concentrations, which may be due to the alkalinity of the Instant Ocean simulated saltwater. Meanwhile, the lower strengths were associated with the constant (and higher) urea. The surface percolation method of MICP typically exhibits crust-like cementation patterns, with higher levels of cementation at the surface, and decreasing with depth. This phenomenon is displayed in Table 2.6, with the cementation levels generally decreasing from the top to the bottom of each specimen. As displayed in Table 2.6, the strengths for the lower urea (columns 4-6) are greater than the strengths observed for the higher and constant urea.

By exhibiting higher strengths, while generating reduced quantities of ammonium, further investigation into the more environmentally friendly option becomes feasible and capitalizes on the two goals of this work: maximizing mechanical strength and simultaneously minimizing effluent concentrations. Based on the properties obtained through this series of testing, the pore fluid batch tests were designed by focusing on the strengths associated with the lower quantities of urea, which is the primary nutrient responsible for the generation of ammonia/ammonium. The chemical combinations used in the pore fluid batch tests have a peak urea concentration of 0.2M, the concentration used for column 4, which exhibited the highest unconfined compressive strength. The urea concentration for the remainder of the testing was decreased to attempt to determine and reduce the nutrients required in order to obtain optimal biochemical compatibility.
2.2.5 Pore Fluid Batch Tests

Overview

In order to further the understanding of the microbial behavior which occurs between the inoculated *Sporosarcina pasteurii* and the saltwater treatment medium, a series of ten batch tests were conducted. The purpose of these series of tests was to determine the amount of time required for the biochemical reaction between the solution medium and the bacteria to reach the maximum pH, or to reach equilibrium. During MICP, ureolytic microbes act as a catalyst to convert urea to ammonia and carbon dioxide, which react and raise the pore fluid pH to approximately 9.25 before calcium carbonate precipitation can occur. In an attempt to recreate this phenomenon and to evaluate the effects of differing urea and calcium chloride concentrations and their ratios, several solutions were mixed and used to determine the reaction time. To find the reaction time, 200mL of the solutions, displayed in Table 2.7, were mixed with 15mL of a *S. pasteurii* inoculated Ammonium-Yeast Extract media with an O.D. (optical density) of 1.0. While being stirred continuously by means of stir plate and stir rod, the pH of each solution was measured initially and at fifteen minute intervals until the pH remained constant for three consecutive readings (45 minutes). In a similar fashion as the selection of nutrient concentrations for the soil column tests 1, several different combinations of calcium chloride and urea were used to attempt to further understand the connection between the microbial reaction time and the expected level of activity when applied to soil.
Table 2.7. Pore Fluid Batch Test Recipes

<table>
<thead>
<tr>
<th>Batch</th>
<th>Urea (M)</th>
<th>CaCl₂ (M)</th>
<th>I.O. (g/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2</td>
<td>0.1</td>
<td>26.8</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
<td>0.05</td>
<td>26.8</td>
</tr>
<tr>
<td>3</td>
<td>0.2</td>
<td>0.025</td>
<td>26.8</td>
</tr>
<tr>
<td>4</td>
<td>0.1</td>
<td>0.1</td>
<td>26.8</td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>0.05</td>
<td>26.8</td>
</tr>
<tr>
<td>6</td>
<td>0.1</td>
<td>0.025</td>
<td>26.8</td>
</tr>
<tr>
<td>7</td>
<td>0.05</td>
<td>0.1</td>
<td>26.8</td>
</tr>
<tr>
<td>8</td>
<td>0.05</td>
<td>0.05</td>
<td>26.8</td>
</tr>
<tr>
<td>9</td>
<td>0.05</td>
<td>0.025</td>
<td>26.8</td>
</tr>
<tr>
<td>10</td>
<td>0.2</td>
<td>0.1</td>
<td>0</td>
</tr>
</tbody>
</table>

Results

The chemical constituents used in soil column test 2 are to be determined based on the results of the cementation and strength levels observed in soil column test 1 and the biochemical results of the pore fluid batch tests. The results of the pore fluid batch tests are displayed in Table 2.8. The first solution to reach the desired pH was batch 10, the solution without the addition of the Instant Ocean. By adding other salts into the solution, those included in Instant Ocean (Table 2.3), the time to reach equilibrium is extended. This
lengthened time may be a result of non-calcium based cementation, due to the abundance of salt types in the solution (Orr, 2008).

Table 2.8. Pore Fluid Batch Tests – Times to Reach Equilibrium

<table>
<thead>
<tr>
<th>Batch</th>
<th>pH after Time (hr.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>7.4</td>
</tr>
<tr>
<td>2</td>
<td>7.6</td>
</tr>
<tr>
<td>3</td>
<td>8.0</td>
</tr>
<tr>
<td>4</td>
<td>7.4</td>
</tr>
<tr>
<td>5</td>
<td>7.4</td>
</tr>
<tr>
<td>6</td>
<td>7.6</td>
</tr>
<tr>
<td>7</td>
<td>7.6</td>
</tr>
<tr>
<td>8</td>
<td>7.4</td>
</tr>
<tr>
<td>9</td>
<td>7.4</td>
</tr>
<tr>
<td>10</td>
<td>7.6</td>
</tr>
</tbody>
</table>
Conclusions/Reassessment

The basis of nutrient selection is the calcium chloride concentration and the time to reach a constant pH. As one of the primary goals of this thesis is to reduce the generation of ammonium, limiting the influent nutrient concentrations is the primary deciding factor during determination of the nutrient ratios for the next round of soil column testing. For the 0.2M urea triplicate, the time to reach biochemical equilibrium is constant for each level of calcium chloride. Due to this trend, the biochemistry reaches equilibrium in approximately similar time, allowing for the ideal reduction of calcium chloride at this level of urea to become a feasible option. A different trend occurs during the 0.1M urea triplicate; as the ratio of the molarities of urea to calcium chloride increases, the time to reach equilibrium decreases. As with the ideal scenario, the option with the least calcium chloride actually reaches equilibrium in a shorter time than the higher calcium chloride concentrations.

Due to the ability to reach equilibrium most rapidly and the concentration of calcium chloride minimized, the nutrient concentrations to be used from this category are the 0.1M urea and 0.025M calcium chloride. The final 0.05M urea triplicate behaves similarly to the first set, with the equilibrium time being relatively equivalent for each level of calcium chloride, although this time is nearly 0.5 hours longer than the equilibrium time for the 0.2M urea triplicate. Due to relatively constant biochemical equilibrium times, the nutrient dosage to be used for the soil column test is 0.025M calcium chloride with varying urea. As noted by the three different concentrations mentioned, the calcium chloride molarity is constant for all of the proposed soil column tests. By holding the calcium chloride concentration constant, the effects of differing urea concentrations can be observed through the differences in unconfined compressive strength and cementation distribution.
2.2.6 Soil Column Test 2

Overview

The second round of soil column tests uses three different nutrient combinations spread across the six soil columns. Duplicates are used for each nutrient dosage; where, the replicates were treated to an equal number of treatments with the equivalent dosages at the same times. Table 2.9 displays the chemical recipes used for the treatment during the second round of soil column tests and their corresponding initial void ratio (immediately after dry pluviation). Based on the observation that the strengths on the columns with constant calcium chloride and varying urea were higher than their higher urea counterparts, the second round of tests were designed to investigate further into the lower urea ranges. Simultaneously, with the intention of reducing the effluent ammonium based on the trend shown in Table 2.6, the highest strengths were shown for the lowest calcium chloride concentration, 0.05M. With the notion that it may be possible to further decrease the calcium chloride concentration for the saltwater based treatment solution based on the alkalinity of the Instant Ocean, the design molarity in this round was selected to be 0.025M. It was hypothesized that the existence of calcium and other salt components in Instant Ocean could provide the required ions for calcium carbonate (or any other carbonate) precipitation to occur. The as-extracted soil columns for soil column test 2 are shown in Figure 2.6.
The target void ratio, was 0.70, but the actual void ratios were considered acceptable as the variation was relatively low. To be most representative of the ideal in-situ bacterial stimulation, the bacteria was introduced into the soil once at the beginning of the treatments and again at the midpoint of the treatment schedule, in between cementation flushes 31 and 32.
At this time, a biological flush with chemical constituents shown in Table 2.4 was performed. Six hours after the biological flush, the next treatment was a consecutive urea and then cementation flush. The overarching purpose of the second biological flush was to stimulate the existing bacteria and to supply new bacteria (augmentation) to increase their biological activity, leading to a more productive calcium carbonate precipitation. The pH is typically used as an indicator of the MICP process as a result of the increase in localized pH from the speciation of ammonium into ammonia (Martinez, 2012).

Table 2.9: Chemical Recipes for Soil Column Test 2

<table>
<thead>
<tr>
<th>Column</th>
<th>Void Ratio</th>
<th>Instant Ocean (\frac{g}{L})</th>
<th>Urea (M)</th>
<th>CaCl₂ (M)</th>
<th>Cementation Fluxes</th>
<th>Biodose Fluxes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.71</td>
<td>26.8</td>
<td>0.2</td>
<td>0.025</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.71</td>
<td>26.8</td>
<td>0.2</td>
<td>0.025</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0.70</td>
<td>26.8</td>
<td>0.1</td>
<td>0.025</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0.69</td>
<td>26.8</td>
<td>0.1</td>
<td>0.025</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.70</td>
<td>26.8</td>
<td>0.05</td>
<td>0.025</td>
<td>62</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>0.71</td>
<td>26.8</td>
<td>0.05</td>
<td>0.025</td>
<td>62</td>
<td>0</td>
</tr>
</tbody>
</table>
Results

For the second round of soil column tests with nutrient concentrations and flush information presented in Table 2.9, the unconfined compressive strength and mass of calcite results are displayed in Table 2.10 below. The exceptionally low average unconfined compressive strengths are relatively similar, with a trend of decreasing strength with decreasing influent urea concentration after 0.1M. Handling the samples at such low cementation and strength levels is difficult which may have potentially led to some sample disturbance during extrusion and during the preparation for unconfined compression testing. Such low strength levels result in slumping of the bottom, more lightly cemented, portion of the cylindrical sample under self-weight. The cementation level for the 0.2M urea is much higher than the other two nutrient recipes without reciprocating a substantially higher unconfined compressive strength. As urea is the component that attributes to the generation of effluent ammonium/ammonia, the chemical recipe that allows for the lowest urea concentration while still maintaining the possibility of a reasonably high unconfined compressive strength is most ideal for MICP application in-situ. Between the 0.2M and 0.1M urea concentration soil columns, the unconfined compressive strengths are similar, although the 0.2M urea concentration produced more cementation. As the goal of this cementation-strength evaluation is to optimize the chemical recipe and the unconfined compressive strength, the concentration difference between the two rounds is unnecessary. It should be noted however, that due to such low cementation levels, the unconfined compressive strengths above are heavily sensitive to sample disturbance that may have occurred during extraction.
Table 2.10: Strength and Cementation Levels for Soil Column Test 2

<table>
<thead>
<tr>
<th>Column</th>
<th>q_u (kpa)</th>
<th>Q_u(average) (kpa)</th>
<th>Mass of Calcite</th>
<th>Replicate Average (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Top (%)</td>
<td>Middle (%)</td>
</tr>
<tr>
<td>1</td>
<td>7.1</td>
<td>6.5</td>
<td>0.53</td>
<td>0.50</td>
</tr>
<tr>
<td>2</td>
<td>5.8</td>
<td>0.51</td>
<td>0.43</td>
<td>0.26</td>
</tr>
<tr>
<td>3</td>
<td>7.7</td>
<td>6.5</td>
<td>0.46</td>
<td>0.39</td>
</tr>
<tr>
<td>4</td>
<td>5.3</td>
<td>0.48</td>
<td>0.30</td>
<td>0.09</td>
</tr>
<tr>
<td>5</td>
<td>5.1</td>
<td>5.7</td>
<td>0.45</td>
<td>0.38</td>
</tr>
<tr>
<td>6</td>
<td>6.3</td>
<td></td>
<td>0.47</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Some of the soil specimens in Figure 2.6 exhibit some slumping under self-weight. This effect is compounded further by the cementation distribution that is shown in Table 2.10. This slumping effect and sample disturbance likely have reduced the strengths of the soil specimens prior to performing the unconfined compressive testing. However, it is assumed herein that as a result of the extraction process and sample preparation being identical for all of the samples, the disturbance is equally distributed and in terms of a sensitivity analysis, they are sufficient. The strength differences between the 0.2M and the 0.1M urea for an equivalent
treatment scheme is negligible, which is an obvious trade-off for a factor of 2 reduction in effluent ammonium/ammonia.

Figure 2.7: Specimens at Failure for Soil Column Test 2: a) Column 1, b) Column 2, c) Column 3, d) Column 4, e) Column 5, f) Column 6.
Conclusions/Reassessment

There was no strength reduction through the reduction of the urea concentration from 0.2M to 0.1M during soil column test 2, although the average cementation level for the 0.2M urea tests were approximately 40% greater than that of the 0.1M urea columns. As the target application in this evaluation are coastal sand dunes, having a lower cementation level is ideal to allow for the continuation of life for coastal flora and fauna, which may be inhibited by higher cementation levels. The conclusion from the pore fluid batch tests was that the effect of urea concentration needed to be further understood, and from this study it had been concluded that 0.1M urea appears to be the optimized concentration for saltwater treatment. This conclusion is based on there being no further strength increase as a result of an increase in influent urea concentration.

Based on the results of the pore fluid batch tests results, the 0.05M calcium chloride concentration (with 0.1M urea) reached equilibrium prior to the 0.025M used in this round of soil column tests. Combined with the lower strengths observed for the 0.025M calcium in test 2 with the higher strengths for the 0.05M calcium chloride in soil column 1, the design calcium chloride concentration for the optimization of the saltwater based MICP treatment is 0.05M. Now that the optimized chemical treatments have been obtained, there is an incessant need for understanding the mechanical behavior at larger cementation levels, as in-situ design strengths would be greater than that observed during the first two rounds of soil column tests. There was some skepticism about the ability of the North Carolinian dune sand to undergo significant cementation, as the observed strengths during the first two rounds of testing were considerably lower than expected.
2.2.7 Soil Column Test 3

Overview

After assessing the optimized nutrient concentrations for strength and ammonium/ammonia concentration, the need to understand the MICP strength behavior at higher cementation levels became evident. For comparisons sake, the treatment methodology for this round of soil columns was derived using the same number of cementation flushes (62), with a biological flush in between cementation flushes 31 and 32. The new approach taken during this round included the addition of biodosing, which is essentially a cementation flush that includes the uerolytic bacteria that was formerly introduced in biologicalFlushes alone (Martinez 2012). It was hypothesized that for the unsaturated conditions, the need for more frequent bacterial stimulation or augmentation was required for achieving the expected strength levels for a particular amount of treatments. Unlike the previous two soil column rounds, the third round included only two soil columns, which were replicates at the optimized chemical compositions. The as extracted soil specimens are displayed in Figure 2.8.

As shown in Figure 2.8b, the soil specimen from column 2 had a horizontal crack through its cross section approximately 2/3 of the way down the sample, which likely contributed to a lower strength than the unbroken counterpart in Figure 2.8a. Compared to the specimens from the first two rounds of soil columns, the cementation is visible, in that there is no obvious damage as a result of the extrusion and the diameter is completely constant through the cross-section. Sample 2 was damaged again during the removal of the bottom Porex filter, which had cemented to the sand and when removed, a portion of the sample was removed as well. Sample 1 was visually undamaged, however it is possible that disturbance occurred
during extraction. The chemical constituents and the flush information is displayed in Table 2.11.

![Figure 2.8: Specimens as Extracted for Soil Column Test 3: a) Column 1, b) Column 2](image)

The number of biodoses were selected to be comparable to the number of treatment for the most heavily cemented column used in Shanahan and Montoya (2014), which used Ottawa 50-70 sand, for the sake of comparison to a different sand type, although treated with freshwater. During the biodose phase of the testing, no further biological flushes were conducted as the required bacterial augmentation was being achieved during each individual treatment, minimizing the periods of lower microbial activity. During the testing pH monitoring, the pH increase for the previous two rounds of soil column tests ranged from 0.0-
0.4, while during the biodose flushes, the pH increased to 9.0, up 3.0 from the influent pH. The sharp rise in pH during biodose treatments is another indication of chemical precipitation within the soil matrix, although some of the precipitation may not be at the particle contacts and leave the system through the effluent.

Table 2.11: Recipes for Soil Column Test 3

<table>
<thead>
<tr>
<th>Column</th>
<th>Void Ratio</th>
<th>Instant Ocean ( \text{g/L} )</th>
<th>Urea (M)</th>
<th>CaCl(_2) (M)</th>
<th>Cementation Flushes</th>
<th>Biodose Flushes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.69</td>
<td>26.8</td>
<td>0.1</td>
<td>0.05</td>
<td>62</td>
<td>59</td>
</tr>
<tr>
<td>2</td>
<td>0.69</td>
<td>26.8</td>
<td>0.1</td>
<td>0.05</td>
<td>62</td>
<td>59</td>
</tr>
</tbody>
</table>

Results

Upon completion of the much more heavily cemented soil columns, the samples were extracted and measured prior to unconfined compressive strength and gravimetric acid washing testing was performed. The post compression, failed specimens are displayed in Figure 2.9 below. During shearing, both of the soil specimens failed in the lower and much less cemented bottom segment of the sample. The second column displayed significantly lower strengths.
than the first, which was most likely a direct result of the extrusion related horizontal crack that extended throughout the entire cross section. The strength and cementation levels for the third soil column test is displayed in Table 2.12. In contrast to the first two rounds of soil column tests, the cementation levels shown in this round are substantially higher, on the order of two magnitudes. The effect of the more heavily cemented sand on the failure type is shown in Figure 2.9, as the failure appears to be a result of an outer crust failing instead of the entire soil mass as shown by the failure patterns in the previous tests. The failure scheme changes from that of the sand to a predominately cementation failure.

![Specimens at Failure for Soil Column Test 3: a) Column 1, b) Column 2.](image)

Figure 2.9: Specimens at Failure for Soil Column Test 3: a) Column 1, b) Column 2.

The unconfined compressive strength for the second column is substantially lower than that of the first column, likely stemming from the damage during extraction. It is probable that
if the crack had not formed or was prevented from propagating, the unconfined compressive strength would have been similar to that observed in column one, based on a similar cementation magnitude and distribution.

Table 2.12: Strength and Cementation Levels for Soil Column Test 3

<table>
<thead>
<tr>
<th>Column</th>
<th>$q_0$ (kpa)</th>
<th>Mass of Calcite</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top (%)</td>
</tr>
<tr>
<td>1</td>
<td>1290.5</td>
<td>15.53</td>
</tr>
<tr>
<td>2</td>
<td>808.3*</td>
<td>14.56</td>
</tr>
</tbody>
</table>

*The unconfined compressive strength is lower than the actual due to sample disturbance.

Conclusions/Reassessment

After finishing the chemical optimization in soil column test 2, the objective became furthering the understanding of saltwater based MICP at higher cementation levels, as the previous rounds of testing had an average cementation level of less than 0.5%. The previous skepticism about achieving high cementation levels was debunked as shown by the average cementation levels in Table 2.11. As a result of this round of soil column tests, the ability of the North Carolinian dune sand to undergo large amounts of cementation was confirmed, although this level of cementation would be devastating to the systematic ecology of coastal
sand dunes. Stemming from the accumulation of data from the first two rounds of testing, which are less than 0.5% average cementation, and the third set, which was around 11.5% average cementation, it was concluded that some information in the relative center of these data points were needed to evaluate the mechanical behavior of saltwater MICP.

2.2.8 Soil Column Test 4

Overview

Based on the reassessment following soil column test 3, a new treatment scheme was devised with the fundamental assumption that the majority of the cementation occurs as a result of the biodose treatments. This assumption was made on the basis of the results of the first two soil column tests, which achieved exceptionally small cementation levels after 62 cementation flushes. To further validate this assumption, the addition of the biodose flushes (59) amplified the cementation levels by a factor greater than 20. With an aim of creating specimens at the center of the two data ranges, the number of biodose treatments was halved, at 29 total, as shown in Table 2.13. The flush constituents were identical to those in soil column test 3, the optimized chemical recipe. Shown in Figure 2.10 are the soil columns as extracted for the fourth round of testing. Similar to the previous round, one of the columns displayed a horizontal crack in the lower third of the column which likely contributed to the strength reduction when compared to the other, non-fractured column. The specimens displayed are similar to their highly cemented counterparts in soil column test 3, as the cementation induced increased strength led to a more uniform cross section than in the first two rounds. There was
some surficial damage during the extrusion of the first column, however the effect was most-likely not significant, as sample disturbance during extraction occurred on all of the soil columns tested here.

Table 2.13: Recipes for Soil Column Test 4

<table>
<thead>
<tr>
<th>Column</th>
<th>Void Ratio</th>
<th>Instant Ocean ($g/L$)</th>
<th>Urea (M)</th>
<th>CaCl$_2$ (M)</th>
<th>Cementation Flashes</th>
<th>Biodose Flashes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.69</td>
<td>26.8</td>
<td>0.1</td>
<td>0.05</td>
<td>62</td>
<td>29</td>
</tr>
<tr>
<td>2</td>
<td>0.69</td>
<td>26.8</td>
<td>0.1</td>
<td>0.05</td>
<td>62</td>
<td>29</td>
</tr>
</tbody>
</table>

Figure 2.10: Specimens as Extracted for Soil Column Test 4: a) Column 1, b) Column 2
Results

Although the intention was to produce a set of soil columns for unconfined compressive strength testing at approximately an average cementation level of 6.0%, the average cementation level was approximately 3.5%. This puts a leftward skew on the data that had been planned for the middle of the cementation range, but does still provide valuable insight about moderately cemented MICP sand. As expected by the large cementation gradient as a function of depth, the failure surface exists primarily in the bottom, where there is much less cementation.

Table 2.14: Strength and Cementation Levels for Soil Column Test 4

<table>
<thead>
<tr>
<th>Column</th>
<th>q_u (kpa)</th>
<th>Mass of Calcite</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top (%)</td>
<td>Middle (%)</td>
</tr>
<tr>
<td>1</td>
<td>176.7</td>
<td>5.79</td>
<td>3.75</td>
</tr>
<tr>
<td>2</td>
<td>124.3</td>
<td>5.76</td>
<td>3.59</td>
</tr>
</tbody>
</table>

As shown in Figure 2.11, the failure surface remained beneath the extrusion crack, and for illustration, the top segment of the column was lifted to show that after unconfined compression failure, the damage occurred solely in the bottom fragment. For column 2, it can be shown in the failed specimen, the previously discussed cementation crust as the failure
surface is not horizontal but formed around the stronger and more cemented outer shell of cementation. When comparing the average cementation levels and their corresponding unconfined compressive strengths, it becomes evident that the cementation-strength relationship is nonlinear.

![Figure 2.11: Specimens at Failure for Soil Column Test 4: a) Column 1, b) Column 2](image)

**Conclusions/Reassessment**

Upon completion of the fourth round of soil column testing, a more fundamental understanding of the moderate cementation zone was achieved for unsaturated, saltwater based MICP for North Carolinian dune sand. The unconfined compressive strengths were, as expected, significantly lower than that of the 59 biodoses (soil column test 3). From this round of testing, a couple of observations/conclusions were noted. The first of which, the
cementation-strength relationship is non-linear and potentially parabolic. Secondly, cementation levels do not behave linearly with the number of treatments. Although counterintuitive, based on the cementation levels and number of treatments in soil column test 3 and 4, the average amount of cementation per treatment increased as the number of treatments increased. This observed phenomenon may be associated with the increased surface area and surface contacts (soil-soil, cementation-soil, and cementation-cementation), which allow for the creation of more pore space menisci and consequently more chemical precipitation from each treatment. Furthering this notion, as the number of contacts increases, causing an increases in the cementation level, the strength therein increases at a more rapid pace.

Based on a compilation of all of the unconfined compressive strengths and mass of calcite measurements for each of the soil column tests, Figure 2.12 was created. This figure shows a large disparity between the freshwater and saltwater data in terms of strength compatibility at the same cementation level, which is compatible with other published literature (Cheng et al., 2014). Another potential factor that led to the deviation in strengths, is that the freshwater data is from Shanahan and Montoya (2014), as summarized in Table 2.1, which uses Ottawa 50-70 sand instead of the dune sand.

It is speculated that changes in influent chemistry altered the cementation patterns and distributions, resulting in differences in observed unconfined compressive strengths. To confirm these assumptions, SEM imagery was obtained using elemental analysis to investigate the cementation at the micro-scale.
2.2.9 Micro-Scale Evaluation

To investigate variations in cementation patterns resulting from the addition of Instant Ocean into the treatment media, scanning electron microscopy (SEM) was used to obtain high-resolution images for visual and elemental analyses. Prior to taking images, the sand samples were powder-coated with gold and palladium to enhance the optical clarity. Samples from saltwater treated North Carolinian dune sand were used and compared with freshwater treated Ottawa 50-70. It was hypothesized based on the reduction in unconfined compressive strength shown between the freshwater and saltwater based treatments, that the addition of the fourteen elements in Instant Ocean (Table 2.3) had a chemical alteration of the MICP equation, which...
resulted in cementation changes that cannot be seen at the macro-scale. The breakdown of where each of the samples used for SEM were obtained from is displayed in Table 2.15.

Table 2.15: Strength and Cementation Levels for Soil Column Test 4

<table>
<thead>
<tr>
<th>Figure</th>
<th>Soil Column Round</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.13</td>
<td>Shanahan and Montoya (2014)</td>
<td>1</td>
</tr>
<tr>
<td>2.14</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>2.15</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>2.16</td>
<td>1</td>
<td>6</td>
</tr>
</tbody>
</table>

Based on freshwater treatment of the Ottawa 50-70 as shown in Figure 2.13, the cementation in most heavily concentrated at the particle contacts, but the particle encapsulation effect is observed as well. It seems to be the case that the initial cementation occurs at the particle-particle contacts during the beginning stages of MICP treatment, but as cementation increases, the amount of surficial cementation (not at the particle-particle contacts) increases. It should be noted too, however, that the average mass of calcite for the freshwater treated Ottawa 50-70 is greater than that of the other two North Carolinian dune sand. Based on the SEM energy-dispersive X-ray spectroscopy, the cementation was determined to be predominately calcium carbonate.
The saltwater based treatments of the North Carolinian dune sand are shown in Figures 2.14, 2.15, and 2.16. Three distinct cementation patterns are observed visually between the three images, Figure 2.14 shows plate-like calcium carbonate precipitation, Figure 2.15 shows a more traditional meniscus shaped calcium carbonate cementation pattern, and Figure 2.16 displays a sodium-based cementation. Figures 2.14 and 2.15 are the sand soil particles, but focused on different cemented particle-particle contacts. Availability of differing nutrients may have led to the observed changes in cementation patterns.

The menisci shaped precipitation shown in Figure 2.15 reflects the notion that in unsaturated conditions, as a function of degree of saturation and matric suction, the nutrients are concentrated within the menisci. As a direct result of the concentration of nutrients at the menisci, cementation occurs preferentially at the particle-particle contacts. This cementation pattern aligns well with expected cementation patterns and is predominately calcium carbonate.

The last series of SEM images are shown in Figure 2.16. Based on the SEM elemental analysis, it was shown that unlike the freshwater based treatment solution, the chemical precipitation between the particle contacts after the addition of Instant Ocean was not calcium-based, but was sodium-based. This observation is crucial for explaining the mechanism that causes the variations in mechanical strength behavior (shown in Figure 2.12), which was associated with the inclusion of Instant Ocean into the treatment solution. As expected based on the chemical changes, the strengths of the sodium-based cementation are substantially different than the more traditional MICP calcium-based cementation. The reduction in strengths stemming from the addition of Instant Ocean into the treatment solutions are significant as shown by Figure 2.12. From these SEM images and their corresponding
elemental analysis, it is hypothesized that the lower strengths were associated with bond breakages in the weaker sodium-based cementation. This breakage at a lower applied stress resulted in a reduction in unconfined compressive strengths, when compared to solely calcium-carbonate based cemented sands.

Figure 2.13: SEM Images of Ottawa 50-70 Sand Treated with Freshwater with Magnification

a) 100x, b) 200x, c) 500x

Calcium-based cementation
Figure 2.14: SEM Images of North Carolinian Dune Sand Treated with Saltwater (1) with Magnification a) 200x, b) 500x, c) 1000x
Figure 2.15: SEM Images of North Carolinian Dune Sand Treated with Saltwater (2) with Magnification a) 200x, b) 500x, c) 1000x

Calcium-based cementation
Figure 2.16: Scanning Electrom Microscopy (SEM) Images of North Carolinian Dune Sand Treated with Saltwater (3) with Magnification of a) 100x, b) 200x, and c) 500x

Sodium-based cementation

Salt Grain
2.3 Conclusions and Recommendations for Coastal Treatment

_Treatment Optimization Conclusions_

The objectives for this evaluation were to optimize the chemistry of the saltwater MICP and to understand the cementation-strength relationship at the optimized chemical recipe. Furthermore, broadening the understanding of the effect of sand type on the cementation was a significant goal. During the first round of soil columns, six specimens were used: three with constant urea and three with constant calcium chloride. The constant urea and calcium chloride columns each had a different concentration of the other nutrient, with the intention of being able to evaluate general trends about varying a particular nutrient while elimination the other variable. From the first soil column test, the general trends indicate that for a constant urea concentration and saltwater treatments (columns 2 and 3), the unconfined compressive strength for the increased as the concentration of influent calcium chloride decreased, even at 0M calcium chloride. Although, the column that used freshwater (column 1) performed better, and with an opposite strength trend (higher calcium chloride lead to a higher strength). It is likely that the higher strength exhibited was a result of the lack of saltwater in the treatment solution. Since the freshwater-based solution relies on the calcium chloride to form the cementation, there are four other ions (Table 2.3) that may bond with the carbonate ions to chemically precipitate at the particle contacts. After the completion of this round of testing, there was not a decisive direction to take the design of the chemical concentrations for the next round, so it was decided to conduct a biochemical pore fluid batch test.

The premise of the pore fluid batch test was to examine the compatibility and reaction times of potential chemical influent concentrations. To design this test, three concentrations
of both components were used: Urea - 0.2M, 0.1M, and 0.05M, and calcium chloride – 0.1M, 0.05M, and 0.025M. Each of the above concentrations were paired with one of the aforementioned concentrations and the time to reach equilibrium (pH ~9-9.2) was evaluated by taking a pH measurement every fifteen minutes. Based on the findings about the different recipes, several conclusions can be drawn. In general, as the concentration of urea decreased, the time to reach equilibrium decreased. There was an exception for the saltwater analysis, which occurred only in the event where the concentration of the calcium chloride was greater than that of the urea, which appears to be the limiting reagent in this scenario. For the remainder of the tests, the time to reach equilibrium varied from 1-1.5 hours. In regards to the varying calcium chloride concentration, in general, for a constant urea concentration, the time to reach equilibrium increases slightly for decreasing calcium chloride concentrations. The effects of the 0.05M and 0.025M calcium chloride appear to be identical in terms of equilibrium time, so to continue forward, the new chemical optimization philosophy became evaluating the strength characteristics associated with a constant 0.025M calcium chloride and varying urea concentrations.

Following the pore fluid batch test, soil column test 2 was conducted using the 0.025M calcium chloride concentration with the following urea concentrations for unconfined compressive strength testing: 0.2M, 0.1M, and 0.05M. After completion of the unconfined compression and acid wash testing, it was concluded that for a constant calcium chloride concentration of 0.025M, as the urea concentration decreased from 0.2M to 0.1M, there was no reduction in average unconfined compressive strength although there was a significant reduction in average cementation level. This is indicative of the excess urea in the 0.2M solution precipitating in a less efficient manner than the 0.1M. It is speculated that the
cementation created during the 0.2M urea precipitated more sufficiently around the individual sand grains, as expected during saturated conditions, than the cementation generated during the 0.1M solution. The third concentration of urea was 0.05M, which exhibited a lower unconfined compressive strength, but only a slightly lower cementation level. As there was a more substantial strength reduction than there was a difference in average cementation, the 0.1M urea concentration was perceived as the design concentration for saltwater applications.

After a comparison of the strengths obtained in soil column test 2 with the strengths reported in soil column test 1, it was observed that for 0.1M urea the strengths for the 0.025M calcium chloride column was lower than the 0.05M calcium chloride in soil column 1, as such, the design calcium chloride concentration was determined to be 0.05M.

After the strength-chemical optimization conducted during the first two rounds of soil column tests and the pore batch fluid test, the next logical step was to understand the mechanical behavior at larger cementation levels, as the design strength would likely be considerably larger than the strength levels reported in the first two rounds of tests. The third test introduced biodosing into the testing scheme and allowed for an average cementation level of approximately 11.5%, contrasting to the fractions of a percent obtained in the first two rounds of testing. Performing the acid washing post-compression lead to a cementation gradient of approximately 8-9% which illustrates the crust-like cementation distribution as discussed before, yet this observation is on a much larger scale. With the unconfined compressive strengths of around 1,000 kPa, the ability of North Carolinian dune sand to precipitate heavily was confirmed, although these cementation levels would never be recommended for the usage in coastal sand dune resiliency, understanding the full mechanical behavior is significant for other applications using saltwater.
Created as a construct of soil column tests 2 and 3, a significant zone of the cementation level scale was devoid of data. To make generalizations about the expected strengths as a function of cementation level, the trends at intermediate cementation levels increase should be evaluated. The design premise of the final round of soil column tests was to provide insight into the middle ranges, and to show the changes in strength incrementally with increases in cementation. It was shown that the shape of the cementation-strength curve is non-linear and it plots well with a parabolic regression.

All of the cumulative data points for the four rounds of soil column tests were plotted on a cementation versus unconfined compressive strength plot. As there is a lack of published literature that investigates MICP using a saltwater solution in unsaturated environment, the results are plotted with unsaturated freshwater-based unconfined compression data by Shanahan and Montoya (2014). As shown in Soil Column Test 1 – column 1, the cemented strengths of the North Carolinian dune sand are similar to that of the Ottawa 50-70. Figure 2.12 shows the relationship between the strengths and cementation levels associated with fresh and saltwater. The variation in strengths associated with the saltwater treatments may have been attributed to sodium based precipitation. Clearly, the saltwater based MICP treatments resulted in much lower strengths for a given cementation level, however, economically, it may be more cost-effective to conduct a larger number of chemical treatments in-situ, than to transport freshwater to the site for each treatment.

Recommendations for Coastal Treatment

Prior to coastal treatments, changes in ecology must be assessed as coastal sand dunes are ecosystems with close proximity to groundwater and many different flora and fauna.
Changes in soil stiffness may negatively affect burrowing animals and plants, changes in oxygen availability may impact indigenous bacteria and flora root structures. The depth of impact for effluent ammonium/ammonia and the interaction with the soil, plants, and potentially the groundwater is one of the most significant parameters to understand prior to in-situ application.

For in-situ application of MICP to coastal sand dunes, recommended strengths would be on the lower range of cementation, on the order of 1.0% to 3.0%. Although as cementation level increases, the differential in freshwater and saltwater based strengths become more evident, at the lower cementation ranges, the deviation is much less significant. Stemming from cost-reduction in terms of water transport, the saltwater based treatments become feasible and likely recommendable. Similar strengths occur at the lower cementation levels, which potentially allows for treatment type to vary on a site to site basis dependent upon a balance between proximity to water for treatment and slightly reduced saltwater treatment strengths.
3 CRITICAL SHEAR STRENGTH EVALUATION OF MICP CEMENTED SANDS

3.1 Background

Scour is a specific form of erosion, during which sediment is removed through hydrodynamic forces acting parallel to the sediment bed. The rate of scour is typically magnified near structures, as a result of wave reflections and wave downrush (Reeve et al. 2012). Localized scour near the toe of structures can relieve earth pressures and reduce skin friction used in design – both of which contribute to global stability of the structure. As shown in Figure 3.1, bridges that span over rivers and other waterways are especially susceptible to large scour magnitudes, which has led to serious structural damage in a vast number of bridges (Briaud et al. 2002). In order to prevent scour-related damages to bridge abutments and bridge piers, methodology for evaluating the scour potential and erosion rate of the soil should be investigated. Viable options for improving the soil to mitigate these excessive losses to scour evaluated afterwards. Due to the spatial cementation distribution shown by Shanahan and Montoya (2014), during unsaturated surface percolation, MICP forms a cementation gradient. During this gradient, the surficial soils are more heavily cemented and the cementation level decreases with depth. This cementation gradient for unsaturated soils makes an application to scour appealing, as a lower average cementation level may be used, since the surface is the critical location for resiliency. Typically, as a result of permeability issues associated with
fine-grained soils, the dominate soil type for MICP are sands, as they maintain a large number of contact points for cementation (Dejong et al. 2010), while gravels have significantly less.

Figure 3.1: Excessive Scour of a Bridge over Montezuma Creek

[Adopted from: UDOT Transportation Blog. May, 3 2012]

3.1.1 Literature Review on Soil Erosion and Critical Shear Stress

Soil erosion has three predominant mechanisms: surface erosion and mass erosion, first observed by Partheniades (1965), and fluidization, as first documented by Mehta (1991). Fluidization occurs when a loss of shear strength in granular soils leads to fluid-like behavior (Alsaydalani and Clayton, 2013). Building on the knowledge obtained from Partheniades
(1965) and Mehta (1991), Briaud et al (2001) discussed two mechanisms of surface erosion for clean sands and gravels: sliding and rolling. During sliding, the resultant forces on the soil particles (assumed to be spherical), are not impacted by the neighboring particles as a result of a lack of relative movement between the aforementioned particles. This simplification reduces the force equilibrium to two components: the parallel acting resultant from the water on the particle and the oppositely acting frictional resistance. During the analysis of clean sands and gravels, the effects of electrostatic and electromagnetic forces may be ignored. Figure 3.2(a) displays the free body diagram used for the simple analysis of sliding. Briaud et al. (2001) also explained the rolling phenomenon, during which, the water exerts a parallel shear force on the top surface of the particle, causing a rotation about the contact point with the neighboring particle (although the neighboring particles do not impart any forces on the particle of interest). This mechanism at the onset of incipient motion, is observed in Figure 3.2(b).

Figure 3.2: Surface erosion mechanisms: a) sliding, b) rolling. [Adapted from Briaud et al. 2001]
Putting the two mechanisms in quantitative terms, sliding can be evaluated using force equilibrium (White, 1940).

\[ \tau_c A_e = W \tan \phi \]  
(Briaud et al. 2001)

Where,  
\( \tau_c \) = Critical Shear Stress  
\( A_e \) = Effective Friction Area of Water on the Particle  
\( W \) = Submerged Weight of the Particle  
\( \phi \) = Friction Angle of the Interface between Two Particles

After some algebraic manipulation and the assumption of the particles being a sphere, the critical shear stress can be expressed:

\[ \tau_c = \frac{2(\rho_s - \rho_w)g \tan \phi}{3 \alpha} D_{50} \]  
(Briaud et al. 2001)

Where,  
\( \rho_s \) = Mass Density of the Particle  
\( \rho_w \) = Mass Density of Water  
\( g \) = Acceleration Due to Gravity  
\( \alpha \) = Ratio of the Effective Friction Area over the Maximum Cross Section of the Spherical Particle  
\( D_{50} \) = The Mean Diameter Representative of the Soil Particle Size Distribution
Using the Erosion Function Apparatus, (EFA), Briaud et al. (1999) showed the approximate relationship, applicable for clean sands and gravels.

\[ \tau_c \left( \frac{N}{m^2} \right) \approx D_{50} \text{ (mm)} \]  
(Briaud et al. 2001)

The expression above was concluded after some experimental testing using the EFA (Briaud et al 1999). Following this conclusion, there was an inconsistency with the simple theoretical model for sliding and the experimentally observed critical shear stresses, indicating that sliding was not the primary erosion mechanism. Investigating the rolling mechanism using moment equilibrium at the onset of incipient motion, a critical condition proposed by Brahms (1753) for which sediment begins to move under the action of flow, the expression for rolling was deduced (White, 1940) to be:

\[ \tau_c = \frac{2(\rho_s - \rho_w)g \sin \beta}{3 \alpha (1 + \cos \beta)} D_{50} \]  
(Briaud et al. 2001)

Where, \( \beta \) = Contact Angle between Grains

Sliding and rolling are theoretical mechanisms for the erosion of soils, however (Briaud et al. 2001) recommends that experimentation is better suited for a more representative critical shear stress than the purely theoretical models. From an experimental perspective, Shields (1936) proposed the following relationship between critical shear stress and the median grain
size, based on laboratory flume tests which studied water flowing over flat beds of sand, as summarized by Kebede (2014).

\[ \tau_c = 0.63D_{50} \quad \text{Shields (1936)} \]

Where, \( \tau_c \) = Critical Shear Stress \( \left( \frac{N}{m^2} \right) \)

\( D_{50} \) = Median Grain Size (mm)

Improvements to this relationship to account for differing soil types was presented by Julien (1995) and later summarized by Fischenich (2001).

\[ \tau_c = 0.5g(\rho_s - \rho_w)d \tan \phi \quad \text{Clays} \]
\[ \tau_c = 0.25d_s^{-0.6}g(\rho_s - \rho_w)d \tan \phi \quad \text{Silts and Sands} \]
\[ \tau_c = 0.06g(\rho_s - \rho_w)d \tan \phi \quad \text{Gravels and Cobbles} \]

Where, \( d_s = d \left[ \frac{(G-1)g}{\nu^2} \right]^{\frac{1}{2}} \)

\( \phi \) = Angle of Repose

\( G \) = Specific Gravity of Particles

\( \nu \) = Kinematic Velocity

\( d \) = Size of the Particle of Interest
Through an adaptation of the original method for additional soil types, the potential of a predictive model for evaluating critical shear stress as a design parameter is substantially increased. Common \textit{in-situ} and laboratory method for determining the critical shear stress use an impinging jet method, as discussed in the following section.

\subsection*{3.1.2 Literature Review on Impinging Jets for Critical Shear Stress}

By definition, a circular impinging jet is “a jet produced from a circular nozzle that is issuing a stationary fluid and is directed to impinge against a boundary or wall” (Mazurek, 2001). The use of jets for the purpose of evaluating the scour and erosion characteristics of soil dates back to 1939 (Rouse, 1939). Complications occur when using impinging jets in cohesive soils as a result of inherent effects of matric suction, organic content, water content, soil fabric, temperature, pore space and fluid chemistry, clay mineralogy, clay content, and soil density (Paaswell, 1973; Hanson and Cook, 1998; Cossette et al., 2012). Several impinging jet methods for determining critical shear stress were compiled (Cossette et al., 2012) and the methods are discussed below.

\textit{Method 1: Hanson and Cook (2004) Method}

Hanson and Cook (2004) proposed a soil erodibility apparatus, testing procedures, and analysis method for evaluating the critical shear stress in-situ. The testing apparatus used an impinging jet inside of a submergence tank with a point gauge aligned with the jet to measure the incremental scour depth. Using this procedure type, the solution methodology for the
determination of the critical shear stress is explained as follows. Figure 3.3 shows a schematic of the testing apparatus tip, induced tractive stress distribution, and the parameter definitions used in the analytical solution. The “diffused jet” refers to the direction of the water after reflecting off of the soil surface, and the “potential core” displays the zone of constant velocity. The induced tractive stress distribution theoretically is zero along the jet centerline (Hanson et al. 1990), and their analysis was presented using the assumption that the peak stress value causes the maximum scour depth (Hanson and Cook, 2004).

Figure 3.3: Submerged Circular Jet with Parameter Definitions and Induced Tractive Stress Distributions [Adapted from: Hanson and Cook, 2004]
As discussed above, the “potential core” is the zone of constant velocity, equal to the magnitude of the velocity at the tip. After the water exits the potential core, the centerline velocity decay is linear with distance from the cone (Albertson et al. 1950).

\[
\tau_0 = C_f \rho U_0^2 \quad \text{(Hanson and Cook, 2004)}
\]

Where,

\[
\tau_0 = \text{Applied Bed Shear Stress} \left( \frac{N}{m^2} \right)
\]

\[
U_0 = \text{Average Jet Velocity} \left( \frac{m}{s} \right)
\]

\[
\rho = \text{Mass Density of Water} \left( \frac{kg}{m^3} \right)
\]

\[
C_f = \text{Local Skin Friction Coefficient} = 0.00416
\]

Applying the physical principles that govern the fluid flow to the soil-erosion problem, as the scour hole depth increases outside of the “potential core,” the erosion rate decreases correspondingly to the centerline decay of the velocity. The centerline velocity decays until the velocity and the soil reach a steady-state condition, the equilibrium depth, as \( J_e \), as shown in Figure 3.3. Once the equilibrium depth is achieved, the stress-state of the soil can be quantified as the critical shear stress. For un-cemented soils, Hanson and Cook (2004) developed an expression for critical shear stress as a function of the applied stress, and the potential core length and the equilibrium scour depth.

\[
\tau_c = \tau_0 \left( \frac{J_p}{J_e} \right)^2 \quad \text{(Hanson and Cook, 2004)}
\]
Where, \( \tau_c \) = Critical Shear Stress
\( \tau_0 \) = Maximum Shear Stress at the Nozzle
\( J_p \) = Potential Core Length
\( J_e \) = Equilibrium Scour Depth

The potential core length is a function of the nozzle diameter and the diffusion constant. Rajaranthnam (1976) evaluated the constant as \( \approx 6.3 \). The expression for the potential core length is shown below.

\[ J_p = C_d d_0 \quad \text{(Hanson and Cook, 2004)} \]

Where, \( C_d \) = the Diffusion Constant \( \approx 6.3 \)
\( d_0 \) = Nozzle Diameter

When the applied shear stress exceeds the critical shear stress, erosion of the soil occurs. The rate at which erosion occurs is a function of the detachment coefficient (erodibility coefficient) and the magnitude of the applied stress relative to the critical shear stress.

\[ \varepsilon_r = k_d (\tau - \tau_c) \quad \text{(Hanson and Cook, 2004)} \]

Where, \( \varepsilon_r \) = Erosion Rate \( \left( \frac{m}{s} \right) \)
\( k_d \) = the Erodibility or Detachment Coefficient \( \left( \frac{m^3}{N - s} \right) = 0.00416 \)
\[ \tau = \text{Applied Shear Stress} \geq \tau_c \]

**Method 2: Equilibrium State Assessment Method**

Hanson and Cook (2004) and later, Mazurek and Gheisi (2009) proposed that in order for a scour hole to reach an equilibrium size, the applied shear stress must be equal to the critical shear stress on the soil. To estimate the equilibrium depth, Blaisdell et al. (1981) presented a curve fitting approach to equilibrium scour hole depth, as shown below.

\[ \tau_c = \sigma_f \rho \left( C_d U_0 \frac{d_o}{H_l + \varepsilon_{cl}} \right)^2 \]

Where, \( \varepsilon_{cl} \) = centerline scour depth relative to the original bed level at equilibrium

\( H_l \) = Height of the Jet above the Soil Surface

**Method 3: Visual Assessment Method**

The visual assessment of the critical shear stress is an operator-dependent method during which the shear stress is calculated at the onset of the first occurrence of erosion (Dunn, 1959). Criticisms of this method are directed at the discretion of the operator associated with the selection of the onset of erosion. During this test, a flow rate less than the flow rate needed to apply the critical shear stress onto the soil is applied and gradually increased until erosion occurs. On the onset of erosion, the shear stress on the soil can be calculated by the following expression.
Where, \( U_{OC} \) = Velocity of the Jet at which Erosion is First Observed

**Method 4: Plot of Jet Test Erosion Rates Method**

The final method for predicting the critical shear stress is to incrementally evaluate the erosion rate for each time interval. This erosion rate is plotted against the average applied shear stress during that time interval. Graphically, the critical shear stress is determined as the value of a regression line through the data across the bed shear stress axis (where erosion rate equals zero).

### 3.2 Impinging Jet Experimental Design

The impinging jet experiment was designed with the objective of understanding and qualitatively assessing the improvement in critical shear stress as cementation level increases. The reported critical shear stresses may be used as a reference, in general, but a designer should use discretion when recommending design values due to differences in cementation patterns and distributions. Based on the methodology used to analyze the results of the In Situ Erosion Evaluation Probe (ISEEP) that was introduced by Gabr et al. (2012), during which an impinging jet is used to evaluate an erosion profile with depth. Contrastingly, the experimental design in the current study has a stationary impinging jet mounted at a specified height above...
the soil. Using a pumping system, the critical shear stress of the cemented soil is assessed using water flow perpendicular to the soil surface and the results are compared with uncemented soil.

Based on unsaturated freshwater-based treatments, as discussed in Chapter 2, four soil specimens were dry pluviated with North Carolinian dune sand (gradation properties in Table 2.1) into acrylic cells with 7.5” inner diameter and 16” height. Each specimen was 10” in height and had a Porex filter directly above and below the sample. The bottom cap of the cell was an acrylic disk with ¼” holes at 1” centers, drilled to allow the columns to have a hydraulic gradient of zero (free-draining). To prevent treatment-induced erosion of the samples, 6” of pea gravel was placed overtop the Porex filter.

Figure 3.4: Experimental Configuration used for Treating Sand Columns to Assess Critical Shear Stress
A 4” filter layer of Ottawa 20-30 was used to prevent soil from leaving the columns, based on a filter design in accordance to Lambe and Whitman (1969). The soil specimens in their treating configuration are shown in Figure 3.4. As a result of the weight of the sand in the column, it was decided to not design bottom caps with plumbing, but instead to build a wooden stand that would not prohibit any flow through the column. This wooden stand was placed inside of a large neoprene tub and the samples were pluviated while on the stand in the tub. The treatment effluent was pumped out when needed.

To specify the flow rate, a variable speed centrifugal pump (Gould GLSSV2), Figure 3.6(a), was paired with an ITT PumpSmart variable speed controller, Figure 3.6(b). The high-capacity pump had its flow rate mechanically reduced using two four-hose splitters as shown in Figure 3.6(c). To correlate the pump speed (rpm) to velocity, a calibration curve was created, as shown in Figure 3.5, with the pump, controller, and 8-hose system. The desired lower range of the calibration curve were preselected based on a back-calculation of the required flow rates needed to induce the critical shear stress. The desired flow rate was mechanically reduced until the velocity produced was lower than the velocity needed to induce the critical shear stress as correlated with median grain size by Briaud et al. (2001). Water was stored in a 200 gallon basin and pumped out into the impinging jet, the remaining seven hoses were pumped back into the basin shown in Figure 3.6(d).

As a function of cone tip diameter, the pumping system has the capacity to provide a 22.8 gpm flow rate and a velocity of 4.4m/s (Kebede et al. 2014). During the creation of the calibration curve, the experimental configuration was identical to the configuration used during all testing. A five gallon bucket was placed beneath the cone tip and the time to fill the bucket was recorded, then used to find the average flow rate. Using this average flow rate, the
dimensions of the cone tip, shown in Figure 3.7(b), and algebraic manipulation of expression below, the average tip exit velocity was determined.

As displayed in Figure 3.7(b), the stainless-steel cone has a cross-sectional tip diameter of 0.75in (19mm) and a 60° truncated cone-tip. The cone tip is interchangeable and threaded onto a ¾” diameter steel pipe. The diameter of the pipe, hose, and cone tip is constant to prevent non-uniformities in the flow rate caused by turbulence within the system. The cone tip is suspended above the soil using a U-bolt onto a wooden frame Figure 3.6(e), which allows for a change in the height of the cone tip.

![Figure 3.5: Calibration Curve for Pumping Systems](image)

Post-treatment and prior to performing the impinging jet experiments, 1” above the soil surface, ¼” holes were drilled into the soil column walls at 1” O.C. to maintain a constant water height during the scour testing. The holes in the acrylic allow the water being supplied from the impinging jet to drain out of the acrylic specimen walls and once the system reaches
equilibrium at a particular flow velocity, the water height remains constant, and ranged from 1” to 2” during a majority of the testing. This can be seen in the experimental configuration shown in Figure 3.6(f). Three pumps were used in order circulate the water used in the impinging jet back to the water storage basin during the testing.

Figure 3.6: Pumping System and Components, a) Gould Pump, b) PumpSmart PS200, c) 8-Hose Split Used, d) 200 Gallon Water Storage Basin
3.3 Materials and Methods

The sample’s void ratios, treatments, and treatment concentrations are shown in Table 3.1. The treatment volume was designed using a treatment fraction of 0.33, meaning that each treatment is nearly one-third of the pore volume, which was determined to be approximately 1500mL per treatment. The treatment concentrations used in this segment of the study are designed based on a highly cemented model-scale soil box used by Shanahan and Montoya (2014). It should be noted, however, that the chemical concentrations used in this portion of the testing were substantially higher than the ecologically optimized saltwater-based
concentrations discussed in Chapter 2 of this thesis. To expedite the cementation process and with effluent chemistry being insignificant in laboratory conditions, the higher concentrations were deemed acceptable. The number of treatments was designed such that, column 1 has half as many flushes as column 2, and column 2 has half as many flushes as column 3. It was though that this treatment schedule would produce a wide spectrum of MICP treated sand, allowing the scour trends to be analyzed over an array of cementation values.

Table 3.1: Impinging Jet Specimen Properties and Treatment

<table>
<thead>
<tr>
<th>Column</th>
<th>Void Ratio</th>
<th>No. of Flushes</th>
<th>Urea (M)</th>
<th>CaCl₂ (M)</th>
<th>ATCC 11859 (mL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.70</td>
<td>15</td>
<td>0.333</td>
<td>0.100</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>0.71</td>
<td>30</td>
<td>0.333</td>
<td>0.333</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>0.70</td>
<td>60</td>
<td>0.333</td>
<td>0.333</td>
<td>30</td>
</tr>
<tr>
<td>Untreated</td>
<td>Target: 0.7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The critical shear stress was evaluated using incremental velocities and their corresponding equilibrium scour depths were calculated based on the curve fitting method presented originally by Blaisdell et al (1981) and discussed again by Hanson and Cook (2004). Typical impinging jet tests occur at a single velocity, however, the nature of this study led to the requirement of conducting and compiling multiple tests into one. To determine the critical
shear stress and to understand cementation-scour trends, the testing protocol presented herein was based on Hanson and Cook (2004) type tests for each velocity. These tests were used to compute the applied shear stress, critical shear stress, erosion rate, and detachment coefficient. Figure 3.8 shows the holes drilled in the specimen walls and the impinging jet during testing.

If a particular velocity did not induce mass erosion after fifteen minutes, the flow was stopped and the velocity was increased using the pump controls and calibration curve. This new velocity was then applied to the surface and the onset of erosion and erosion rate was monitored using the equilibrium scour depth. From the erosion rate, the change in detachment coefficient was calculated for each test and the critical shear stress evaluation component of the testing is completed. Post-testing, the average cementation level within the scour zone of interest was determined using a gravimetric acid washing method. Discrepancies occur when these results are compared to other experimentally obtained critical shear stress parameters. The results from this study violate the typical testing methods, which are conducted in free-field. To allow for a more efficient MICP treatment regime, the treatment and impinging jet testing was performed in a 7.5” inner diameter acrylic tube. However, this most likely has effects on the reported critical stresses. As a direct result of the fluid-structure interaction, the induced hydrodynamic forces along the acrylic surface are larger than they would be if the testing condition was a free-field. Conservatism is built into the experimental configuration for evaluating the critical shear stress.
3.4 Results and Discussion

After completion of the four impinging jet tests, it was not possible using the available pump and cone tip to induce scour on any of the cemented. To counteract the inability to obtain the critical velocity, the hose splitters were removed and the water exiting the pump was directed with full intensity into the impinging jet. Removing the hose-splitters and using only one exit hose lead an observed exit velocity of 4.83 $\frac{m}{s}$. The three cemented specimens were subjected to increasing velocities to determine when the critical shear stress was reached. With the one hose configuration, this was not possible, the maximum tip velocity achievable with
the pumping system and cone tip, was applied to the surface for a fifteen minute interval without any scour occurring.

It was shown in the laboratory that velocities below the minimum velocity of the calibration curve, 0.6 gpm as shown in Figure 3.5, induced scour in the un-cemented sand. Although the preliminary un-cemented scour test was conducted in different conditions, a five gallon bucket, it was shown that the spheriecal scour zone induced by the jet nozzle had a smaller diameter than the acrylic that confines the other tests. As the scour hole geometry did not impede on the projected area of the acrylic column, the test was considered to be comparable to the cemented tests. The comparable assumption will remain until the actual test can be completed after the experimental configuration is altered to create an increased applied stress (smaller cross-sectional flow of water out of the nozzle and a reduced distance from the tip to the original soil surface.

The velocity that corresponds to the critical shear stress increases 1-2 orders of magnitude between the un-cemented and the lightly cemented soil. This effect is magnified during the computation of the applied shear stress, as it is a function of the square of the velocity. Due to the inability to determine the critical shear stress, Figure 3.8 shows the minimum possible critical shear stresses, the applied shear stress is plotted as a minimum value (no scour at the applied shear stress refers to a critical shear stress that is larger than the applied shear stress). The critical shear stresses shown in Figure 3.8 were calculated as the applied shear stress using the Hanson and Cook (2004) method.
The critical shear stress as a function of mass of calcite plot shown in Figure 3.8 is relatively inconclusive in terms of understanding the scour behavior quantitatively. Based on preliminary testing, the critical shear stress of the un-cemented soil (M.O.C. = 0), was determined to be 0.2Pa. However, it can be shown that the critical shear stress increases by a minimum factor of 500, using the un-cemented critical shear stress as determined experimentally, and the applied shear stress (which is likely substantially lower than the actual critical shear stress). To confirm the actual improvement from the cementation, the un-cemented scour test will be conducted after the cemented tests in one of the acrylic columns used for the treatment of the cemented columns, to be representative of the other boundary conditions. The soil surfaces before and after the impinging jet tests are shown in Figure 3.9 below.
Through a critical shear stress improvement by a factor greater than 500, the susceptibility of sandy soils to scour is reduced significantly. The scour mechanism shifts from that of clean sand towards the mechanism for cohesive, fine-grained soils. Instead of scouring particle by particle like sands predominantly do, mass erosion occurs as the brittle cementation bonds break at the critical shear stress of the soil, reached when the magnitude of the cohesive strength is applied by the impinging jet.
3.5 Conclusions and Future Work

Based on the qualitative assessment of the increase in critical shear stress as a function of a cementation level, it was found that the scour susceptibility decreases drastically. From this observation, it was concluded that relatively low amounts of cementation have large repercussions on the reduction in the scour behavior of sands and the primary erosion mechanism switches from particle by particle erosion to mass erosion. The bond strength between sand grains is likely the critical parameter in assessing the critical shear stress that causes particle detachment, as the bond strengths are the weakest component of the cemented soil matrix.

Future work includes evaluating the scour behavior in the free-field at larger applied stresses and numerically modeling the scour mechanism under wave action and other scouring phenomena. Furthering the quantitative understanding of the behavior of cemented sands during scouring has the potential to provide a cost effective alternative to hard-barrier scour control structures and to reduce cost in the design of bridge and other scour-susceptible foundation elements by reducing the design scour depth and effectively increasing the soil resistance along the element. Through the application of MICP to soils surrounding structures near flowing water, MICP may provide scour resistance through in-situ soil improvement and may prevent the necessity for the installation of rigid structures to dampen the erosive effects from flowing water. The development of predictive methodologies for estimating the critical shear stress is imperative for MICP based scour design.
4 MICP TREATED DUNE PERFORMANCE UNDER WAVE ACTION

4.1 Background

Sand dunes are geological necessities for the fortification of coastal communities from wind and wave induced erosion. As the climate changes, there have been documented increases in storm intensity and frequency, which have had drastic effects on coastal geomorphology, in particular, erosion induced changes in elevation (Miller, 2015). Acting as the first natural defense mechanism, coastal sand dunes dampen the erosive effects of wave action and protect the inland infrastructure. Large storm events can lead to wave-induced overwash, which may result in complete dune destruction if storm conditions persist (Figlus et al. 2011). Erosion of the sandy coastal soils can have devastating impacts on coastal lifelines such as highways, pipe lines, and other utilities, as shown in Figure 4.1. Damages from coastal erosion may lead to extensive property damage, loss of revenue, and large repair costs. Preventing the erosion of coastal sand dunes can improve the resiliency of these coastal systems, minimizing damages and repair costs after storms. During Hurricane Sandy, coastal infrastructure that was protected by sand dunes had substantially less damage than infrastructure that was not protected by sand dunes (GEER, 2013).
Natural, coastal sand dunes are functional ecosystems, supporting life and providing infrastructure protection. In order to reduce erosion of sand dunes and to continue to provide life to a multitude of flora and fauna, improvement methods must be sustainable and not lead to drastic changes in microbial activity, oxygen availability, pore space bio-geochemistry, and leach pollutants into the groundwater. To properly assess the feasibility of MICP on coastal sand dunes, the induced ecology and the interaction of the induced ecology on the indigenous bacteria, flora, and fauna should be understood.

Figure 4.1: Storm Surge Erosion of Highway 12 and Surrounding Infrastructure in Rodanthe, N.C. after Hurricane Irene [Adopted from: Beach Recovery Foundation]
4.2 Literature Review

Coastal sand dunes and other active and vegetation-stabilized Aeolian sand deposits cover nearly 6% of the globe (Pye and Tsoar, 2009, Koenig, 2012). Vegetation is one of the primary forms of coastal sand dune stabilization (Duran & Moore, 2013), and is one of the prominent components of the creation of coastal sand dunes. In contrast to dry desert sand dunes, coastal sand dunes require the interaction of physical (wind-transport) and biological (vegetation) processes (Haim & Pye, 2009). Typical temporary sand dune stabilization methods include: sand shielding, chemical stabilizers, biological crusting, and sand fences (Shehata, & Al-Rehaili, 2005). Permanent dune stabilizations to date are generally vegetation based, especially on the North Carolina coast (O’Connell, 2008). A mechanics-based understanding of the mechanisms governing the formation and subsequent erosive behavior of coastal sand dunes are imperative for the design of resilient infrastructure.

Predictive coastal sand dune erosion models assist coastal engineers in evaluating existing sand dune configurations based on probabilistic design storms, and the potential impacts on the dunes and the nearby infrastructure. The premise of the study presented herein is to evaluate reduction in erosion as a result of the addition of cementation into the soil matrix and between particles at the particle contacts. Dune erosion is heavily dependent upon the soil gradation and dune density (Overton et al., 1994), indicating that sand dune systems with different particle gradations will experience unique erosion patterns during identical loading conditions. Several analytical solutions are available for predicting dune erosion (Bruun, 1962; Edelman, 1972, Komar et al., 1999), one of the most prominent methods was presented by Larson et al. (2004). Based on wave impact theory, the transport relationship is used in
conjunction with sediment volume conservation, resulting in the following governing equation for the average rate of dune erosion.

\[
q_D = \frac{dV}{dt} = -\frac{1}{2} C_E \rho \frac{u_0^4}{\rho_s g^2 T (1 - p)}
\]

Where:

- \( q_D \) = average rate of dune erosion \( \left( \frac{m^3}{m^2s} \right) \)
- \( V \) = eroded volume \( \left( \frac{m^3}{m} \right) \)
- \( t \) = time \( (s) \)
- \( C_E, C_u \) = empirical coefficients
- \( \rho \) = water density \( \left( \frac{kg}{m^3} \right) \)
- \( \rho_s \) = sediment particle density \( \left( \frac{kg}{m^3} \right) \)
- \( u_0 \) = speed of bore \( \left( \frac{m}{s} \right) \)
- \( g \) = acceleration of gravity \( \left( \frac{m}{s^2} \right) \)
- \( T \) = wave period density \( (s) \)
- \( p \) = porosity

The above governing equation can be conveniently programmed into a numerical modeling program. An analytical solution was developed based on several simplifying assumptions for periodic wave loading. Through relevance to the experimental testing in this
chapter, the analytical solution for eroded volume at a specific time for a sand dune experiencing periodic waves can be defined as follows (Larson et al., 2004).

\[
\Delta V_E = 8 \frac{C_s}{T} \left[ \frac{R_T^2}{2n + 1} \left( \frac{T_s}{2} - \left( \frac{2t_L}{T_s} \right)^{2n} t_L \right) - 2 \frac{R_T z_D}{n + 1} \left( \frac{T_s}{2} - \left( \frac{2t_L}{T_s} \right)^n t_L \right) + z_D^2 \left( \frac{T_s}{2} - t_L \right) \right]
\]

Where:

- \( \Delta V_E \) = eroded volume at a specific time \( \left( \frac{m^3}{m} \right) \)
- \( C_s \) = empirical coefficient
- \( R_T = R_a + z_a \) (m)
- \( R_a = \) runup height amplitude during surge (m)
- \( z_a = \) amplitude of water level variation during surge (m)
- \( T_s = \) duration of surge (s)
- \( t_L = \) time when bores start impacting the dune during surge
- \( z_D = z_i - R_i \) (m)
- \( z_i = \) initial elevation of dune foot (m)
- \( R_i = \) runup height at \( t = 0 \) (m)

### 4.3 Wave Tank Experimental Design

**Overview**

To evaluate the mechanical improvement of the North Carolinian dune sand during wave action, a series of wave tank tests were conducted at varying levels of cementation. The
sand used for all of the wave tank testing, North Carolinian dune sand, was collected from a sand dune near Jeanette’s Pier in Nags Head, North Carolina, at the location shown in Figure 4.2. Before sample pluviation, the North Carolinian dune sand was oven dried to obtain a more accurate initial void ratio calculation. After completion of wave tank testing, eighteen samples were obtained from each specimen to create a cementation distribution using the post-gravimetric acid washing procedure discussed in Chapter 2. From the average cementation level at the most critically eroded section, relative trends between eroded area and cementation level are discussed in the results section. A total of five wave tank tests were conducted, with void ratios and treatment constituents displayed in Table 4.1. In order to facilitate a rapid cementation rate and since the induced ecology is not an issue when operating in laboratory conditions, a treatment fraction of 0.50 was used. Using this treatment fraction, which refers to the volume of the treatment media normalized by the volume of voids per treatment, the volume per treatment and the design specimen volume was optimized and a treatment volume of 1.5L was used for a specimen size of 12” x 12” x 3” at a void ratio of 0.7.
The specimen treatment numbers shown in Table 4.1 were designed at 15 flush intervals to display trends at high cementation levels while still allowing for some understanding of the behavior of lightly cemented sands to wave action. It should be noted however, that the specimens were treated identically as the specimens in Chapter 3 and are to be used as a method of connecting the critical shear stress from Chapter 3 with the eroded area presented here.
Table 4.1: Specimen Void Ratio and Treatment Schedule

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Void Ratio</th>
<th>No. of Flushes</th>
<th>Urea (M)</th>
<th>CaCl₂ (M)</th>
<th>ATCC 11859 (mL/Flush)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated</td>
<td>0.70</td>
<td>0</td>
<td>0.333</td>
<td>0.1</td>
<td>30</td>
</tr>
<tr>
<td>1</td>
<td>0.70</td>
<td>15</td>
<td>0.333</td>
<td>0.1</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>0.70</td>
<td>30</td>
<td>0.333</td>
<td>0.1</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>0.69</td>
<td>45</td>
<td>0.333</td>
<td>0.1</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>0.71</td>
<td>60</td>
<td>0.333</td>
<td>0.1</td>
<td>30</td>
</tr>
</tbody>
</table>

*Sand Boxes*

The sand specimens were created using a composite system, consisting of PVC sheeting and an elastic membrane/dowel support system. The ¼” thick, 12” x 12” PVC sheet had ¼” holes drilled at one inch centers to create a free-draining system. Spanning half of the height of the soil, a 1.5” high PVC sheet was connected around the perimeter of the bottom sheet using PVC Purple Primer and PVC Cement. A Porex filter was cut and placed into the PVC structure and caulked onto the PVC sheets to prevent leaking during testing. To reduce sample disturbance, which in the case of cemented soils leads to the breakage of non-recoverable cementation bonds, a membrane support system was used for the upper 1.5” of soil height. To stabilize against the tensional effects of the membrane, wooden dowels were inserted into holes that had been pre-drilled into the PVC/Porex. After inserting and caulkign
the dowels in to place for waterproofing, the membrane was stapled to the dowels to allow for easy removal once inside the sand dune model. To reduce treatment-induced erosion, a Porex filter was placed on top of the 3” of sand, followed by 2” of pea gravel. Photos of the sand dune boxes and their stand with effluent storage are shown in Figure 4.3a and 4.4b.

Figure 4.3: Sand Boxes used During Treatment and Inserted into Sand Dune Models

**Sand Dune Models**

After treating the sand specimens to the desired number of treatments, they were inserted into the sand dune model. The sand dune model is a wooden enclosure, created to display a 15° surface for the sand box to be placed into. To account for wave-action based
boundary conditions, the 12” x 12” sand box was then put into the 24” x 24” sand dune model, leaving 6” of untreated soil on all four sides of the sand box. The sand box is created using 2” x 4” wooden framing with ½” plywood on the exterior. The connections between plywood sheets were caulked using silica-based caulking to create watertight seals. During the first trial of the submerged stand for the sand dune models, the legs and the rest of the support system were constructed with wood. After attempting to place the stand into the wave tank, buoyancy lead to the entire structure floating, leading to a redesign using denser materials. A metal cart was repurposed to act as a stand, and four 50 pound steel grates were bolted to the cart to effectively eliminate rocking during application of the design wave loading. The sand dune model was bolted onto the cart. The sand dune model connected to the repurposed cart, with the metal grates are shown in Figure 4.4.

Figure 4.4: Sand Dune Model and Repurposed Cart Stand
Laser Morphology

To quantitatively assess the wave action induced erosion, the soil surface was scanned using 2D laser morphology. The laser used was a scanCONTROL 2600-25 line laser scanner by Micro-Epsilon. The line laser was mounted at a constant height with a 15 degree angle to match the angularity of the design soil surface. This scanner is capable of taking 640 data points per profile, attributing to high precision in the generated morphology surfaces. Laser readings were taken at a 2” spacing beginning and ending at the edges of the 12”x12” soil specimen, for a total of seven readings per sample. Due to limitations in the allowable length in order to maintain precision, at each of the seven locations, the upper portion and the lower portion of the specimen were measured separately and connected using a constant height point. Figure 4.5a displays the laser mounting system before a measurement was taken and Figure 4.5b shows the laser scanning a line segment on the centerline of a specimen.

Figure 4.5: Micro-Epsilon Line Laser a) Being Mounted to Fram, b) While Scanning
4.4 Materials and Methods

Wave Tank

The wave tank testing was performed at the Coastal Studies Institute in Wanchese, N.C. using an Omey Labs wave tank, as shown in Figure 4.6. The wave tank has dimensions of 33’ x 9.5’ x 5’ (L x W x D), with an optimal operating water height of 4’4”. To physically generate the waves, the wave maker uses precise movements of an electrical linear planetary gear screw actuator. Used to counteract the hydrostatic water pressure on the wave maker by the water in the tank, a pneumatic air spring was used for pressure equalization. The capabilities of the wave generator include monochromatic waves, seismic waves, several different design spectra, and irregular waves, the testing conducted in this study used exclusively monochromatic (periodic) waves. The waves used during the testing presented herein are similar to that of Figure 4.6 in both wave amplitude and period.

Figure 4.6: Omey Wave Tank (Source: www.omeylabs.com)
The desired wave properties were programmed into the wave generator and once the software is ran, the wave maker begins ramping up. The wave generator makes smaller movements at the beginning of the operation and the lateral displacements of the actuator increase until the desired wave properties are obtained. Once the software is triggered to stop producing waves, the wave generator acts conversely, ramping down by successively reducing lateral displacements until the actuator comes to a complete halt.

On the opposite end of the wave tank, to reduce (or minimize) the effects of wave reflection and to dissipate the wave energy, Omey Labs designed a wave absorber. This wave absorber removes energy from breaking waves through flexing of polycarbonate sheeting. To dampen the breaking wave energy, the water entrained beneath the sheeting creates an equal matching force for absorption of the breaking wave. The wave generator is shown on the left end of the wave tank in Figure 4.6, while the wave absorber is the black parabolic structure inside the right end of the wave tank.

Wave Gauges

For comparison between wave tests, it had been determined that relying on input signals from the software for the generated waves would not provide the accuracy needed to quantitatively compare the actual waves that act on the soil surface. Two USS10 acoustic wave gauges made by General Acoustics were used. To monitor the wave properties as they propagate down the tank and break on to the sand surface, the software UltraLab by General Acoustics was used for general data acquisition. The signal receiver, UltraLab ULS HF416 was used in conjunction with the wave gauge sensors. The signal receiver operates through a series of ultrasound pulse echoes, while the wave gauge sensors measure the echoes and level.
An indication of signal quality can be obtained by the number of received echoes and the level shown is an averaged water level, determined every ten seconds over one minute.

The first wave gauge was placed midway between the actuator and the soil specimen, while the other was placed above the center point of the soil specimen. Since the wave tank was designed to have low energy damping as the waves propagate (relatively frictionless glass panels are used at the interface between the water and the wave tank framing), the profile obtained by the first wave gauge is used as the wave input function. The second wave gauge is placed at the horizontal and vertical midpoint of the sand specimen. During wave action, the second wave gauge was used to record the waves as they act on the surface of the specimen, and as a second measure of assessing the sediment transport during wave drawdown.

**Sand Dune Model Pluviation**

To prepare the sand dune model, the bottom section of the wooden model was dry-pluviated prior to the insertion of the sand box. Next, the sand boxes were placed into the sand dune model and the sand box was lifted and more sand was pluviated under the box until the angularity of the sand box matched that of the final surface (15°). The space in between the sides of the sand dune model and the sand boxes were pluviated to the target void ratio and the membrane holding the sand box was cut and removed, creating an interface between the cemented soil and the pluviated untreated soil. Having the pluviated soil outside of the sand box acting on the sand box allowed for minimal sample disturbance for the lightly cemented boxes that may not have had the strength to hold a vertical face. After cutting and removing the membrane, the wooden dowels were cut at the surface and the initial laser morphology surfaces were obtained.
4.5 Results and Discussion

Untreated Wave Test

The untreated wave test was used as a baseline for comparison between North Carolinian dune sand at several degrees of cementation. Using the experimental components described above, the sand dune model was pluviated completely to a target void ratio of 0.7. One of the wave gauges was mounted to the sand dune model at the horizontal and vertical center. Figure 4.7a shows the input wave function, which had a design wave height of 0.1m (~4 inches) for approximately 1 minute. The ramp up and ramp down periods are shown as the wave height increases to the design wave height (ramp up) and correspondingly decreases back to zero (ramp down). Prior to beginning the test, the actuator was out of place, resulting in an abrupt movement and the cause of the scatter in the input function. When evaluating the effect on the soil specimen, it was shown in Figure 4.7b that the smaller waves created by this abrupt movement, had no effect on the specimen. It is also shown at the tail end of the input function in Figures 4.7a that the presence of the sand dune model frame in the wave tank had some interference with the efficiency of the wave absorber. This phenomenon is evident by the constructive and destructive interference shown during the ramp down period and into the settling period of the wave tank. Figure 4.7b shows the soil surface wave profile, which is measured from the center of the soil specimen. The figure shows that after approximately 1.5 minutes the soil surface begins decreasing in height as the wave height in between wave peaks trends in the negative direction and the equilibrium height changes.
Shown in Figure 4.8a is the surface prior to the application of wave loading and Figure 4.8b shows the eroded surface after wave action. As shown, the eroded volume is substantial under the design loading for the untreated sand. The large erosion produced in the laboratory is analogous to the susceptibility of coastal infrastructure during storm events that induce wave
action on sand dunes. Wave loading was applied for minutes in the laboratory, while most storm events last extensively longer.

Figure 4.8: Soil Surfaces Before and After Wave Loading - Uncemented, a) Initial, b) Final

After completion of laser morphology scanning, the profile about the centerline of the sample was obtained and the specimen area before and after wave action was calculated using the numerical integration tool, Trapezoidal Rule. Due to limitations in measuring 3D morphology directly, 2D data was obtained and instead of reporting eroded volumes, the most critical section was determined (the section where the largest magnitude of erosion occurred) and the corresponding eroded area (about that cross-section) was reported. The centerline profiles of the untreated specimen before and after the application of wave loading are shown in Figure 4.9.
As shown by the eroded surface in Figure 4.9 and the final surface in Figure 4.8b, the eroded area was extensive. The area differential between the initial and final states was calculated to be 3.0 in\(^2\). Extrapolating this value to the length of the specimen, calculated by assuming that the critically eroded surface extends the full length of the specimen (12 inches), the eroded volume becomes 36.0 in\(^3\). Extrapolating again, assuming that the critically eroded surface extends for 1 mile, as most coastal sand dune systems extend for tens to hundreds of miles, under the design loading, the rate of erosion is 4.1 yd\(^3\)/min. Considering that most severe storms occur for extended periods of time, unprotected sand dunes have the potential to be damaged excessively.
15 Flushes Wave Test

The 15 flush wave test was conducted identically to the previously discussed untreated test, with the input function shown in Figure 4.10a. Although the input functions vary slightly, through a comparison of the input function for the untreated wave test shown in Figure 4.7a to the input function Figure 4.10a for the 15 flushes wave test, it was determined that the differences were negligible. After making this assumption, comparisons of eroded areas can be made directly and extrapolated to in-situ expected volumes. Figure 4.10b displays the soil surface wave profile. Unlike the untreated sand model, the variation in equilibrium is minor. Shown by the height of the peaks in the soil surface waves, the depth of water at the center of the model can be shown. Increased peak heights refer to an increased water depth above the datum (the original soil surface). As the water retreats, with no erosion the sensor would read the datum value. However, when erosion occurs, the measured height becomes negative (below the original datum). It was shown between comparisons of Figure 4.7b to Figure 4.10b that erosion was reduced at the center, which was shown with laser morphology as well (Figures 4.9 and Figure 4.12).
The initial and final soil surfaces for the 15 flushes wave test are shown in Figure 4.11a and 4.11b, respectively. It was shown that after 15 flushes, the amount of erosion was reduced significantly, as the soil from the cemented sand box can be seen clearly, at a height greater than the surrounding uncemented soil. The wooden dowels can be seen on the sides of the
specimen as it was decided to leave the dowels in to prevent breakage of the brittle calcite bonds. Similarly, some of the pea gravel was cemented to the soil surface, as shown on the front of the specimen in Figure 4.11b, and in order to reduce sample disturbance, these gravel particles were left in place. Figure 4.11b shows a non-uniform erosion pattern, occurring primarily in the center of the specimen.

Figure 4.11: Soil Surfaces Before and After Wave Loading – 15 Flushes, a) Initial, b) Final

The flexible properties of the membrane lead to membrane elongation which created an uneven and concave down soil surface. As the membrane elongated, the soil displaced laterally and preferential flow occurred on the outer edges of the soil specimen. With an increased void ratio, the permeability increased and the nutrient flow thereby increased. The increased nutrient flow resulted in a larger treatment fraction and consequently, higher
cementation levels in these areas. This trend occurs throughout all of the wave tank test specimens, as the experimental configuration was constant for each specimen/test.

Using laser morphology on the soil surface before and after wave action, the surface profiles are shown in Figure 4.12. Numerical integration using the trapezoidal rule indicated an eroded area about the centerline of 2.1m². The addition of the cementation from the 15 flushes resulted in a 30% erosion reduction at the most critical section. Extrapolating this value to the length of the specimen, calculated by assuming that the critically eroded surface extends the full length of the specimen (12 inches), the eroded volume becomes 25.2in³. Extrapolating again, assuming that the critically eroded surface extends for 1 mile, most coastal sand dune systems extend for tens to hundreds, under the design loading, the rate of erosion is 2.9 yd³/min. Through light levels of cementation, the erosive resistance was improved dramatically. It is speculated that this increase in stiffness will most likely not be systematically destructive for the ecosystem, as burrowing would still be possible.
In order to complete an acid washing procedure for determining the mass of calcite, eighteen samples were taken from the sand box to assess the distribution. Nine samples were obtained from the surface and nine additional samples were gathered from different depths in the model. The generated surface and depth contour about the center, (in the direction of wave action), which was determined to be the erosion critical section, is shown in Figure 4.13. The cementation generally increased as the distance from the center of the specimen increased. The cementation decreased with depths, but the decrease was relatively negligible for the 3 inch depth used during this testing. The center, which is used as the average cementation level for the critical section, because it was the zone where the majority of the erosion occurred, was measured to be 1.2%. The gradient increased to 5.7% around the edges of the specimen.
30 Flashes Wave Test

Following the same procedures as the previous wave tests, the input and soil surface wave profiles are shown in Figure 4.14a and 4.14b, respectively. Unlike the previous input functions, the wave height in Figure 4.14a reduced more rapidly after the peak wave. Although this phenomenon occurred and likely imparted a comparatively reduced applied wave energy, the function was assumed to be identical to the previous functions for the sake of erosion comparisons. Shown in Figure 4.10b is the soil surface wave profile, during which the waves that caused in increase in water height above the center point began around 0.65 minutes into the testing and was concluded at time 2.15 minutes. Before and after these points, there was
no wave action onto the soil surface at the center during the wave loading. Unlike the notable
elevation change shown in Figure 4.7a, the elevation change in Figure 4.14a was much less
significant (the wave gauge was mounted perpendicular to the ground, not the soil surface,
making the observed data the vertical distance, rather than the distance from the normal to the
soil surface).

Figure 4.14: Wave Gauge Data - 30 Flushes, a) Input Function and b) Soil Surface Wave
Profile
The soil surface before and after wave action for the 30 flush wave test are shown in Figure 4.15. Following a similar cementation pattern, but at higher cementation levels than the 15 flush test, the soil box perimeter maintained its integrity for the most part, as shown by the differential in soil height on the final soil surface. The height differential shows a side-by-side comparison of the effect of the bio-cementation on the erosion behavior (with the caveat that, the vertical wooden dowels could facilitate some structure support, although the initially non-cohesive sand would have had no increased effect without any confinement). Shown in Figure 4.15a are the tips of the exposed wooden dowels that had been cut at the surface, and the wet appearing portion of the model is in the box that was placed into the pluviated sand dune model. Figure 4.15b shows pea gravel that had cemented to the sand that was decided to remain in place during testing to reduce sample disturbance. Also shown on either side of the wooden dowel perpendicular to the direction of wave action, the shape of the soil box is arch-like as a result of the membrane elongation, which led to the observed and unexpected cementation distribution, as shown by the final surface.

Figure 4.15: Soil Surfaces Before and After Wave Loading – 30 Flushes, a) Initial, b) Final
The erosion occurred, for the most part, primarily in the center of the sand box, as this was the location with the lowest strength, i.e. critical shear stress. Post wave action, the specimen was dissected and eighteen samples were obtained to be acid washed for mass of calcite (cementation level) testing. The distribution of cementation along the surface of the sand box changed dramatically as distance from the centerline increased. A contour map of the cementation distribution is shown in Figure 4.16. Following the previous model, the cementation was greater along the edges and decreased towards the center, which is in accordance to the observed erosion surface. Varying from approximately 7.3% on the perimeter to less than 4% in the center, the erosive behavior of the specimen was governed by the strength of the most lightly cemented zone.

Figure 4.16: Cementation Distribution – 30 Flushes, a) Plan View of Soil Surface, b) Profile View along Centerline
Prior to sample dissection, the soil surface was scanned using laser morphology. The initial and final soil surfaces at the centerline are shown in Figure 4.17, with an eroded area calculated with trapezoidal rule of 1.4in². By the addition of the cementation, the erosion reduction was 53% when compared to the untreated baseline test. Extrapolating this value to the length of the specimen, calculated by assuming that the critically eroded surface extends the full length of the specimen (12 inches), the eroded volume becomes 16.8in³. Extrapolating again, assuming that the critically eroded surface extends for 1 mile, as most coastal sand dune systems extend for tens to hundreds of miles, under the design loading, the rate of erosion is 1.9 yd³/min. The 53% improvement is significant, but this suggests that the additional 15 flushes accounted for the added 23% from the 15 flush test, where the first 15 flushes provided a 30% erosion reduction.

Figure 4.17: Laser Morphology Obtained Soil Surface Before and After Wave Action – 30 Flashes
45 Flushes Wave Test

The third cemented specimen, 45 flushes, was tested with the input function in Figure 4.18a, which resulted in the soil surface wave profile in Figure 4.18b. Although there are some discrepancies when comparing to the untreated baseline, the input function is nearly identical to that of the 30 flushes test. The discrepancies are noted in the post-peak section of the input function, which shows wave height that decreases more rapidly than the untreated test. Consequently, the applied wave energy was most likely lower during the 45 flush test, but this reduced wave energy is most likely relatively negligible for comparison purposes and is thereby ignored during the reporting of eroded areas and the projected in-situ volumes. The soil surface wave profile in Figure 4.18a shows that the decrease in surface elevation is not significant and is relatively constant throughout the testing, as suggested by the initial and final soil surfaces shown in Figure 4.19a and 4.19b. Similarly to the final surface for the 15 and 30 flush tests, there is pea-gravel cemented to the front surface of the sand box, which was left on the surface to reduce the likelihood of increased sample disturbance. Visually, the eroded volume is much less than that of the previous two tests and the eroded area in the plan view decreased as well. As shown in the previous wave tests, the eroded area was most significant in the center of the specimen with decreasing erosion as distance from the center increased. Analogously, the cementation distribution matches the surficial erosion pattern, with the lightly cemented areas experiencing a larger erosional loss than the more heavily cemented areas. The range of cementation ranges from 9.8% on the parameter of the specimen to 5.7% in the center most point. The depth profile at the centerline (the most critical section) is displayed in Figure 4.20b, which follows closely with the expected results based on the membrane elongation based preferential flow discussed previously.
Figure 4.18: Wave Gauge Data - 45 Flushes, a) Input Function and b) Soil Surface Wave Profile
Figure 4.19: Soil Surfaces Before and After Wave Loading – 45 Flushes, a) Initial, b) Final

Figure 4.20: Cementation Distribution – 45 Flushes, a) Plan View of Soil Surface, b) Profile View along Centerline
The initial and final soil surfaces at the centerline are shown in Figure 4.21, with an eroded area calculated with trapezoidal rule of 0.8in². By the addition of the cementation, the erosion reduction was 73% when compared to the untreated baseline test. Extrapolating this value to the length of the specimen, calculated by assuming that the critically eroded surface extends the full length of the specimen (12 inches), the eroded volume becomes 9.6in³. Extrapolating again, assuming that the critically eroded surface extends for 1 mile, as most coastal sand dune systems extend for tens to hundreds of miles, under the design loading, the rate of erosion is 1.1 yd³/min. The 73% improvement is significant, but this suggests that the additional 15 flushes accounted for the added 20% from the 30 flush test, where the previous 15 flushes provided a 23% erosion reduction.

Figure 4.21: Laser Morphology Obtained Soil Surface Before and After Wave Action – 45 Flashes
60 Flashes Wave Test

The final wave tank test was treated to 60 flushes prior to testing. The input function and soil surface wave profile are shown in Figure 4.22. The tail end of the input function shown in Figure 4.22 shows some erratic behavior, associated with a pressure relief of the air compressor used to counteract the hydrostatic pressure on the actuator of the wave tank.

Figure 4.22: Wave Gauge Data - 60 Flush Test, a) Input Function and b) Soil Surface Wave Profile
Fortunately for this test, as observed by the soil surface wave profile in Figure 4.22b, the waves induced by the pressure relief did not have any direct effect on the center of the sample (the most lightly cemented zone), indicating that although the waves were irregular during the ramp down period, there was likely no effect on the test.

Figure 4.23 shows the initial and final soil surface states for the 60 flush wave test. There is a large differential between the cemented soil surface and the eroded untreated soil surrounding the sand box in Figure 4.23b. The final surface does not display an obvious signs of erosion at the centerline, an indicator that the larger cementation values had drastic effects on improving the potential of the sand to prevent erosion.

Based on the dissected soil box, eighteen soil samples were obtained using a hammer and chisel (due to high strengths associated with the large cementation values). These samples were used to create the cementation contour distribution shown in Figure 4.24a and the cross-
section at the centerline shown in Figure 4.24b. Cementation levels ranged from 6.6% at the center to 12.9% around the perimeter of the specimen.

The laser morphology obtained profile before and after the application of wave loading are shown in Figure 4.25. Logically, following the more heavily cemented distribution, the observed erosion was reduced to 0.7in² at the centerline. By the addition of the cementation, the erosion reduction was 83% when compared to the untreated baseline test. Extrapolating this value to the length of the specimen, calculated by assuming that the critically eroded surface extends the full length of the specimen (12 inches), the eroded volume becomes 6.0in³.
Extrapolating again, assuming that the critically eroded surface extends for 1 mile, as most coastal sand dune systems extend for tens to hundreds of miles, under the design loading, the rate of erosion is 1.9 yd³/min. When comparing the percent erosion reduction by flush numbers, the fourth set of 15 flushes added an additional 10% erosion reduction, meaning that the fourth round of 15 flushes were less efficient than the previous two rounds.

![Figure 4.25: Laser Morphology Obtained Soil Surface Before and After Wave Action – 45 Flushes](image)

Discussion

Based on the compilation of the eroded area calculated for the most critical cross-sections and the corresponding cementation level, Figure 4.26 shows the observed erosion reduction-cementation trend. As expected, as the cementation level increased, the stiffer soil
with improved critical shear stress, as shown in Chapter 3, become less susceptible to cementation. The average cementation level shown on the abscissa is the average cementation at the surface of the soil specimens at the center, where the majority of the erosion occurred. It was deemed unreasonable to take a global average of the surficial cementation as there was relatively no erosion on the most cemented outer perimeter of the sand boxes.

![Figure 4.26: Erosion of the Most Critical Section as a Function of Average Cementation Level](image-url)

Figure 4.26: Erosion of the Most Critical Section as a Function of Average Cementation Level
Using the data in a different manner, looking more into a design-type approach, for the nutrient concentrations used in this study, the erosion reduction as a function of the number of flushes are shown in Figure 4.27. The plot follows an approximately parabolic distribution, however, when extrapolating into a design recommendation, attention must be paid to understanding the input parameter, as the membrane action likely created lower average cementation levels and effectively caused a rightward shift of the data. If the sand box structures had been rigid, a more uniform cementation would have been observed and the data would have been more representative for a particular number of flushes. The trend, however, would have remained constant with a different slope.

Figure 4.27: Erosion of the Most Critical Section as a Function of the Number of Flushes
It is significant to note however, that as the number of flushes increase, the rate of change of eroded area per flush between data points decreases. The decrease in slope implies that each subsequent flush has a slightly lower effect on the decrease in eroded than the previous flush had. To assess the loss of efficiency during each addition flush, the loss in efficiency was gauged by comparing the eroded area from of the wave tank tests. The incremental reduction of erosion was evaluated by determining the percent reduction of a particular test when compared to the baseline and then taking the difference between two successive tests. Figure 4.28 shows the reduction of erosion (a) and incremental reduction of erosion (b) as a function of the number of flushes.

The erosion reduction curve shown in Figure 4.28a shows a large reduction of erosion in the first increment, up to 30% at 1.2% calcite. If the erosion-cementation level trend was linear, a 60% reduction would occur at 2.4% calcite, but based on the trend in Figure 4.28a, the interpolated calcite value would be 4.2% calcite. This trend occurs in each of the increments, and the incremental reduction of erosion is plotted in Figure 4.28b. Following the logic discussed for Figure 4.28a, the rate of incremental reduction of erosion decrease occurs linearly until approximately 5.7% calcite. After this value, the incremental reduction of erosion decreases rapidly. It is speculated that as cementation increases, the reduction of erosion curve approaches 100% which indicates the incremental reduction of erosion curve would approach zero analogously.
Figure 4.28: Effect of Mass of Calcite on a) Reduction of Erosion, b) Incremental Reduction of Erosion
4.6 Conclusions and Future Work

Following the completion of the wave tank testing, it was concluded that through the modification of the sandy soils with MICP, the erosive potential was dramatically reduced. Relatively low levels of cementation resulted in a 30% reduction in erosion while maintaining an apparent stiffness similar to that of snow. As expected, higher cementation values lead to increased resistance to erosion. To achieve larger cementation levels, the required effort increased as the cementation increased. More flushes are needed to facilitate a higher cementation as the flush to cementation level relationship is non-linear and the required flushes to get from 1% to 2% mass of calcite is less than the flushes required to get from 2% to 3%. In terms of erosive behavior, as the cementation level increases the overall reduction of erosion increases while the incremental reduction of erosion decreases. This has implications when doing a cost analysis for design, the cost/improvement ratio increases throughout the cementation process and if very large cementation levels are recommended, other approaches may be more economical.

To make the results obtained herein appropriate for design, it would be necessary for further study surficial cementation distributions with different boundary conditions. Coastal sand dunes are geological structures with non-horizontal surfaces, aiding in the difficulties associated with down-slope runoff of the treatment solution in-situ. As shown by the cementation distributions in this study, un-level surfaces tend to develop uneven cementation conditions in the unsaturated condition when a rigid boundary is not present to support a head of treatment solution. A modification of the treatment strategy to physically apply the nutrients
to the sand dune would be necessary for improving the resiliency of coastal sand dunes using MICP.

Future work entails an in depth analysis into the induced ecological parameters of the MICP treatments and the interaction with the existing ecosystem. Unlike most other geotechnical soil improvement applications, sand dunes are highly regulated by environmental agencies. For MICP to be truly applicable to coastal sand dunes, the environmental impacts would need to be minimized. A more thorough fundamental understanding of the mechanical behavior would be required prior to large-scale recommendation becoming a realistic option. This interdisciplinary issue would require geological, ecological, microbiological, and geotechnical engineering expertise to optimize and fully evaluate the repercussions of in-situ treatment.
5 CONCLUSIONS AND FUTURE WORK

In this thesis, the author investigated the suitability of MICP to coastal sand dunes using a three-pronged approach, ranging from the elemental-level in Chapter 2, the model-scale in Chapter 3, and the system-scale in Chapter 4. The conclusions from this thesis are summarized in this chapter, followed by a discussion of the recommended future work.

5.1 Conclusions

Treatment Optimization for Unsaturated Coastal Sands

At the elemental scale, Chapter 2 worked towards the optimization of the saltwater treatment solution with respect to effluent ammonium concentrations and the measured unconfined compressive strength. It was discovered that typical treatment concentrations used in the published literature were providing excess nutrients that were deemed to have entered the system and left without any interaction with the bacteria, resulting in no additional calcium carbonate precipitation. Through a series of unconfined compressive strength tests paired with acid washing and a pore-fluid batch test, it was shown that a 2:1 (urea to calcium chloride) nutrient ratio with Instant Ocean was found to be the optimal solution for saltwater based MICP treatments. The strengths obtained from this solution were comparable to those at much higher
concentrations while and the molarity of the effluent ammonium was reduced by a factor of 2 to 3.

The addition of the saltwater into the treatment solution did have an effect on the unconfined compressive strengths, especially at the larger cementation ranges. It was shown by the relationship between unconfined compressive strength and cementation level that at low cementation levels the saltwater and freshwater based treatment solutions provide a similar strength. Following the x-ray diffraction and SEM analysis, it was determined that contrary to the freshwater solution, the saltwater solution precipitated some sodium-based cementation, which likely lead to the strength differential between treatment types.

**Critical Shear Stress Evaluation of MICP Cemented Sands**

The addition of MICP into the soil matrix led to a several order of magnitude decrease in observed scour. Relatively low levels of cementation lead to a dramatic decrease in scour susceptibility. As expected, the failure mechanism switched from the sand erosion mechanism, particle by particle, to the cohesive soil mechanism, mass erosion. Although the actual critical shear stress could not be determined in the laboratory, the qualitative increase in scour resistance shown here may be used as a proof of concept for future study. It is hypothesized that the presence of the calcite cementation requires the applied stress to overcome the strength of the cementation instead of the frictional resistance.

**MICP Treated Dune Performance under Wave Action**

Relatively low cementation values lead to notable erosion reduction, on the order of 30%. During the treatment scheme, it was shown that during low cementation ranges, the
effect of cementation on the reduction of erosion was substantial, but as the cementation level increased, the incremental improvement because to diminish. Surficial cementation is most significant for reducing erosion and small variations in spatial cementation distributions result in drastically different erosion patterns.

Interconnection of Testing Scales

Through a systematic approach from three different scales, a broadened perspective on improving the resiliency of coastal sand dunes was ascertained. Focusing at the elemental level, induced ecological changes were considered while focusing on maximizing strength. Investigating the model scale, critical shear stress of the cemented soil was assessed experimentally, which is a key parameter in the modeling of dune erosion. System-level experimental testing was conducted to show that MICP can reduce erosion.

5.2 Future Work

Future work on the elemental-level include a comprehensive study of the induced ecological changes as a result of MICP, including: direct measurements of ammonium concentration and a study of the fate of ammonium, changes in available oxygen, monitoring localized pH increases, activity/presence of indigenous bacteria, and interaction with pre-existing dune grass. Further investigation into the chemical cementation induced during saltwater treatments is necessary prior to in-situ implementation, as the expected strength is likely a design parameter and correlated to the erosion resistance.
On the model-scale, actual defining of the critical shear stress is required to numerically connect the effect of cementation on the erosive behavior of MICP cemented sands. Numerical modeling using Xbeach (a two-dimensional model for wave propagation, long waves and mean flow, sediment transport and morphological changes of the nearshore area, beaches, dunes, and backbarrier during storms) is planned in the near future, prior to graduation of the author. Using this numerical modeling, it is expected to connect the laboratory obtained critical shear stresses with the observed erosion patterns at the varying cementation levels.
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