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EVALUATION OF FIELD BASED LIQUEFACTION APPROACHES FOR CALCAREOUS SANDS USING SHEAR WAVE VELOCITY

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**EVALUATION OF FIELD BASED LIQUEFACTION APPROACHES FOR
CALCAREOUS SANDS USING SHEAR WAVE VELOCITY**

BY

ALESANDRA CRISTINA MORALES VÉLEZ

**A DISSERTATION SUBMITTED IN PARTIAL FULLFILLMENT OF THE
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DOCTOR OF PHILOSOPHY DISSERTATION

OF

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ABSTRACT

The objective of this research is to assess the applicability of current field-based liquefaction approaches for calcareous sands. This is of importance because a.) there are many coastal communities in areas of high seismicity founded on calcareous sands (e.g. Haiti, Puerto Rico, Hawaii, Dubai, etc.), and b.) existing field-based approaches were developed almost entirely for case studies in silica sands. Recent earthquakes in Guam (1993), Hawaii (2006) and Haiti (2010) caused significant damage to coastal infrastructure, in part, because of liquefaction of calcareous sand deposits. Despite this field evidence, a review of existing research involving laboratory studies shows contradictory results as to how the cyclic behavior of these unique soils compares to silica sands.

This research was accomplished by performing a combined in situ and laboratory-testing program. The in situ testing program was conducted at a site in western Puerto Rico, which is an ideal site for this research because of the extent of calcareous deposits throughout the island, the relatively high seismicity of the region and the high population density. The in situ testing program consisted of conventional drilling and sampling techniques, Standard Penetration Tests with hammer energy measurements, Cone Penetration Testing with shear wave velocity measurements and Dilatometer testing with shear wave velocity measurements. The laboratory testing program consisted of cyclic triaxial and cyclic simple shear tests with shear wave velocity measurements. This work builds upon existing research that shows that values of shear wave velocity measured

appropriately in the laboratory and in situ can provide a link between laboratory testing and field measurements for characterizing the in situ behavior of soils that are difficult to sample, such as loose, uncemented, calcareous sands.

The laboratory tests results suggests that the cyclic resistance - shear wave velocity relationship ($CRR-V_{s1}$) for the Monterey and Cabo Rojo sand appears to be soil-specific and independent of sample preparation techniques. Also, this relationship appears to be less sensitive to changes in shear wave velocity than those reported in previous studies. Overall, the Cabo Rojo sand exhibited higher resistance to liquefaction than the Monterey sand.

The comparison of tests results for this study with existing field-based correlations suggests that the Monterey and Cabo Rojo sand lies outside the liquefaction resistance curves used in practice, which means that existing shear wave velocity field-based approaches would significantly overestimate the liquefaction resistance of both the Monterey and Cabo Rojo sand. There was reasonable agreement between the soil specific $CRR-V_{s1}$ and the CPT field-based approaches, however the soils at the site were so loose that definitive conclusions regarding the field-based approaches cannot be made at this time.

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CHAPTER 1 INTRODUCTION

1.1 Statement of the Problem

Calcareous sands are generally formed from the bodies of marine organisms and are composed of calcium carbonate (CaCO_3), usually 75% or more by weight (Scanlon et al., 1998; see Figure 1.1). These sands are often found in tropical and subtropical coastal regions of the United States, including Puerto Rico, Hawaii and Florida. Calcareous sands are typically characterized by higher void ratios, a wide variety of particle sizes and shapes, higher grain crushability, higher specific gravity, brittle stress-strain behavior, and higher compressibility compared to silica sands.

Figure 1.1. Scanning electron micrographs of a typical calcareous sand from a coastal site in Cabo Rojo, Puerto Rico (Cataño and Pando 2010).

Because of their unique characteristics, there are many geotechnical challenges in dealing with calcareous sands, including evaluating pile capacity, the effectiveness of ground improvement, and evaluating liquefaction potential. Liquefaction of calcareous sands occurred during earthquakes in Guam in 1993, Hawaii in 2006, and in Haiti in 2010 (Figure 1.2). During the earthquake in Haiti, in particular, there was significant liquefaction in the ports of Port-a-Prince which hampered international aid efforts.

Figure 1.2. (a) Ground cracks resulting from lateral spreading and liquefied sand ejected from ground cracks, Hawaii 2006 Mw=6.7 earthquake (adapted from Medley, 2006) and (b) Pre- and post-earthquake satellite/aerial imagery of the port at Puerto Principe, Haiti, 2010 Mw=7.0 Earthquake showing extensive damage and sand ejecta (adapted from Rathje et al., 2010).

The state-of-the-practice in the United States for evaluating the liquefaction potential of soils is to use field-based correlations that relate an in situ measurement of a soils' state (e.g. Standard Penetration Test, Cone Penetration Test, or Shear Wave Velocity) to the

earthquake-induced cyclic stresses required to make that soil liquefy. These correlations are built almost entirely on field studies where liquefaction either occurred or did not occur at sites where the predominant soil is quartz sand. As a result, there is considerable uncertainty in applying these field-based methods for calcareous sands.

1.2 Objective and Scope of Research

The overall objective of this research is to better understand how existing field based approaches for the assessment of liquefaction potential apply to calcareous sands that can be found in many tropic and subtropical coastal regions. This work builds upon existing research that suggests that values of shear wave velocity measured appropriately in the laboratory and in situ can provide a link between laboratory testing and field measurements for characterizing the in situ behavior of soils that are difficult to sample.

This research was accomplished through a detailed field and laboratory study of a calcareous sand site in Puerto Rico. Field testing included Standard Penetration Tests with energy measurements, Seismic Cone Penetration Tests, and Seismic Dilatometer Tests as well as the collection of representative samples. Laboratory tests included cyclic triaxial and cyclic simple shear tests with shear wave velocity measurements and mineralogical studies of the calcareous sands.

This dissertation is organized into seven chapters. In addition to this introduction, Chapter 2 presents a review of the literature review regarding the cyclic resistance of soils with a special emphasis on calcareous soils and field-based methods for assessing

liquefaction potential. Chapter 3 summarizes the field testing program and Chapter 4 describes the laboratory testing program developed for this study. Chapter 5 presents the results of the laboratory testing program, including detailed studies of both a calcareous and silica sand. The primary result of the laboratory testing program is a soil-specific relationship between cyclic resistance and shear wave velocity for the calcareous sand. Chapter 6 presents the assessment of the field-based methods for liquefaction potential and Chapter 7 summarizes the findings and conclusions of this dissertation.

1.3 Puerto Rico as the Study Area

Puerto Rico was chosen as the study area to evaluate the applicability of field-based liquefaction approaches for calcareous sands. Puerto Rico is an ideal site for this research because of the extent of calcareous deposits throughout the island and the relatively high seismicity of the region.

Scanlon et al. (1998) collected over 2500 samples of surficial sediments around Puerto Rico and classified them into three major types: marine sediments with high calcium carbonate content (more than 75% of CaCO_3); terrigenous sediments with low carbonate content (less than 25%); and mixed sediments with calcium carbonate content ranging between 25% and 75%. The distribution of these sediments in Puerto Rico is shown in Figure 1.3.

Puerto Rico lies along the Northern Caribbean Plate Boundary Zone (NCPBZ), which is a seismically active region characterized by convergence and lateral translation of the North American and Caribbean plates (Mann et al., 2002). The major tectonic elements of the region include the Puerto Rico and Muertos Trough subduction zones (located north and south of the island, respectively), the Anegada and Mona Passage (located east and west of the island, respectively), and segments of the Great Southern Puerto Rico fault zone, as shown in Figure 1.4. Geodetic data for the northeastern Caribbean region indicated an approximate 20-mm/yr. rate of relative motion across the NCPBZ. (Mann et al, 2002). A series of large-magnitude historical earthquakes reported in 1670, 1787, 1867 and 1918 demonstrate the seismic potential of the area. These earthquakes produced Modified Mercalli Intensities (MMI) between VII and VIII.

Based on these factors, a site in western Puerto Rico was chosen for the in situ testing and sampling program: Puerto Real, Cabo Rojo. The approximate location is shown in Figure

1.3. This site was also selected based on the results of previous research performed at the University of Puerto Rico at Mayagüez (UPRM) (Cataño and Pando 2010; Pando and Sandoval 2010). At the Cabo Rojo site, x-ray diffraction and thermo-gravimetric analyses indicated that the sands there contain more than 90% calcium carbonate (CaCO_3), making it an ideal test site for the proposed study.

Figure 1.4. Regional tectonic setting for the island of PR (Bachhuber et al. 2008).

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

This chapter presents a review of the basic concepts of soil liquefaction during earthquakes and documents both the state-of-the-art and state-of-the-practice of assessing liquefaction potential. A summary of research on the liquefaction resistance of calcareous sands is also presented. The field-based approaches for the assessment of liquefaction potential are described and the applicability of shear wave velocity to link laboratory and insitu behavior is explained.

2.2 Soil Liquefaction Overview

The shaking of ground during earthquakes can cause a loss of strength or stiffness of the soils resulting in unwanted settlement of structures, landslides, failure of earth dams, etc. This process leading to complete loss of strength is called soil liquefaction. Soil liquefaction occurs when loading in an undrained condition leads to an increase in pore pressure and a reduction in effective stress essentially to zero (i.e., a complete loss of shear strength). It can be initiated by either monotonic loading (e.g., a sudden or fast increase in load on an embankment) or cyclic loading (e.g. wave action or earthquake shaking). Liquefaction is most likely to occur in loose saturated cohesionless soils with poor drainage. Because it happens in saturated soils, it is mostly observed, but not limited to natural soils and man-made fills near rivers, beaches or bays.

The term liquefaction often encompasses several related phenomena: (1) flow liquefaction, (2) lateral spreading and (3) sand boils. Flow liquefaction can occur when the strength of the soil drops below the shear stresses needed to maintain stability under static loads. This type of failure is generally driven by static gravitational forces and can produce very large movements (e.g. earth dam collapse and foundation failure). An example of flow liquefaction was the flow failure in the upstream slope of the Lower San Fernando Dam in California caused by the San Fernando 1971 earthquake (Kramer, 1996).

Lateral spreading is characterized by incremental displacements during earthquake shaking. These displacements can range from negligible to large depending on the earthquake magnitude, peak ground acceleration, duration of shaking and strength of the stress pulses compared to the strength of the soil. Lateral spreading occurs on gentle sloping ground (e.g. 0.3 to 5% slopes) and it can affect large areas. Level-ground liquefaction does not involve large lateral displacement and is easily identified by the occurrence of sand boils (i.e. surface manifestation), which are produced by the ejection of ground water to the surface. This might not be as damaging as flow liquefaction and lateral spreading however it clearly indicates the existence of high groundwater pressures whose eventual dissipation can lead to subsidence, differential settlement and possible failure. For purposes of this manuscript, the term liquefaction will be used to describe the behavior of cohesionless soils, more specifically sands, with a contractive (loose) response that leads to a complete loss of strength when subjected to cyclic loads.

2.2.1 State of knowledge

Knowledge concerning liquefaction and its effect has come mainly from three distinct efforts. These are: (1) field observations during and following earthquakes, (2) experiments in the laboratory on saturated soil samples and models of foundations and earth structures, and (3) theoretical studies (NRC, 1985).

2.2.2 Field observations during and following earthquakes

Some of the most dramatic illustrations of liquefaction related damage to civil infrastructure were observed after the 1964 Niigata, Japan earthquake and 1964 Prince William Sound, Alaska earthquake, which helped to identify liquefaction as a major problem in earthquake engineering (Idriss and Boulanger, 2008). In the mainland United States, the 1971 San Fernando and 1989 Loma Prieta, California earthquakes caused extensive damage as well. All of these events drew attention to the problem of earthquake-induced liquefaction and liquefaction-induced settlement. The 1995 Kobe, Japan earthquake caused massive liquefaction damage along the manmade islands in the Kobe region. One of the largest container port facilities in the world is located in this region and liquefaction in these fills caused extensive damage to quay walls, cranes and supporting structures around the port facilities. The extent of the liquefaction damage for this event was so massive that almost caused complete loss of functionality for all the ports around Kobe, which translated to a huge economic loss.

More recent events, such as the 2010 and 2011 Christchurch, New Zealand earthquakes and the 2011 Fukushima, Japan earthquake also brought a lot of attention and interest to the liquefaction phenomena. All of these earthquakes produced examples of failures due

to shaking of saturated cohesionless soils such as structurally undamaged structures that experienced bearing capacity failure, buildings torn apart and broken pipelines due to lateral spreading, massive slides, sand boils, tilting and differential settlement of buildings, sinkholes and road and bridge damage. A review of all of these events clearly reveals that such failures are rather common effects of seismic shaking. A great deal of information on liquefaction behavior has come from postearthquake field investigations, which have shown that liquefaction often recurs at the same location when soil and groundwater conditions have remained unchanged (Youd, 1984).

2.2.3 Experiments in the laboratory

Numerous reports can be found in the literature explaining the concept of liquefaction on both undisturbed and reconstituted samples in the laboratory (i.e. Silver et al., 1976, Casagrande, 1975 and Castro, 1987). Results from these reports provide a basis for understanding the liquefaction problem, identifying the factors that control or govern the soils response or resistance to liquefaction and some indication of how accurate and reliable laboratory measurements can be.

It is well understood that the nature of the damage caused by the earthquake is strongly influenced by the response of the soils to cyclic loading, which is controlled by the mechanical properties of the soil. The dynamic properties of soils depend on the state of stress in the soil prior to loading and on the stresses imposed by the loading (Kramer, 1996). Numerous tests have been developed for the measurement of dynamic soil properties. Some of these tests are oriented toward measurement of low-strain properties and others toward behavior at larger strains. Laboratory tests are usually performed on

relatively small and reconstituted samples. Special care must be taken when dealing with reconstituted samples because soil fabric plays an important role on soil behavior. It is very important to understand that the ability of laboratory tests to provide accurate and reliable measurements of soil properties will completely depend on the ability to replicate insitu conditions (both in terms of state of stress and soil layering) and the loading conditions of interest.

The most common laboratory tests for determining small strain soil properties are (1) resonant column test, (2) ultrasonic pulse test and (3) piezoelectric bender element test. Bender elements have an advantage in that they can be installed in the end caps of both cyclic simple shear and cyclic triaxial tests. They are used to measure shear wave velocity, which can then be converted to small strain shear modulus. The resonant column test is used to measure low strain properties of soils (e.g. small strain shear modulus and damping). These two parameters changes as a function of shear strain. The ultrasonic pulse test is used in the laboratory to measure wave propagation velocities at very high frequencies. The test is performed using ultrasonic transmitters, which are made of piezoelectric ceramics that are attached to the end caps. The cyclic triaxial tests and cyclic direct simple shear tests will be described later on this manuscript.

An important aspect of any laboratory testing program is sample preparation. In principle, the cyclic behavior of a soil could be determined by obtaining high-quality field samples (i.e. frozen samples) and then testing them in the appropriate laboratory equipment at similar stresses in the field. Previous research and experience has shown that samples

obtained by conventional drilling and sampling methods are highly disturbed therefore recreating field conditions in the laboratory is a challenge. Measuring the cyclic strength of such samples will most likely result in unreliable measurements.

2.2.4 Models of foundations and earth structures

A group of researchers in the 1970's (e.g., Finn et al. 1971; Yoshimi and Tokimatsu 1977) investigated the dynamic properties of soils by means of shaking table tests. A major concern when dealing with small scale laboratory testing such as shaking table tests, cyclic triaxial tests, and cyclic direct simple shear tests is that the stresses developing from the weight of the soil are much less than those in the real field situation. We know that the stress-strain response of the soil is very non-linear, and aspects such as dilatancy can be significantly different for the small static shear stresses encountered in models rather than for the stresses associated with full-scale earth structures or foundations.

One way to overcome this major concern is to test models in a centrifuge where they are subjected to an increased gravitational field. For example, considering a structure that is 100 ft. high, a model can be easily constructed so that the structure is 1 ft. tall and tested on a centrifuge with a centrifugal acceleration of 100 g. Thus, the stresses at the points of interest in the model will be identical as those in the full-scale structure. Based on the assumption that the stresses are the same, and if other relevant properties between both the model and prototype are scaled appropriately, the resulting strains and deformations will also be the same. This makes centrifuge testing good for situations where we want to

observe or obtain the general pattern of soil behavior or response to ground motions and also provide reliable data for calibrating models.

Model tests on foundations and earth structures in a centrifuge have been very useful for studying the complex distribution of pore pressure and deformations that can develop once the soil has reached a zero effective stress state. For example, such tests have confirmed that even if a localized portion of an earth mass reaches a condition of liquefaction, a building foundation or an earth structure as a whole may remain stable (NRC, 1985). On the other hand, these tests have also shown that pore pressure can decrease in some localized parts of a denser soil element that has been subjected to ground motions.

2.2.5 Liquefaction fundamentals - cyclic loading behavior of saturated sands

Soil liquefaction can be defined as a phenomena occurring in saturated cohesionless soils, such a sands, gravels, non-plastic silts, where they lose all their strength as a result of a decrease in effective stress due to a sudden or fast build-up of pore water pressure during cyclic loading (i.e. undrained loading conditions such as earthquake loading, wave action or pile driving). The proper definition for soil liquefaction has been a continuing subject of debate within the geotechnical community over the past decades. Castro and Poulos (1977) argued that liquefaction should be clearly distinguished from a variation of it known as cyclic mobility, which can be defined as a phenomenon that occurs when the static shear stresses are less than the shear strength of the soil. Cyclic mobility can result in large permanent deformations after an earthquake. As stated previously, for purposes of this manuscript, the term liquefaction will be used to describe the behavior of

cohesionless soils, more specifically sands, with a contractive (loose) response that leads to a complete loss of strength when subjected to undrained cyclic loads.

Liquefaction results from the tendency of soils to contract under the application of shear stresses. When dealing with a loose and saturated soil deposit subjected to shear stresses the soil particles will tend to arrange into a denser structure (more particle contact) which results in less void space (i.e. low void ratios, high relative densities) as the pore water is forced out. Under undrained conditions (i.e., no drainage allowed, no volume change) the pore water pressure will start to build-up or accumulate as the soil is loaded (e.g. earthquake loading, sudden placement of an embankment, rapid drawdown) which will eventually result in a decrease of effective stress given the fact that the water has nowhere to go. A decrease in effective stress will lead to a decrease in shear stress. If the shear resistance of the soil becomes less than the static, driving shear stresses, the soil will liquefy resulting in large deformations (Martin et al. 1975; Seed and Idriss 1982).

Figure 2.1 shows this mechanism of pore pressure generation during cyclic loading. We can observe from this figure that soils would move from point A to B due to the changes in volume caused by drained cyclic loading conditions. But for undrained, saturated conditions, where no change in volume is allowed, the soil skeleton will exhibit a reduced effective stress (i.e. moving from point B to point C). At failure, the cyclic loading would completely break apart the load-bearing contacts among sand particles in such a way that the sand skeleton would be carrying zero normal stress ($\sigma' = 0$), and the pore water would be carrying the entire normal stress ($u = \sigma$) (Idriss and Boulanger 2008).

Figure 2.1. Mechanism of pore pressure generation during cyclic loading (Idriss and Boulanger 2008).

Figure 2.2 shows typical results for Sacramento River sand to undrained cyclic loading under triaxial testing conditions. These cyclic triaxial tests were carried out under anisotropically consolidated and undrained shear conditions. The Sacramento River sand is a clean sand and samples were prepared at medium-dense conditions. A uniform sinusoidal stress was applied during the undrained shear.

Figure 2.2. Response of Sacramento River sand to undrained cyclic triaxial loading (Idriss and Boulanger 2008).

A typical way to present the excess pore water pressures generated during shear is by means of the excess pore water pressure ratio (r_u), which is defined as:

$$r_u = \frac{\Delta u}{\sigma'_{3c}} \quad (2.1)$$

where Δu is the excess pore water pressure and σ'_{3c} is the minor effective consolidation stress. The minor principal stress is kept constant during the tests therefore the maximum value for $r_u = 1.0$ or 100%, which occurs when the excess pore water pressures equal the effective consolidation stress. This same concept applies to the standard cyclic simple shear tests but r_u is calculated based on σ'_{vc} which is the vertical effective consolidation stress, which is also kept constant during the shear phase. The $r_u = 1.0$ condition is often called “initial liquefaction” and is a very common failure criterion when running cyclic tests. From figure 2.2 it can be observed that that r_u increased gradually throughout the

cyclic shear phase until it failed at approximately 27 cycles. Right after initial liquefaction, the axial strains increased to about 2% in less than two additional cycles.

It is possible for a dense (dilative) material to reach a temporary zero effective stress condition (i.e. zero shear resistance). Cyclic loads can produce a reversal in the shear stress direction when the initial static shear stress is low. This means that the stress path passes through a state of zero shear stress. Given these conditions, a dense or dilative soil may accumulate enough pore water pressures to reach a condition of zero effective stress, which can potentially lead to the development of large deformations. Robertson (1994) called this behavior cyclic liquefaction. Liquefaction of a dense-of-critical sand during cyclic loading results in limited strains because the sand exhibits dilative behavior under subsequent monotonic loading (Idriss and Boulanger 2008).

It is well known that the cyclic strength of sands or their resistance to liquefaction depends on several factors such as number of loading cycles, relative density, confining stress, depositional method, fabric, structure, stress-strain history, age, cementation, mineralogy, and other environmental factors (Idriss and Boulanger, 2008; Kramer, 1996). Liquefaction of saturated sands can be triggered by different combinations of uniform cyclic shear stress ratio (CSR is the ratio of cyclic shear stress to initial effective confining stress) and number of loading cycles (N). The greater the CSR, the fewer loading cycles need to trigger liquefaction, whereas a smaller CSR will definitely require more loading cycles. This concept is illustrated with a series of shaking table tests carried out by De Alba et al. (1976) on Figure 2.3. It is important to clarify the definition of CSR

depending on the type of tests and boundary conditions. For shaking tables or cyclic simple shear tests, CSR can be defined as:

$$CSR = \frac{\tau_{cyc}}{\sigma'_{vc}} \quad (2.2)$$

where τ_{cyc} is the cyclic shear stress acting on the horizontal plane and σ'_{vc} is the vertical effective consolidation stress. For isotropically consolidated cyclic triaxial tests the CSR is typically defined as:

$$CSR = \frac{q_{cyc}}{2\sigma'_{3c}} \quad (2.3)$$

where $q_{cyc}/2$ is defined as the maximum cyclic shear stress and σ'_{3c} is the isotropic consolidation stress.

Figure 2.3. The CSR required to reach initial liquefaction ($r_u=100\%$), from shaking table tests by De Alba et al.1976 (adapted from Idriss and Boulanger 2008).

The point at which a sample fails (e.g., by pore pressure ratio equal to unity) under a given CSR is typically called the cyclic resistance ratio (CRR). The CRR-N plots as a straight line on a log-log plot, whereas it plots as a curve on the semilog plot format shown in Figure 2.3. From this figure it can be observed that the CRR of the tested sands increases with increasing relative density, as expected given that the denser samples will require more work to cause liquefaction.

As previously stated, the CRR also depends on the confining stress, which reflects the ability or tendency of sands to dilate or contract. This concept is illustrated on Figure 2.4 for a series of isotropically consolidated undrained cyclic triaxial tests carried out on Fraser Delta sand by Vaid and Sivathayalan (1996). The plot at the left shows the CSR required to reach 3% shear strain in 10 cycles versus the effective consolidation stress for samples made at relative densities ranging from 32 to 72%. It can be observed from this plot that the cyclic strength of the Fraser Delta sand increased as the effective consolidation stress increased for all the different relative densities. At low relative densities ($D_r = 31\%$ and 40%) the CSR- D_r relationship is almost linear, but at higher relative densities the relationship becomes non linear. The plot on the right shows the CRR at 10 cycles versus relative density. From this plot we can observe that the CRR increased as the relative density increased but that it also decreased as the effective consolidation stress increased from 50 kPa to 400 kPa at relative densities greater than 30%.

Figure 2.4. Cyclic triaxial tests results for clean Fraser Delta sand, showing that cyclic stress and the CRR cause a 3% shear strain in 10 uniform cycles at D_r values of 31-72% and effective consolidation stresses of 50-400 kPa (adapted from Idriss and Boulanger, 2008).

In 1983, Seed introduced the overburden correction factor (K_σ) to represent the dependence of CRR on consolidation stress. This overburden correction factor is defined as:

$$K_S = \frac{CRR_{\sigma'_c}}{CRR_{\sigma'_c=1}} \quad (2.4)$$

where $CRR_{\sigma'_c}$ is the CRR under a specific value of effective consolidation stress and $CRR_{\sigma'_c=1}$ is the CRR at 1 atm (100 kPa). The overburden correction factor varies with both relative density, type of tests and is also affected by soil fabric (e.g. laboratory reconstituted samples versus tube samples of natural soils).

2.3 Cyclic Resistance of Calcareous Soils

Considerable research has been performed over the past 30 years to better understand the behavior of these soils due to the increased development of offshore oil platforms, port facilities and coastal infrastructure in tropical and coastal areas.

Calcareous sediments, particularly in uncemented state, have been known to liquefy under dynamic loads (e.g., Frydman et al. 1980; Ross and Nicholson 1995; Flynn 1997; Morioka and Nicholson 2000; and Pando and Sandoval 2010; among others). The damage caused by strong motions on calcareous sediments has been made clear by numerous natural disasters such as the Guam 1993 earthquake, Hawaii 2006 earthquake, and Haiti 2010 earthquake. In each of these cases, calcareous sediments experienced liquefaction that resulted in extensive damage to homes, hospitals, schools, government and port facilities, and offshore structures.

It is also known that calcareous soils pose a challenge to coastal and offshore construction and ground improvement projects, particularly when installing driven piles and performing vibro-compaction (e.g., Datta et al. 1982; Nauroy and LeTirant 1985; Al-Homoud and Wehr 2006).

Research on the cyclic resistance of calcareous sands in the laboratory dates back to 1980 when Frydman et al. (1980) carried out a series of cyclic triaxial tests on reconstituted and undisturbed samples of cemented calcareous sand. This initial study suggested that silica sands have higher resistance to liquefaction than cemented calcareous sands. Since then, many researchers (e.g. Kaggwa et al., 1988; Datta et al., 1982; Airey and Fahey, 1991; Ross and Nicholson, 1995; Hyodo et al., 1996; Sharma and Ismail, 2006; Mao and Fahey, 2003, Cataño and Pando, 2010; Pando and Sandoval, 2012; and others) have investigated the engineering properties and cyclic resistance of these soils.

The following section presents a brief discussion of different studies available in the literature regarding the cyclic behavior of calcareous sands.

2.3.1 Frydman et al. (1980)

Frydman et al. (1980) carried out a liquefaction study of cemented sands from the Mediterranean coastal plain of Israel. The calcareous sands deposits found in this area, known as kurkar deposits, extend almost entirely along the coastline of Israel. The kurkar deposit is extremely variably cemented throughout its depth. The variability ranges from uncemented sand to well cemented hard, calcareous sandstone. Standard penetration tests were carried out at the site to define the soil profile and collect samples for further laboratory testing. Early in the investigation it became evident that the recovery of undisturbed samples by conventional drilling and sampling techniques was going to be very difficult. There was doubt with both interpretation of the blow counts and classification of the soil layers due to the cemented nature of the sand deposit and the apparent degree of breakdown of the cementation bonds that occur during driving of the split spoon sampler.

Efforts were made to study a possible correlation between (1) blow counts and the gradation of the soil retained in the split spoon sampler, (2) SPT and field density tests results and (3) blow counts and measured relative density. No correlation or relationship was found between them. These efforts only helped to confirm that the SPT leads to considerable breakdown of cemented particles.

Because recovery of undisturbed samples by conventional drilling and sampling methods was practically impossible, block sampling was used to obtain samples for laboratory testing. These were obtained by carving the soil from around a block and enclosing it on a wooden box. The spacing between the soil and the box was filled with paraffin wax. The block sample was placed into a freezer at a temperature of -20°C for at least 10 hours and then transferred to an insulated dry-ice box at a temperature of -45°C for 24 hours. Block sample was frozen at moisture water content (w) of 12%, corresponding to a degree of saturation (S) of 60%, allowing enough air void room for expansion during ice formation without excessive damage to soil structure. Once the block sample was frozen, cylindrical specimens (70.5 mm diameter and 155 mm height) were trimmed using a diamond-tipped core barrel and CO_2 as drilling fluid. The cyclic stress of the undisturbed samples of kurkar-sand was evaluated by means of cyclic stress-controlled triaxial testing. The frozen trimmed sample was placed in the triaxial cell surrounded by a latex membrane under a cell pressure of 0.35 kg/cm^2 and allowed to thaw. Once the specimens thawed completely, they were saturated by first flowing CO_2 and then deaired water under backpressure. Samples were saturated once the Skempton's pore pressure B -coefficient was no less than 0.96. Samples were consolidated under different initial effective confining pressures. Results of the cyclic triaxial tests on the kurkar sand are shown in Figure 2.5. The failure criterion was selected as when the pore pressure ratio equaled unity ($r_u = 1$). It can be observed from Figure 2.5 that the cyclic strength of the intact kurkar specimens is generally lower than of the reconstituted Monterey sand specimens. As part of this study, a series of cyclic triaxial tests were carried out on reconstituted specimens of the sand fraction recovered in the site investigation with the

purpose of comparing the effects of soil fabric on soil behavior. Specimens were prepared to relative densities between 60% to 95% using the wet tamping technique developed by Silver (1976). These results are also shown in Figure 2.5. It is evident that the cyclic strength of the reconstituted samples is considerably lower than that of Monterey 0 sand samples prepared at the same relative density. Furthermore, only the medium dense to dense samples had cyclic strength comparable to those of the intact specimens. The authors of this study suggest that the cyclic strength of the intact specimens may be dependent on the uncemented portion of the soil structure.

Figure 2.5. Cyclic strength of kurkar sand for intact and reconstituted specimens at $\sigma'_c = 1 \text{ kg/cm}^2$ (adapted from Frydman et al. 1980).

2.3.2 *Ross and Nicholson (1995)*

The Hawaiian Islands are known to have a great deal of seismic activity and deposits of calcareous sands surround its coastal areas. They have experienced both on land and offshore high magnitude earthquakes in the past. Recent major infrastructure has been developed along the coastline of Hawaii and engineers have become concerned about the soil behavior and strength of these calcareous deposits. Back in 1990 a study was commissioned to determine the liquefaction potential in the low-lying areas of south Oahu between the Honolulu International Airport and Waikiki. The study relied on old historical boring logs and applied the Seed and Idriss modified method to evaluate liquefaction potential. They found that for some areas the probability of liquefaction in the event of a magnitude 6.5 to 7.0 earthquake is greater than 48%. This study was made under the assumption that calcareous sands behave in the same manner as the silica sands that Seed and Idriss used to develop their method to evaluate liquefaction potential. Also, they assumed that the measured penetration data in calcareous deposits represents similar engineering characteristics to penetration data in silica sands of more temperate climates.

A series of stress-controlled cyclic triaxial tests were carried out to investigate the differences between calcareous and silica sands. Three different gradations of calcareous sand and one gradation of silica sand were used. The calcareous sands were obtained from locally derived dredged material that had been widely used as a fill material. Samples were prepared using a wet tamping technique and were vacuum-saturated to replace any air in the specimen with water. Densities ranged from 39%-45% for the silica specimens and 43%-56% for the calcareous specimens, after consolidation. Samples were isotropically consolidated to an effective pressure of $\sigma'_3 = 200$ kPa. This value was

chosen because it was found to be a convenient level at which to run the tests; it didn't necessarily represent any specific depth of interest. Samples were sheared at a frequency of 1 Hz and failure criterion was selected as double amplitude strain of 5% (DA=5%). Figure 2.6 shows the results of the cyclic triaxial tests carried out on the three different calcareous materials and the silica sand. It can be observed that the three calcareous sands exhibit very similar cyclic strength curves. Minor differences can be attributed to difference in gradation, fine content and membrane compliance errors. The authors stated that variations in specimen density also played a key role in the variability of the results. Membrane penetration was visually present in the 10/20 coral specimens. The silica specimens were smooth, with no visual indication of penetration. The cyclic strength for the silica sand was significantly lower than those of the coral materials in the range between 0 and 20 cycles. This can be attributed to the lower relative densities at which the silica sands were tested. The silica sands tested in this study were uniformly graded fine materials that would seem to require only moderate build-ups in pore pressure to cause grain sliding. The calcareous material exhibits somewhat greater strength due to the angular and softer nature of its grains and their higher friction angle.

Figure 2.6. Silica vs. Coral Cyclic Strength Curves (Ross and Nicholson 1995).

2.3.3 Morioka and Nicholson (2000)

A series of cyclic triaxial tests and full-scale penetration tests in a calibration chamber were carried out on calcareous and silica sands to develop relationships between cyclic strength and relative density, tip resistance and relative density, and cyclic strength and tip resistance.

For this comparative study, two soils were chosen: (1) a buff to grayish-tan, poorly graded, carbonate sand from a quarry in the Ewa Plains on the southwest coast of Oahu, Hawaii and (2) a Monterey Silica sand. The Ewa Plain sand was considered to be the perfect candidate because it exhibits a number of traits found in many calcareous deposits such as poor grading angular grains, high specific gravity, and high maximum and

minimum void ratios. The calcium carbonate content was approximate 98%, the specific gravity was 2.72 and the minimum and maximum void ratios were 0.66 and 1.3, respectively. The Monterey sand is a buff-to-tan very fine to coarse, poorly graded, terrigenous sand with a specific gravity of 2.63 and minimum and maximum void ratios of 0.33 and 0.71, respectively. This sand was sieved so that its gradation matched the Ewa Plains sand. The Monterey sand was used as the control sand. Results were compared and used to investigate the validity of using silica correlations to predict the properties of calcareous sands.

For the calibration chamber tests, the Ewa Plains and Monterey silica specimens were prepared at relative densities ranging from 40% to 70% and were subjected to cone penetrometer tests in the laboratory under different confining pressures. In order to investigate the effects of light cementation on tip resistance and cyclic strength of calcareous materials, both natural and artificial aging experimental programs were developed. A 60-day aging period was selected to approximate the effects of short-term aging on calcareous deposits based on the fact that previous studies carried out by Ross and Nicholson (1995) and Flynn and Nicholson (1997) showed that the majority of the effects of short-term aging occur within a few months. Samples were prepared at a relative density of 65% because samples prepared at this density were more uniform and consistent than those made at lower density. Loose samples exhibited large and irregular amounts of consolidation during initial confining stress application and mold removal. Specimens were saturated and aged under an effective stress of 100 kPa. Quikrete cement mix was used as the bonding aging for the artificially cemented specimens. Samples were

also prepared at a relative density of 65% and mixed with 2%, 4% and 6% dry weight ratios of the cement mix. These samples were allowed to cure for 3 days under atmospheric conditions prior to shearing. Figure 2.7 shows the results for the corrected tip resistance versus the relative density of the tested specimens. This figure shows that the tip resistance of silica sand is generally 1.5 to 2 times greater than that of calcareous sands at similar relative densities, meaning that the use of existing correlations for silica sand to estimate relative density of calcareous sands would underestimate the relative density. This might also lead to the underestimation of many other engineering design parameters such as liquefaction resistance. Even though the tip resistances in the artificially aged Ewa Plains samples were much closer to that of Monterey silica sand one can assume that, regardless of short-term aging effects, the use of silica correlations to determine relative density for loosely deposited calcareous material would be conservative.

Figure 2.7. Corrected Tip Resistance vs. Relative Density (Morioka and Nicholson 2000).

The stress-controlled cyclic triaxial tests were carried out on both sands at relative densities of 40%, 50% and 65% and sheared at varied cyclic shear stresses. Cyclic stress ratio was defined as the ratio of one-half the applied vertical stress divided by the effective confining stress ($CSR = \sigma_D/2\sigma'_3$). Cyclic triaxial tests were also performed on naturally and artificially aged Ewa Plains specimens. The cyclic testing program was carried out at an effective confining pressure of 100 kPa on all samples, to establish a consistent baseline and future database. Failure was defined as $DA = 5\%$. Figure 2.8 shows the results for the cyclic strength of all the tested materials. This figure suggests that at similar relative densities, greater cyclic loads are required to induce liquefaction in the carbonate sands than the silica sands. Also, it can be observed from this figure that calcareous sands exhibited greater liquefaction resistance under cyclic loading.

Figure 2.8. Cyclic Stress Ratio Curve Comparison (adapted from Morioka and Nicholson 2000).

Another significant difference between these two different sands was their pore pressure response. The Monterey Silica sand typically showed a steady increase in pore pressure until sudden failure, which occurred at approximately a pore pressure ratio of 75% to 85%. Pore pressure ratio is defined as the ratio between the amount of excess pore water developed during cyclic loading and the initial effective confining stress. The Ewa Plains calcareous sand showed a greater pore pressure relaxation between cycles and failed at a pore pressure ratio between 85% and 95%. This is illustrated on Figure 2.9. These differences might be attributed to the angular nature of the calcareous materials.

Figure 2.9. Pore pressure response for the Ewa Plains and Monterey Silica sands (Morioka and Nicholson 2000).

Figure 2.10 shows the correlation developed for tip resistance and cyclic stress ratio for both materials. Earthquake magnitudes of 6.0, 6.5, and 7.0 were chosen because this range of magnitudes for seismic events is considered to be probable maximums for design purposes in the Hawaiian Islands. In general, it can be concluded that the calcareous materials appear to have more resistance to liquefaction than silica sands. Therefore, using the correlations available in the literature for silica sands will most probably underestimate the relative density of calcareous materials. These existing

correlations developed for terrigenous or silica sands provide a good basis for estimating liquefaction potential but it is clear that more detailed correlations for calcareous sands must be developed and verified.

Figure 2.10. CSR_i vs Corrected Tip Resistance (adapted from Morioka and Nicholson 2000).

2.3.4 Sharma and Ismail (2006)

A series of undrained isotropically consolidated cyclic triaxial tests were carried on reconstituted samples of two different calcareous sands (1) Goodwyn Sand (GW) and (2) Ledge Point Sand (LP). Differences between the grain structure of GW and LP can be clearly observed from the ESEM micrograph shown in Figures 2.11 and 2.12. One or two

shell types, particularly a thin-walled mollusk and thick-walled foraminifera, control the grain structure of the GW soil. The soil matrix of the LP soil is a mixture of grains of clastic and bioclastic origin. The gradation curve of the soils is shown in Figure 2.13. It can be observed that the GW soil is finer than the LP soil. According to the USCS, the GW soil classifies as silty sand (SM) and the LP soil as sand (SP). Triaxial samples (71.5 mm in diameter and 163 mm in height) were prepared using a dry pluviation technique. A small vacuum of 10 kPa was applied to the samples during the preparation. Samples were saturated with CO₂ and flushed with de-aired water. Backpressure saturation was performed under an effective stress of 10 kPa; this guaranteed B-values greater than 0.95. Tests were performed under different combinations of cyclic stresses, effective confining stress and initial relative density. The tests were classified as “one-way” or “two-way” tests, depending whether there was a principal stress reversal during cyclic loading. “One-way” refers to cycling with a positive deviator stress regime while “two-way” refers to cycling with both positive and negative deviator stress.

The cyclic strength of GW and LP sands tested under different confining stresses and relative densities is shown in Figures 2.14 and 2.15. Results are shown in terms of the cyclic stress ratio ($q_{cyc}/2p'_o$), where p'_o is the initial mean effective stress, and number of cycles to failure (N_f). Failure was defined as either when the sample could not sustain the imposed q_{max} or when the accumulated strain exceeded 30%, whichever happened first. Generally speaking, it can be observed from these figures that the cyclic strength depends on both the initial confining stress and relative density of the soils. It is also clear from these figures that the type of shearing, either one-or-two way, has a significant effect on

the cyclic strength of the soils. Samples tested under one-way loading conditions showed the higher cyclic resistance.

Figure 2.11. ESEM micrographs of GW soil showing: (a) angular particles of different origins; (b) hollow fragile particles (Sharma and Ismail 2006).

Figure 2.12. ESEM micrograph of LP soil: (a) particles with different origin; (b) close-up view of the particles showing rounded edges with high aspect ratio (Sharma and Ismail, 2006).

Figure 2.13. Particle size distribution (adapted from Sharma and Ismail 2006).

Figure 2.14. Cyclic strength curves for GW sand at $p_o' = 200$ kPa and $D_r = 75\%$ (adapted from Sharma and Ismail 2006).

Figure 2.15. Cyclic strength curves for LP sand at $p_0' = 50$ and 200 kPa and $D_r = 30$ -75% (adapted from Sharma and Ismail, 2006).

2.3.5 Brandes (2011)

A series of constant volume (undrained), K_0 -consolidated cyclic simple shear tests were carried out on three different calcareous sands from the Hawaiian Islands (East Island sand, Maui Dune sand, and Kawaihae Harbor sand), one sand from Tampa Bay and one quartz sand (Ottawa 20/30). The purpose of this study was to investigate the dynamic behavior of such unique sands and compare it with quartz sands of terrigenous origins. The calcareous sediments used in this study were composed of detrital skeletal grains derived from the breakdown of coral reef structures, mollusks, algae, echinoderms and sponges. Grains have been broken down by wave and current action. They are of Holocene age and reflect a mineralogy dominated by aragonite and magnesium-rich

calcite, which is typical of very young sediments. There was no evidence of cementation. The Kawaihae Harbor sand was obtained from areas that liquefied during the 2006 earthquake. Table 2.1 shows a summary of the index properties of the sands. Figure 2.16 and 2.17 show a picture of the sand grains and the grain size distributions of the sands, respectively.

Figure 2.16. Nevada (quartz), Maui Dune (calcareous) and Kawaihae Harbor (calcareous) sands (Brandes 2011).

Table 2.1. Index properties of the sands tested (Brandes 2011).

Sand	USCS (-)	Fines (%)	Grain angularity (-)	$\gamma_{\min}$ (g/cm ³)	$\gamma_{\max}$ (g/cm ³)	CaCO ₃ (%)	G _s (-)
Ottawa 20/30	SP	0	rounded	1.49	1.9	0	2.65
Nevada	SP	1	rounded to subrounded	1.38	1.73	0	2.68
Tampa Bay	SP	3.5	subrounded to subangular	1.38	1.83	11	2.74
Maui Dune	SP	0.6	subangular	1.51	1.71	100	2.76
East Island	SP-SM	8.7	subangular	1.28	1.52	100	2.8
Kawaihae Harbor	SM	13.9	subangular to angular	1.34	1.68	100	2.75

Figure 2.17. Gradations (Brandes, 2011).

Sand specimens were prepared using dry pluviation. A flexible wire reinforced membrane was used to provide confinement to the sample. This membrane prevents lateral strains and imposes a K_0 lateral stress conditions, which represents insitu conditions. A new batch of sand was used for every test. Samples were 16 mm in height and had an area of 35 cm^2 . Loose samples were pluviated into the membrane with little if any need for further densification. Dense samples required different amounts of densification (tamping and vibration) after pluviation. Once the samples were made, they

were loaded vertically and allowed to consolidate for a short period of time until no vertical displacement was observed. Then samples were subjected to a sinusoidal constant shear stress amplitude loading at a frequency of 0.5 Hz. Tests were continued until they reached 20% (one-way) shear strain. The shear phase was carried out under constant volume conditions (constant height) meaning that as the specimen is cycled, the normal stress decreases, with the reduction equivalent to the increase in pore water pressure under undrained conditions. Liquefaction was assumed to occur at a shear strain of 3.75%.

Typical results from the cyclic simple shear tests are shown in Figure 2.18 for the quartz (Nevada) sand and the Kawaihae Harbor sand. The difference in cyclic behavior is very clear. These particular tests shown in Figure 2.18 were carried out at similar relative densities, although the initial void ratio for the Kawaihae sand is slightly larger. Despite the lower void ratio and CSR for the Nevada sand test, it liquefied after 14 cycles, compared to 22 cycles for the Kawaihae sand. Thus it seems that the Nevada sand is more susceptible to liquefaction. This trend becomes more evident when all the results for all the materials are combined in terms of standard cyclic liquefaction resistance curves, which are shown in Figure 2.19. It can be observed that the Nevada sand trend line is located below the ones for the calcareous sands. It is clear that the cyclic strength of the calcareous sands tested in this study is larger than that for the quartz sand. This holds true even when one considers the effects of the initial stress and void ratio (densities) and these results agrees with those of Morioka and Nicholson (2000). Differences in the cyclic strength of these soils can be attributed to differences in particle

shape and gradation. The Kawaihae and Maui calcareous sands are more elongated and angular than the Nevada sand. Elongated and angular particles are known to require more work to accommodate cyclic loading. Therefore a larger number of cycles are necessary for the calcareous sands to liquefy. The gradation of the Nevada sand is narrower than that of the two calcareous sands, as it can be observed from Figure 2.17. It is well known that sands with uniform grain sizes are more susceptible to liquefaction than well-graded ones. No visual evidence of particle breakage was observed after testing for the stress levels in this study.

Figure 2.18. Cyclic simple shear test results for (a) Nevada sand and (b) Kawaihae Harbor sand (Brandes 2011).

Figure 2.19. Cyclic liquefaction resistance for calcareous and quartz sands (Brandes 2011).

As part of this study, all the sands were subjected to bender element testing using the same setup as the cyclic DSS tests. The purpose of this part of the study was to evaluate the small-strain response of the sands. Samples were prepared using dry pluviation technique, were loaded vertically and shear wave velocities were measured with a set of bender elements at various vertical stresses. The maximum shear modulus was calculated based on the shear wave velocities assuming an elastic response using the following equation:

$$G_{\max} = \rho V_s^2 \quad (2.5)$$

where ρ is density and V_s is shear wave velocity. Results are summarized in Figure 2.20.

Figure 2.20. (a) Shear Wave values and (b) shear modulus values from bender element tests (Brandes 2011).

The range of shear wave velocities and small-strain shear stiffness are similar to those found in the literature (Lee and Santamarina 2005 and Hanchar 2006), although very few measurements have been documented for calcareous sands or for normal stresses beyond 200 kPa. It can be observed from Figure 2.20a that the shear wave velocities for the

calcareous specimens are significantly higher than those for the Nevada specimens, despite the fact that these sands were prepared at lower relative densities and higher void ratios. This behavior might be attributed to larger grain-to-grain contact areas that developed as interparticle stresses cause some grinding and refitting of the interfaces, which at the same time results in tighter and better grain contacts that are able to transmit acoustic energy faster and more effectively. The differences in shear wave velocities for calcareous and quartz sands is a bit problematic when assessing the liquefaction potential using field methods based on shear wave velocity measurements.

2.3.6 Sandoval et al. (2011)

The authors carried out an experimental laboratory investigation on the liquefaction susceptibility of uncemented calcareous sands from southwestern Puerto Rico. This is the same sand used for the current study. The experimental program consisted of stress-controlled undrained isotropically consolidated cyclic triaxial tests on reconstituted samples of the calcareous sands. Ottawa sand samples were also tested under similar conditions to establish a baseline for comparison purposes. The calcareous sands used in the study have grains composed of marine organisms as illustrated in the micrographs shown in Figure 1.1. A unique characteristic of these sands is their intraparticle void structure, which combined with the skeletal void ratio can result in total void ratios as high as 2.1. Another peculiar characteristics of this sand is its high specific gravity of approximately 2.84, which is significantly larger than the typical specific gravity value of 2.65 for silica/quartz sands. The calcium carbonate content for this sand was measured to be at least 91%. Table 2.2 shows a summary of the index properties of the sands used in this study and Figure 2.21 shows the grain size distribution of the materials.

Table 2.2 Properties of the sands used in this study (Sandoval et al. 2011).

Parameter	Cabo Rojo Sand	Ottawa #20-30 Sand	ASTM Standard
D_{10} (mm)	0.24	0.65	
D_{30} (mm)	0.3	0.71	
D_{50} (mm)	0.37	0.75	ASTM D422-63
D_{60} (mm)	0.41	0.78	
C_u	1.75	2.1	
C_c	0.94	1.1	
G_s	2.84	2.65	ASTM D854
γ_{min} (kN/m ³)	9.1	14.6	ASTM D4254
e_{max}	2.07	0.78	
γ_{max} (kN/m ³)	11.1	17.3	ALTERNATIVE METHOD
e_{min}	1.51	0.5

Figure 2.21. Grain size distribution for the two tested sands (Sandoval et al. 2011).

Cyclic triaxial samples were prepared using the moist tamping undercompaction method developed by Ladd (1978). Sample dimensions after preparation were 102 mm in height and 51 mm in diameter. Moisture contents in between 4% to 25% and 2% to 8% were used for the calcareous and Ottawa sand, respectively. Lower moisture contents were used to achieve looser specimens whereas higher moisture contents were used to achieve a denser packing. Relative densities ranged between 20% to 75% prior consolidation and 29% to 85% after consolidation. Specimens were flushed with CO₂ at 15 kPa for about 40 minutes and then flushed with de-aired water in an attempt to displace any entrapped air and ensure fully saturated conditions. Specimens were isotropically consolidated at 50, 100 and 200 kPa. Once consolidation was finished, specimens were allowed to stabilize under the desired stress for 30 minutes and then the cyclic phase was initiated. Samples were sheared at a frequency of 1 Hz. For cyclic triaxial tests, the cyclic stress ratio (CSR) can be defined as the ratio between the maximum applied cyclic shear stress and twice the effective consolidation stress. The CSR values for these experiments ranged in between 0.13 and 0.45. At least three to five cyclic triaxial tests were carried out on the specimens per condition (i.e. relative density and effective confining stress). Failure (liquefaction) was defined as when the pore pressure ratio equaled unity or when the double amplitude strain was 5%. In this way, the test results allowed the development of CSR curves for each relative density and effective consolidation stress level. A summary of the results, in terms of cyclic resistance and numbers of cycles to failure for the calcareous sand specimens is shown in Figure 2.22. From this figure it can be observed that the liquefaction resistance of these sands increased with increasing relative density.

Figure 2.23 shows the cyclic triaxial results for the calcareous and Ottawa specimens prepared at a relative density in the range of 23%-27% and isotropically consolidated at 50 kPa. A similar comparison is shown on Figure 2.24 for specimens prepared at a medium dense state ($D_r \sim 64\text{-}68\%$) and consolidated to 100 kPa. From these two figures one can conclude that the Cabo Rojo calcareous sand exhibits, in general, a much higher liquefaction resistance than the Ottawa silica-based sand when prepared and tested under similar densities and stress level.

Figure 2.22. Cyclic resistance curves for the Cabo Rojo calcareous sand (Sandoval et al. 2011).

Figure 2.23. Cyclic resistance curves for the Cabo Rojo and Ottawa sands at loose relative densities (Sandoval et al. 2011).

Figure 2.24 Cyclic resistance curves for the Cabo Rojo and Ottawa sands at medium dense relative densities (Sandoval et al. 2011).

Figure 2.25. Excess pore pressure generation curves for medium to dense Cabo Rojo and Ottawa sand isotropically consolidated to an effective stress of 100 kPa (Sandoval et al. 2011).

A key aspect when running cyclic triaxial tests is the excess pore pressure generation. Figure 2.25 shows a summary of the pore pressure generation curves for medium to dense state for both Cabo Rojo and Ottawa sands consolidated to 100 kPa. In this figure, residual excess pore pressures (Δu_{res}) normalized with respect to the consolidation stress (σ'_{3con}) are presented as a function of normalized numbers of cycles to reach liquefaction (N/N_f). Seed and Lee defined residual excess pore pressure as the excess pore pressure value when the applied deviator stress is zero during each load cycle. We can observe several differences in excess pore pressure generation characteristics between the two sands used in this study. First, the calcareous sand developed larger excess pore pressures than the Ottawa sand during the earlier stages of the cyclic phase. Second, between loading cycles, the calcareous sand showed larger fluctuations in the excess pore

pressure, which might suggest some sort of stress relaxation between loading cycles. Third, the Ottawa sand showed a slow and gradual generation of excess pore pressures during the initial cyclic loading phase and very small fluctuations of pore pressures between cycles. Also, it can be observed that towards the end of the tests, as the specimens approached liquefaction, the Ottawa sand typically exhibited an abrupt or sudden increase in the excess pore pressure, while the Cabo Rojo sand showed a more gradual or incremental increase of the pore pressures as it reached liquefaction. The large fluctuations in the excess pore pressures shown by the calcareous sands could be attributed to some particle rearrangement and unique characteristics of the sand grains mentioned earlier on such as angularity, surface roughness and intra-grain porosity. These excess pore pressure generation characteristics exhibited by the Cabo Rojo calcareous sands was found to be similar to the one reported by Morioka and Nicholson (2000) and Ross and Nicholson (2000) for calcareous sand from the Hawaiian Islands.

Even among sands of similar mineralogy there is considerable variability in the measured cyclic resistance. This illustrates that special attention is required when dealing with this type of sands and performing liquefaction and site response analyses. Due to the unique grain characteristics, including surface roughness, microporosity, and compressibility, it is expected that excess pore pressure generation of these sands is different than silica sand particles. The important differences in particle shape and characteristics result in important differences in fabric, small strain stiffness, and pore pressure generation characteristics under undrained loading. Clearly more research on this aspect is warranted.

2.4 Field-Based Approaches for the Assessment of Liquefaction Potential

The state-of-the-practice for evaluating the liquefaction potential or cyclic resistance of a level ground site since the early 1980's has been to use in situ test-based approaches (Peck 1979). Cyclic resistance in the field is quantified by the Cyclic Resistance Ratio (CRR) defined by the ratio of the average horizontal cyclic shear strength (i.e. the cyclic shear stress necessary to cause liquefaction) to the initial vertical effective stress. The CRR is then correlated to an in situ test, such as the Standard Penetration Test (SPT), the Cone Penetration Test (CPT), or shear wave velocity (V_s). The use of these methods is summarized in Youd et al. (2001) and Idriss and Boulanger (2008) and is described briefly below.

The SPT-based approach was initially proposed independently by Seed and Idriss (1971) and Whitman (1971). The current standard-of-practice described in Youd et al. (2001) and Idriss and Boulanger (2008) utilizes the data and correlations of Seed et al. (1985) with some modifications. Cyclic resistance ratio is correlated to $(N_1)_{60}$, defined as the Standard Penetration Test blow count corrected to an effective overburden stress of 1 atm (~100 kPa) and a hammer efficiency of 60%. This correlation, shown in Figure 2.26 was developed from field evidence of liquefaction (e.g. sand boils, settlements, ground cracking) observed at sites that had experienced an earthquake. For each site, a representative value of cyclic stress ratio, CSR, and $(N_1)_{60}$ was selected and those data points are also shown on Figure 2.26. The CSR data used in this figure corresponds to an effective stress of ~100 kPa and an earthquake magnitude (M_w) of 7.5. The CRR correlations are defined by the boundaries that reasonably separate data from sites that

showed evidence of liquefaction from those that did not. A review of the data, which these curves are based on, indicates that none of the data points shown in the Figure were obtained from calcareous sand deposits (Cetin, 2000).

Figure 2.26. SPT clean-sand base curve for Magnitude 7.5 earthquakes with data from liquefaction case histories (Youd et al. 2001).

The procedure developed by Robertson and Wride (1998) constitutes the current standard-of-practice for evaluating liquefaction potential using the CPT (Youd et al. 2001). This approach correlates CRR with cone tip resistance normalized (q_{c1N}) to an effective overburden stress of 1 atm (~100 kPa) (Figure 2.27). Like the SPT-based

procedure, the cyclic resistance correlation was developed from case histories in relatively clean sands containing less than about 35% fines. Again, no calcareous sands are included in the case studies used to develop the CPT approach.

Figure 2.27. Curve recommended for calculation of CRR from CPT data along with empirical liquefaction data from compiled case histories (Youd et al. 2001).

The existing correlation for evaluating liquefaction resistance using shear wave velocity field data was developed by Andrus and Stokoe (2000). In this correlation, the cyclic resistance ratio (CRR) is correlated with shear wave velocity normalized to an effective stress of 100 kPa (V_{S1}). Like the SPT and CPT based methods, Figure 2.28 was developed using observations of liquefaction at sites subjected to earthquakes.

Figure 2.28. Liquefaction relationship recommended for clean, uncemented Holocene soils with liquefaction data from compiled case histories (Youd et al. 2001).

Another method used to evaluate the liquefaction potential of soils is the flat dilatometer tests (DMT). This procedure is summarized in ISSMGE TC-16 (2001). This method was developed by Reyna and Chameau (1991). Figure 2.29 shows the suggested curve to estimate the cyclic resistance ratio (CRR) from the horizontal stress index (K_D).

Figure 2.29. Recommended curve for estimating CRR from K_D (Reyna and Chameau 1991).

2.5 Applicability of Shear Wave Velocity to link Laboratory and Insitu Behavior

Previous research has shown that the cyclic resistance-shear wave velocity (CRR- V_s) relationship is soil specific and can be used to link in situ test results such as CPT and SPT with laboratory determined values of CRR. Shear wave velocity (V_s) has been shown to be a good measure of the combined effects of stress and fabric, and it is hypothesized that samples consolidated or prepared to in situ values of V_s yield the in situ CRR. This is supported by several studies in which reconstituted samples had the same

cyclic behavior as frozen samples (Tokimatsu et al. 1986), high quality piston samples (Wang et al. 2006), and block samples (Baxter et al. 2008) provided the reconstituted samples were prepared to the same V_s as the undisturbed samples.

Figure 2.30. Variation of cyclic resistance and shear wave velocity for two soils illustrating that the CRR- V_s relationship is both soil specific and independent of sample preparation method (adapted from Tokimatsu et al. 1986 and Baxter et al. 2008).

This concept is illustrated in Figure 2.30, which shows CRR- V_s relationships for Niigata sand (Tokimatsu et al. 1986) and Olneyville Silt (Baxter et al. 2008). The CRR- V_s relationship is unique for both soils and is clearly independent of sample preparation techniques. In the case of Niigata sand, samples were prepared by air pluviation, overconsolidation to the desired V_s , and preshearing to the desired V_s . For the Olneyville Silt, samples were moist-tamped (MMT) at different molding water contents

(corresponding to different degrees of saturation) and compared to an undisturbed sample carved from a block.

Figure 2.31. Comparison between existing the field-based correlations developed by Andrus and Stokoe (2000) and laboratory-based correlations from two silts from RI and three clean sands (Baxter et al. 2008).

To add on this link between laboratory and field behavior using shear wave velocity measurements, Baxter et al. (2008) carried out a series of isotropically consolidated undrained triaxial tests on block samples and reconstituted samples of Providence, RI silts and collected laboratory testing data from Tokimatsu et al. (1986) for Niigata and

Toyoura sands and from Huang et al. (2004) for Mai Lia sand. The purpose of this study was to compare the data with the field based correlation developed by Andrus and Stokoe (2000). They concluded that there is significant underestimation of the liquefaction resistance of the Mai Liao and Niigata sands, whereas there is overestimation of the resistance for the Toyoura sand and Olyneville Silt obtained from Providence, RI. This is illustrated on Figure 2.31. The authors recommend the development of soil-specific correlations when assessing the liquefaction potential of a soil deposit, especially when dealing with unusual or difficult soils (i.e. calcareous soils, residual soils, organic soils, peat, contaminated soils).

A recent study performed by Kayen et al. (2013) combined a global catalog of 422 in situ case histories for shear wave velocity–liquefaction occurrence. Data was collected from different countries such as China, Japan, Taiwan, Greece and the United States of America. They used a Bayesian framework and structural reliability methods to estimate the probability of liquefaction and compared their results to previously proposed and well accepted and commonly used models developed by Andrus and Stokoe (2000) and others. Figure 2.32 shows the new model developed by Kayen et al. (2013) against the in situ based model developed by Andrus and Stokoe (2000) and the laboratory based correlation developed by Zhou and Chen (2007). Kayen et al. (2013) concluded that for a given soil, V_{s1} correlates directly with liquefaction resistance through the e - D_r (void ratio–relative density) relationship. They also believe that soil types with unusual origin, such as calcareous deposits, will correlate different given that these soils will exhibit different and specific e - D_r relationship. This new correlation by Kayen et al. (2013) was developed

for a probability of failure (liquefaction) (P_L) of 15%, $M_w = 7.5$ and an overburden stress (σ'_{vo}) of 1 atm or 100 kPa.

Figure 2.32. Comparison of in situ based models by Andrus and Stokoe (2000) and Kayen et al. (2013) with laboratory based correlation from Zhou and Chen (2007).

Another recent study carried out by Ahmadi and Paydar (2013) evaluated the uniqueness of the CRR- V_{s1} and the applicability of it to all soils. The authors performed a series of stress-controlled isotropically consolidated undrained cyclic triaxial with bender elements tests on two clean sands from Iran, used extensively on geotechnical engineering research and testing. The overall objective of this research was to study whether the CRR- V_{s1} correlation is unique for all soils or not and investigate the necessity of developing soil-

specific correlations. They found that not only the percentage of fines influences and changes the liquefaction resistance, but the type of fines made an impact as well. This strongly supports that the $CRR-V_{s1}$ relationship is soil specific. This is supported by their findings from the laboratory testing and is illustrated on Figure 2.33.

Figure 2.33. New correlation between $CRR-V_{s1}$ for sands with $FC=15\%$ (Ahmadi and Paydar 2013).

CHAPTER 3 FIELD TESTING PROGRAM

3.1 Introduction

A geotechnical site investigation was performed at the study site in Cabo Rojo, Puerto Rico to evaluate the applicability of current field-based liquefaction procedures on calcareous sands. This investigation included geotechnical drilling with hollow stem auger borings, standard penetration tests with energy measurements, seismic cone penetration tests and seismic dilatometer tests. This chapter presents a description of these tests and the results of the site investigation. This site was selected based on the results of previous research performed at the University of Puerto Rico at Mayaguez (UPRM) by Cataño and Pando 2010 and Pando and Sandoval 2010. At the Cabo Rojo site, x-ray diffraction and thermo-gravimetric analyses indicated that the sands there contain more than 90% calcium carbonate (CaCO_3), making it an ideal test site for the proposed study.

3.2 Site Location

The site is located in Punta Ostiones, Cabo Rojo, Puerto Rico and consists of uncemented, very loose, white calcareous sands with carbonate contents greater than 90%. The site is located at latitude 18.094° and longitude 67.194° (Geographic, NAD83 coordinates). This GPS data was collected with a handheld Garmin GPS-map 62S. Figure 3.1 shows a map of PR and the location of the site. Table 3.1 shows the exact coordinates for all the SPT and CPT soundings.

Figure 3.1. (a) Map of Puerto Rico, with the Cabo Rojo area highlighted in red and (b) a zoomed-in picture of the site (photo courtesy of Google maps).

Table 3.1. Coordinates of the SPT and CPT locations at the Cabo Rojo site.

Location of the SPT-borings and CPT-soundings		
ID	Northing (m)	Easting (m)
SPT 1	229068	119383
SPT 2	229100	119392
CPT 1	229076	119384
CPT 2	229100	119391
ID	Latitude (°)	Longitude (°)
SPT 1	18.09444	67.19496
SPT 2	18.09474	67.19488
CPT 1	18.09452	67.19495
CPT 2	18.09473	67.19489

3.3 Standard Penetration Tests with Energy Measurements

Five boreholes with Standard Penetration Tests (SPT) were performed at the site in accordance with ASTM D1586. The SPT's were carried out using a 140 lb automated hammer and a 4-inch diameter standard split spoon sampler with a core catcher to ensure sample recovery. The tests were performed continuously in the sand and in 5-foot increments within the underlying clay layer. The augers used for drilling were 5 feet hollow stem augers filled with water to maintain a water head to try to avoid sand running into the bottom of the borehole. Energy measurements were carried out in the clay layer at depths of 24 to 35 feet. The energy efficiency of the automatic hammer was measured using a PDI pile driver analyzer and energy efficiency measurements ranged from 80% to 83%. The depth of the groundwater table depth ranged from 1.5 to 2.5 feet. The number of blows from the 140 lb hammer applied for every 6-in interval was recorded until the sampler advanced the complete 18-in interval. Based on SPT blow counts, the upper sand layer was found to be extremely loose, with most of the N-values recorded as weight of hammer/rod or with N values below 10. Based on the SPT boring logs, the upper meter consisted of a sandy fill, followed by approximately 4 meters of the Cabo Rojo sand. After the Cabo Rojo sand, more competent soils were found which appeared to be a mix of silts and clays. Soil profiles for both for B1 and B2 were almost identical. SPT results for each boring and the energy calibration report can be found in Appendix 2. Figure 3.2 shows the SPT-rig used for the drilling and sampling and Figures 3.3 and 3.4 shows photographs of the energy measurement system mounted on the automatic hammer.

Figure 3.2. SPT test in progress at the Cabo Rojo, PR site.

Figure 3.3. Dr. Miguel Pando performing the energy measurement test.

Figure 3.4. Illustration of the Pile Driving Analyzer used for the energy measurements of the hammer.

3.4 Cone Penetration Tests with Shear Wave Velocity Measurements

Two seismic Cone Penetration Tests (sCPT) were performed in accordance with ASTM D5778. A Vertek® digital electronic penetrometer with a 60° apex angle, a cone area of 10 cm², a sleeve area of 150 cm² and a maximum tip force of 5 tons was used for the sCPT tests. This cone consists of two strain gauge transducers, with the cone electronics placed directly behind the transducers. The location of the porous element used for the pore pressure measurements is directly behind the friction sleeve, which is called the u_2 position. The seismic standard tri-axial geophone (true DC response) has a range of $\pm 2g$. The sCPT tests at the site included continuous measurements of tip resistance, sleeve resistance, pore water pressure, as well as shear wave velocity measurements at every 3-5 feet intervals. The cone penetrometer was pushed with the SPT-rig at a rate of 2 cm/s.

A local geotechnical company, GeoConsult, carried out both SPT and seismic CPT tests. Results for the sCPT tests can be found in Appendix 2. Figure 3.5 illustrates the setup at the site for measuring the tip and sleeve resistance, pore water pressure and shear wave velocity.

Figure 3.5. Illustration of the setup for the seismic cone penetration tests at the site: (a) cone penetrometer being pushed into the ground with the SPT-rig and (b) shear wave velocity setup.

3.5 Flat Dilatometer Tests with Shear Wave Velocity Measurements

Dilatometer tests results provide measurements of penetration resistance, lateral stress and deformation modulus. The flat dilatometer is a stainless steel blade with a flat, circular, steel membrane mounted flush on one side. The blade dimensions are 95 mm width and 15 mm thick. It has a cutting edge to penetrate the soil with an apex angle between 24° to 32° . The lower tapered section of the tip is 50 mm long and the blade can

safely withstand up to 250 kN of pushing thrust. The circular steel membrane is 60 mm in diameter and 0.2 mm in thickness. Although membranes of 0.25 mm in thickness might be used in soils that might damage or perforate the membrane (adopted from TC-16, 2001).

Figure 3.6. Schematic of the flat plate dilatometer (ISSMGE, TC16, 2001).

This blade is connected to a control unit on the ground surface by a pneumatic-electrical tube, which transmits gas pressure required to expand the membrane and it is supplied by a gas tank connected to the control unit by a pneumatic cable. This control unit consists of the pressure regulator, two pressure gages, a galvanometer and audio-visual buzzer signal, an electrical ground cable connection, and vent valves to control gas flow and vent

the system (Marchetti, 1980). Figure 3.6 shows the general layout of the dilatometer test. The blade was pushed into the ground at a rate of 2 cm/s using the SPT-rig operated by Dr. Ricardo Ramos, from the University of Puerto Rico at Mayaguez.

The test starts once you push the blade into the ground. Right after penetration, the operator inflates the membrane with the control unit and two readings are taken: (1) A-pressure, which is required to just begin to move the membrane against the soil, and (2) B-pressure, which is the pressure required to move the center of the membrane 1.1 mm against the soil. A third reading can be made, depending on the purpose of the site investigation, and it's called the C-reading, which is taken by slowly deflating the membrane after B-pressure is reached. A and C-readings are often called "lift-off" and "closing pressure", respectively. C-readings were not recorded for this study. After recording the B-pressure, the blade is then advanced into the ground at depth increments of 20 cm and the procedure of recording A and B-pressures is repeated at each depth of interest. The DMT tests is suitable for sands, silts and clays but not for gravels. However, the blade is robust enough to cross gravel layers of about 0.5 m in thickness.

Two seismic flat plate dilatometer tests (sDMT) were performed in accordance with ASTM D 6635 for this study. Dr. J. Brian Anderson from Auburn University performed the DMT tests as well as the energy calibration. The sDMT setup was borrowed from the University of North Carolina at Charlotte. Results can be found in Appendix 2. Figure 3.7 shows the flat dilatometer, the control unit and the shear wave velocity measurements setup used at the geotechnical site investigation.

Figure 3.7. (a) Flat Dilatometer control unit and (b) Flat Dilatometer blade and shear wave velocity measurements setup at the Cabo Rojo site.

3.6 Results of Field Testing Program

Figure 3.8 shows the location of all the tests performed at the site in Cabo Rojo, PR. Typical soil profiles with the relevant parameters obtained from the site investigation is illustrated in Figures 3.9 and 3.10. Figures 3.11 through 3.14 show the results obtained for the flat dilatometer tests performed at the Cabo Rojo, PR site.

Figure 3.8. Location of borings 1-5, CPT 1-3 and Dilatometer tests 1 and 2 at the Cabo Rojo, PR site.

Figure 3.9. Final soil profile B1 location.

Figure 3.10. Final soil profile B2 location.

Figure 3.11. Field measurements from the seismic flat dilatometer test, DMT1.

Figure 3.12. DMT1 results for the calcareous sands at the Cabo Rojo, PR study site.

Figure 3.13. Field measurements from the seismic flat dilatometer test, DMT2.

Figure 3.14. DMT2 results for the calcareous sands at the Cabo Rojo, PR study site.

CHAPTER 4 LABORATORY TESTING PROGRAM

4.1 Introduction

The laboratory testing program for this investigation included the following tasks: (1) soil classification and determination of baseline properties for the silica and calcareous sands, (2) mineralogical analyses on the calcareous sand, (3) design and fabrication of a shear wave measurement system in the cyclic direct simple shear apparatus, and (4) evaluation of the cyclic resistance of the selected sands by means of cyclic triaxial tests and cyclic direct simple shear tests.

This work was accomplished in two different laboratories at the University of Rhode Island (URI). The cyclic triaxial tests were carried out at the Kirk Laboratory on the Kingston campus by Dobling (2013) and the rest of the tests were conducted at the Marine Geomechanics Laboratory (MGL) on the Narragansett Bay Campus. This chapter presents a summary of the index properties of the two sands used in this study and detailed descriptions of the design and fabrication of the bender elements end caps, the cyclic simple shear apparatus, and the cyclic triaxial apparatus.

4.2 Index Tests

A series of tests were carried out to classify the two test sands selected for this study (e.g. silica sand and a calcareous sand). To determine its baseline properties tests such as grain size analysis, visual description and specific gravity were carried out. Also, maximum

and minimum dry densities and maximum and minimum void ratio tests were performed on the sands.

Figure 4.1 shows photographs of the two sands selected for this study: (1) a silica sand called Monterey #0/30 and (2) a calcareous sand from southwestern PR, more specifically Puerto Real, Cabo Rojo. The Monterey sand is commercially available from Kleen Blast Abrasives, Danville, CA; it was chosen because of its extensive use in laboratory liquefaction studies in the literature (e.g., Silver 1976, Mulilis 1977, De Alba et al. 1984). The calcareous sand from PR is the same sand used by Sandoval and Pando (2012) in a liquefaction study and was gathered from the near beach surface using shovels and buckets by the author. The Monterey sand has rounded grains that are tan to brown in color whereas the Calcareous sand has angular grains that are ivory to white in color and the presence of shells can be clearly observed.

Figure 4.1. Pictures of the sands tested in this study: (a) Monterey sand and (b) Calcareous sand. The scale is in mm.

Figure 4.2 shows the grain size distribution for the two sands. The gradation curves presented in Figure 4.2 show that both sands exhibit a fairly uniform gradation with grain sizes ranging from 0.25 mm to 0.8 mm and 0.17 mm to 2 mm for the Monterey and Calcareous sands, respectively. Both test sands had no fines contents ($< \#200$). According to the Unified Soil Classification System (ASTM D 2488-00) both soils classify as poorly graded sands (SP). Table 4.1 presents a summary of the main index properties obtained from the soil characterization testing. We can observe from this table that the calcareous sands exhibit very high specific gravity (2.87) and very maximum and minimum high void ratios (>1.0) compared to the Monterey sand.

Figure 4.2. Grain size distribution for the Monterey and Cabo Rojo sand used in this investigation.

Table 4.1. Index properties of the selected sands.

Parameter		Monterey Sand	Cabo Rojo Sand	ASTM Standard
D ₁₀	mm	0.33	0.24	
D ₃₀	mm	0.45	0.3	
D ₅₀	mm	0.55	0.37	ASTM D 422-63
D ₆₀	mm	0.58	0.42	(98)
C _u		1.76	1.75	
C _c		1.06	0.89	
G _s		2.66	2.87	ASTM D 854-06
γ_{min}	kN/m ³	14.4	10.2	ASTM D 4254-00
e _{max}		0.808	1.75	
γ_{max}	kN/m ³	16.4	12	ASTM D 4253-00
e _{min}		0.589	1.34	

4.3 Mineralogy of the Cabo Rojo Sand

The mineralogy of the calcareous sand was evaluated by means of X-Ray Diffraction (XRD) analysis. XRD is a qualitative mineralogical characterization used to determine the structure and arrangement of a material. A diffractometer was used in the Cardace Laboratory at the University of Rhode Island. Tests were performed on the Calcareous soils collected in Cabo Rojo, Puerto Rico. Soils collected during the geotechnical site investigation (Boring-1 and Boring-2; see Chapter 3) were also analyzed for bulk carbonate content. As expected, all of the diffractograms reveal a predominance of carbonate materials such as calcite and aragonite at $2\Theta = 29$ to 30° . A summary table of the soil composition can be found in Table 4.2. The results of all of the XRD tests can be found in the Appendix 1.

Table 4.2. Summary of results for the XRD Mineralogy tests performed at URI.

ID ¹	CALCITE	ARAGONITE	MAGNESITE	SIDERITE	DOLOMITE
Bucket	Yes	Maybe	No	No	No
B1S1	Yes	Yes	No	No	Maybe
B1S2	Maybe	Yes	No	Maybe	Maybe
B1S3	Maybe	Yes	No	Maybe	No
B1S4	Yes	Yes	Maybe	No	No
B1S5	Yes	Yes	Maybe	No	Maybe
B1S7	Yes	Yes	No	No	No
B2S1	Yes	Yes	No	No	No
B2S6	Yes	Yes	No	No	No
B2S8	Yes	Yes	No	No	No

¹ Bucket = surficial samples; B#S# = Boring number/sample number

4.4 Fabrication of the End Caps with Bender Elements for Shear Wave Velocity Measurements

A major task for this investigation was the design and fabrication of end caps with bender elements for the cyclic direct simple shear test (CDSS) in order to allow measurements of shear wave velocity. A similar bender element system was first developed at the University of Rhode Island by Hanchar (2006), which was successfully used by Bradshaw (2006), Sharma (2011), Dobling (2013), Baffer (2013) and Guadalupe (2013). The design and fabrication of a shear wave velocity measurement system for the CDSS was completely governed by the location of the bender elements and height constraints of the apparatus. Also, these end caps were designed and built to be user friendly, easily portable for one person, provide easy access to the benders to be replaced when needed, and most importantly to reduce all grounding issues and electrical cross talk. The shear wave velocity system was jointly designed and constructed by the author and Brian Baffer (Baffer 2013).

The selection of the material for the end caps was based on several factors: (1) corrosion resistance, (2) modulus of elasticity, and (3) electrical conductivity. Three different materials were considered: (1) PVC, (2) Steel and (3) Brass. PVC is ideal because of its resistance to corrosion and ease to machine with but it is nonconductive therefore it wasn't the best candidate. A36 Steel would have worked from a conductivity standpoint but it would have been easily corroded given that soil samples might contain salt. Brass was the material selected primarily because it is easy to machine with, has high stiffness, and good electrical conductivity.

Figures 4.3 and 4.4 shows the final design of the top and bottom end caps. A significant design consideration of the CDSS is the height restriction. The bender elements have to be fully enclosed within the end caps, therefore only approximately 40 mm of height was left to work with. The top cap has a 19.4 mm diameter hole in the center (not shown) so that it can be tightly screwed in onto the vertical rod. It also has one drainage hole, and the slot for the bender element. The top cap was made with tight tolerances between the center hole and the vertical rod to minimize rocking of the top cap during shear. The bottom end cap, which is not as tall as the top cap because it is mounted on a steel plate, has one drainage hole, two screw holes for the bottom cap to be rigidly connected to the steel base, and the slot for the bender element. The bottom steel plate which is connected to the bottom end cap carries the weight of the Teflon stacked-rings that provide lateral confinement to the sand specimens and allows them to move side-to-side with the sample without causing any significant frictional resistance. Figure 4.5 shows the finalized brass top and bottom end caps before the installation of the bender elements.

Figure 4.3. Final design for the bottom end cap.

Figure 4.4. Final design for the top end cap.

Figure 4.5. Final brass top and bottom caps before bender element installation.

A significant challenge when using stacked rings in CDSS tests is the sample preparation. There is no easy or obvious way to stretch a membrane around a sand sample. To accomplish this, a specialized split-mold housing developed by Dr. Ravi Sharma (2011) at URI was modified to cater to the specific needs of the CDSS and the new shear wave velocity system. This mold is shown in Figure 4.6. The split mold is sealed against the bottom end cap with electrical tape and vacuum grease, the membrane is stretched over it and a small vacuum is applied through the stacked rings. The split mold is removed once

the sample is pluviated, the top cap has been carefully put on, and some vertical stress has been applied to the sample.

Figure 4.6. Split PVD mold used as a membrane stretcher and guide for the top cap into place onto the soil sample (Baffer 2013).

4.4.1 Installation of the Bender Elements

The bender elements were mounted into the end caps with only a few millimeters extending into the sample. The length of the benders that was going to be embedded in the sample was selected based on previous work carried out at MGL by Dr. Christopher Baxter and Brian Baffer on DSS samples similar in height as the cyclic DSS (~25.4 mm or 1 in). Piezosystems, Inc. of Cambridge, MA, manufactured the bender elements used for this study. Table 4.3 summarizes the material properties and Figure 4.7 shows a schematic of the bender elements with their dimensions. The 303 bending actuator is a

standard item for Piezosystems, Inc. and comes already with the wiring bracket attached. The inclusion of a standard quick-mount bracket offers additional protection by increasing the stiffness for the electrical connections, decreasing the electrical noise by proper insulation, and by having a bleed resistor for the unpredictable electrical spikes (Piezosystems, Inc., 2008).

Table 4.3. Summary of the material properties for the bender elements (Piezosystems Inc.).

Part Number	(-)	Q220-A4-303YB
Piezo Material	(-)	5A4E
Weight	(g)	2.3
Stiffness	(N/m)	760
Capacitance	(nF)	52
Rated Voltage	(V)	± 90
Resonant Frequency	(Hz)	275
Free Deflection	(μm)	± 315
Blocked Force	(N)	± 0.24

Figure 4.7. Illustration of the bender element and schematic of the 303 bender actuator in inches (mm in parentheses) (Piezosystems Inc.).

The bender elements consist of a piezoelectric ceramic, which is a material that deforms either mechanically when a voltage drop is placed across it and vice versa (i.e. a voltage is generated as it deforms). These ceramics are typically made of lead zirconate titanate, barium titanate or lead titanate.

Once the element is excited by applying a voltage, compression waves are generated by producing a voltage drop parallel to the polarization of the metal component, as shown in Figure 4.8. This creates a change in thickness of the ceramic element. But in order to be able to generate and measure shear waves, two piezoelectric ceramics with opposite polarization are mounted onto a metal shim. This causes one side to lengthen and the other to shorten when a voltage is applied, which results in a bending motion of the element. Elements in this configuration are called “bender elements” and have been proven to be excellent for generating and measuring shear waves (see Figure 4.9).

Figure 4.8. Application of voltage across a piezoceramic to cause thickening of the element (Piezo Systems).

Figure 4.9. Bender elements wired in series and parallel (adapted from Piezo Systems).

The metal component of the bender elements can be wired in series or in parallel. Wiring the metal component in parallel will cause the element to generate more bending for a given voltage, as illustrated in Figure 4.9. Typically this configuration is used as a transmitter of shear waves. Conversely, wiring the metal component in series will cause the element to generate a larger electrical signal for a given mechanical movement. Bender elements wired in series are usually used as receivers. An example of a typical transmitted and received signal from bender elements is shown in Figure 4.10. For purposes of this study, the receiving and transmitting bender elements were wired in series and parallel, respectively, with the top cap being the transmitter and the bottom cap being the receiver. The parallel connection does not split voltage therefore both of the ceramic plates are influenced by the same amount of voltage, making it ideal for a transmitter. On the contrary, the series connection is influenced at the same time across both ceramic layers; it is better suited as a receiver. Wiring the bender elements in this fashion has been proven to be successful by several other researchers such as Hanchar (2006), Landon (2007), and Deniz (2008). Therefore the factory wiring of the bender element was removed and a Belden 8240 R6 58/U coaxial cable was attached with solder

in order to make the benders ready to transmit and receive signals and allowed the connection between the signal generator and receiver to the bender elements to be as clean as possible, minimizing any potential magnetic field interference. This is shown in Figure 4.11. At the end of the coaxial cables are factory installed BNC connectors. These BNC connectors allowed for a clear connection to the equipment and the bender elements. This set up produced a very clean signal with little electrical noise.

Figure 4.10. Examples of input and output readings of shear waves generated by and received from bender elements (Brignoli et al. 1996).

The bender elements were encapsulated and secured in the brass end caps using a two part electrical resin, Scotchcast 5 Resin, manufactured by 3M® followed by a couple of thin layers of polyurethane, as suggested by Hanchar (2006). This resin has several

advantages: (1) it is an electrical epoxy considered to be rigid and fully insulating, (2) any stray of current cannot jump from the bender element to the brass cap and vice versa, which will reduce the chances of the signal being altered or heavily disturbed by electrical noise or grounding issues, and (3) if the bender element needs to be replaced, it can be easily removed without damaging the end cap or the piezoceramic by applying heat (e.g using a commercial hair blow dryer). Setting the epoxy on the bender-brass surface and creating a uniform coating around the bender is not an easy task. A mold was used to hold the bender in place from the wire end and to prevent any contact between the ceramic and the brass. Another mold is then needed to pour the epoxy around the bender element and to keep it contained within the area of interest (end cap) while it sets, which is typically 1 hour. Once it settles, a soldering iron was used to trim/remove the excess epoxy into a nice box shape that surrounds the whole bender element. Figure 4.12 shows the end caps, with the encapsulated bender elements and the friction plastic inserts. Mr. Robin Freeland at URI designed these plastic inserts. These plastic inserts were used instead of the traditional sand-based porous stones and the design was based on the assumption that these inserts were going to provide less friction and help prevent any potential slipping of the sand particles during shear.

Figure 4.11. Soldering of the bender element in series.

Figure 4.12. Top and bottom caps with plastic friction inserts and the bender elements covered in the two-part epoxy.

4.4.2 *Shear Wave Velocity Measurement System*

The end caps described in the previous section were used to measure the shear wave velocity during consolidation for the cyclic direct simple shear tests. There is significant experience in the measurement of shear wave velocity of soils at URI (e.g., Baxter 1999; Baxter and Mitchell 2004; Bradshaw 2006; Hanchar 2006; Baxter et al. 2008; Sharma 2011; Dobling 2013; Baffer 2013; Guadalupe 2013) and this experience was used and applied to the design of the bender elements for shorter specimens. The interpretation of the bender element data collected in a short CDSS sample to determine shear wave velocity can be quite complicated. This is because shear waves are affected by numerous factors such as: (1) wave form generated by the transmitter, (2) frequency of the signal, (3) separation distance, and (4) the applied voltage. These factors are discuss

Effect of Transmitted Wave Form - Leong et al. (2005) carried out an investigation on using bender elements to measure shear wave velocity. More specifically they looked into the effects of transmitting different waveforms, such as, a square wave versus a sinusoidal wave. Figure 4.13 shows the processed received signal for both waveforms on three different soil specimens. It can be observed that the square wave caused greater ambiguity in arrival time when compared to the sinusoidal wave. They concluded that a square wave does not resemble the original transmitted signal, which agrees with a similar observation made by Jovicic et al. (1996) and Blewett et al. (2000) and that the received signal for the square wave have more distortion at the beginning of the signal.

A similar study was carried out by Lee and Santamarina (2005), which evaluated both the type of transmitted waveform and frequency content of the received signal. They

determined that the bender element response is enhanced when the frequency of the input sinusoidal signal approaches the resonant frequency of the bender element-soil system. If the resonant frequency is unknown, or if it is expected to change suddenly during tests, a step input signal is advantageous. This is because a step signal includes all frequencies and a clear response is measured regardless of the soil stiffness. Figure 4.14 illustrates a comparison of different waveforms and frequencies at a resonant frequency of 3.6 kHz. We can observe from this figure that at larger frequencies the received signal is distorted, but as the frequency is lowered the signal appears better until it hits the resonant frequency of the bender elements-soil system.

Figure 4.13. Effect of wave form on received signals (Leong et al. 2005).

Figure 4.14. Bender element installation: (a) sequence frequency response functions; and (b) input and output signals. Tip-to-tip distance: $L = 105$ mm, cantilever length: 6.5 mm, $V_s = 93$ m/s. Resonant frequency of bender element $f_r = 3.6$ kHz, $L/\lambda = 4$ (Lee and Santamarina 2005).

For purposes of this investigation, different input signals (i.e. sine wave versus square wave) at different frequencies were evaluated as part of the validation of the new bender element system for the cyclic direct simple shear test. The following tables show a summary of the results obtained for both Monterey and Cabo Rojo sand specimens prepared at very similar heights (i.e. similar void ratios). These tables include: test number, frequency, height and change in height at a vertical effective stress of 100 kPa, the measured time delay and the calculated shear wave velocity. For the Monterey sand, it can be observed from Table 4.4 that the time delay ranged from 62 μ s to 67 μ s. This results in a standard deviation equal to 1.75. If we look at it in terms of the shear wave

velocities, we can see from this table that the V_s ranged from 243 m/s to 267 m/s, with an average of 254 m/s. This results in a standard deviation of 7.7. The calculated value for the shear wave velocity does not only depends on the measured time delay between the transmitted and received signal but on the tip-to-tip distance between the bender elements, which will ultimately depend on the final height, after consolidation, of the sample. Given the difficult task of making identical samples, it is expected for the shear waves not be identical, but within a reasonable range, as the one observed for the Monterey sands and a sine wave. On the other hand, Table 4.5 shows the results, also for Monterey sand samples, for a square wave at 30 Hz. The frequency of the square wave was not varied because this type of wave carries a lot of frequency content. We can observe from this table that the range for the time delay ranged from 62 μ s to 66 μ s (standard deviation equal to 1.48). The shear wave velocities corresponding to these travel time values ranged from 244 m/s to 261 m/s, with an average value equal to 252 m/s. This results in a standard deviation of 6.5. Therefore, the difference obtained for the shear wave velocities calculated with a sine wave (average value = 254 m/s) versus a square wave (average value = 252 m/s) was 2 μ s.

For the calcareous sands, we can observe from Table Table 4.7 that the time delay for a sine waveform at frequencies of 10, 15 and 20 kHz ranged from 65 μ s to 75 μ s (standard deviation equal to 2.88). The shear wave velocities corresponding to these travel times ranged from 218 m/s to 257 m/s, with an average value of 236 m/s. This results in a standard deviation of 11.02. For a square waveform, the travel times ranged from 64 μ s to 71 μ s (standard deviation equal to 2.74). The shear wave velocities corresponding to

these travel times ranged from 232 m/s to 257 m/s, with an average value of 243 m/s. This results in a standard deviation of 10.11. Therefore, the difference obtained for the shear wave velocities calculated with a sine wave (average value = 236 m/s) versus a square wave (average value = 243 m/s) was 7 μ s. Overall, this shows excellent agreement if we take into consideration the difference factors affecting this value such as waveform, frequency and sample height after consolidation.

Table 4.4. Variability in the measurement of the travel time for Monterey samples consolidated at a vertical effective stress of 100 kPa and a sine wave.

Monterey Sand, $\sigma_c = 100$ kPa, Sine Wave, 8 μ s time delay					
Test # (-)	f (kHz)	h (mm)	Δh (mm)	Δt (μ s)	V_s (m/s)
113	15	25.51	0.497	67	244
	20			65	252
	25			64	257
115	15	25.37	0.548	67	243
	20			65	251
	25			64	256
116	15	25.34	0.436	66	249
	20			66	249
	25			67	245
117	15	25.2	0.427	66	249
	20			64	258
	25			62	267
121	15	25.2	0.503	63	261
	20			63	261
	25			62	266

Table 4.5. Variability in the measurement of the travel time for Monterey samples consolidated at a vertical effective stress of 100 kPa and a square wave.

Monterey Sand, $\sigma_c = 100$ kPa, Square Wave, 7 μ s time delay					
Test # (-)	f (Hz)	h (mm)	Δh (mm)	Δt (μ s)	V_s (m/s)
113	30	25.51	0.497	66	244
115		25.37	0.548	65	247
116		25.34	0.436	64	253
117		25.2	0.427	64	253
121		25.2	0.503	62	261

Table 4.6. Variability in the measurement of the travel time for Cabo Rojo samples consolidated at a vertical effective stress of 100 kPa and a sine wave.

Cabo Rojo Sand, $\sigma_c = 100$ kPa, Sine Wave, 8 μ s time delay					
Test # (-)	f (kHz)	h (mm)	Δh (mm)	Δt (μ s)	V_s (m/s)
1	10	25.42	0.721	71	229
	15			71	229
	20			68	241
2	10	25.42	0.586	75	218
	15			73	224
	20			70	235
7	10	25.42	0.546	73	225
	15			68	244
	20			65	257
10	10	25.42	0.757	71	229
	15			66	249
	20			66	249
13	10	25.42	0.542	71	232
	15			71	232
	20			68	244

Table 4.7. Variability in the measurement of the travel time for Cano Rojo samples consolidated at a vertical effective stress of 100 kPa and a square wave.

Cabo Rojo Sand, $\sigma_c = 100$ kPa, Square Wave, 7 μ s time delay					
Test # (-)	f (Hz)	h (mm)	Δh (mm)	Δt (μ s)	V_s (m/s)
1	30	25.42	0.721	68	241
2		25.42	0.586	70	235
7		25.42	0.546	67	248
10		25.42	0.757	64	257
13		25.42	0.542	71	232

Wave Form Travel Distance - The distance traveled from the transmitter to the receiver is usually measured from tip to tip of each bender element. This travel distance also affects the measured shear wave velocity. Tip-to-tip distance has been adopted by numerous researchers (Lee and Santamarina 2005; Leong et al. 2005; Brignoli et al. 2006, Baxter et al. 2008) and it is supported by data gathered with bender elements and resonant column tests (Dyvik and Madshus 1985). At greater tip-to-tip distances, more attenuation of the received signal will occur. Conversely, if the bender elements are too close, near field effects might hamper interpretation of the received signal. Near field effects can be easily picked-up where the first deflection of the received signal is reversed. This effect is attributed to a component of the shear wave that travels at the velocity of a compression wave. Salinero et al. (1986) recommended to keep the ratio of tip-to-tip distance (L) and wavelength (λ) greater than approximately 2 to avoid near field effects. Leong et al. (2005) more recent study suggested to increase the minimum L/λ ratio to 3.33 to ensure the shear wave velocity measurements were not made in the near field.

First Arrival - Another controversial aspect of bender element signal interpretation is the selection of the first arrival of shear wave velocity signal. Researchers have discussed different methods to obtain the most accurate shear wave velocity measurements from detection of the first arrival such as: (1) peak to peak, (2) first zero crossing, and (3) first take off. These methods are all time domain methods, meaning they are direct measurements based on plots of the electrical signals versus time. Lee and Santamarina (2005) illustrate the difference between these approaches, as shown in Figure 4.15.

Figure 4.15. Typical wave signal within near field ($L/\lambda=1$): (A) first deflection, (B) first bump maximum, (C) zero after first bump, and (D) major first peak (Lee and Santamarina, 2005).

Suggested criteria and recommendations vary depending on installation, application, and input signal (Abbis 1981; Dyvik and Madshus 1985; Mancuso et al. 1989, Fam and Santamarina 1995; Viggiani and Atkinson 1995a, b; Jovicic et al. 1996; Jovicic and Coop 1997; Santamarina and Fam 1997; Blewett et al 1999; Lohani et al 1999; Kawaguchi et al. 2001). Alternative signal processing methods have been a topic of research over the

course of the years to avoid picking a travel time. These include cross correlation and frequency domain analyses. These methods involve analyzing the spectral breakdown of the signals and comparing phase shifts of the components (e.g., Viggiani and Atkinson 1995, Brocanelli and Rinaldi 1998; Arroyo 2001). However, it is important to note that no method is yet proven to be superior to the others, as most recently reported by Yamashita et al. (2009) in an international parallel bender element tests exercise involving 23 institutions from 11 countries (Chan 2010).

Electromagnetic Cross-talk - Another phenomenon that must be taken into consideration when measuring shear waves is cross-talk. This will only be present on saturated samples and can distort the received signal. Due to the induced electromagnetic field caused by the bender element, the received bender element will register a cross-talk signal that can be misinterpreted for the real signal as shown in Figure 4.1716. This electromagnetic field is essentially a ground, which energizes the cross-talk signal. Lee and Deniz (2005) suggested using a polyurethane coat for all the bender elements and a conductive paint for the bender element wired in series to prevent cross-talk. However, at the Marine Geomechanics Laboratory (MGL) at the University of Rhode Island (URI) using the combination of the polyurethane and conductive paint has not proven to be successful. Through trial and error, it has been determined that the most effective methods is to use a 3M Scotchcast 5 Resin followed by a couple of thin layers of polyurethane (Hanchar, 2006). This resin is a two part electrical epoxy that is considered to be rigid once cured, which makes it ideal for encapsulating the bender elements (3M Company®). With the

combination of the resin and polyurethane any induced cross-talk should appear smaller in size, if even detectable, compared to the desired signal.

Figure 4.16. Cross talk effects (Lee and Santamarina 2005).

Figure 4.17 shows the electronics (i.e., function generator, amplifier/analog filter, oscilloscope) used for the measurement of the travel time between the transmitted and received signal, which is the key when calculating shear wave velocity. For this study two different waveforms were selected: (1) sinusoidal wave and (2) square wave, both at an amplitude of 20 Volts peak to peak, as illustrated in Figure 4.18. This figure corresponds to a test using a Cabo Rojo sand specimen under 100 kPa vertical effective consolidation stress. Also, several frequencies were evaluated for both of the selected waveforms. Given that the CDSS specimens are short in height, frequency ranges that would give clear signals with little or no amplification were in the range of 15-25 kHz for the sinusoidal wave input and 30 Hz for the rectangular wave input. Traveled distance was selected as tip-to-tip distance. First arrival of the shear wave was selected as the first

zero crossing. Measured signals were very clear, with no evidence of near field effects or cross-talk. All of these selected criteria were chosen by trial and error and by ensuring good grounding and fabrication of the end caps and bender elements. The system delay was measured and removed by touching the bender elements and measuring the time delay directly (Baxter et al. 2008).

Figure 4.17. Shear wave velocity setup at the MGL at URI.

Figure 4.18. Illustration of the two waveforms (sinusoidal wave versus rectangular wave) used for this investigation (25.4 mm in height dry pluviated Cabo Rojo sand sample).

4.5 Cyclic Direct Simple Shear Apparatus

The cyclic resistance of the Monterey and Cabo Rojo sand was assessed by means of K_0 constant volume cyclic direct simple shear tests. There is no laboratory test method capable of fully reproducing all in situ conditions accurately but the CDSS is the one that represents level ground conditions most closely. The apparatus was manufactured by the Geocomp Corporation and is shown in Figure 4.19.

Figure 4.19. Shear Track II system for performing CDSS tests (Geocomp 2011).

The ShearTrac II system is a fully automated load frame capable of applying normal stresses for consolidation and horizontal shear stresses either for the application of static shear stress, monotonic loading, or cyclic loading. Deformation in the vertical and horizontal directions is measured using displacement transducers. Consolidation occurs under approximately K_o conditions through the use of Teflon-coated stacked rings surrounding the samples.

The CDSS test in the Geocomp system is not sheared under “truly undrained” conditions but rather under “constant volume” conditions, meaning that the height of the sample is kept constant during the shear phase and there is no direct measurement of the excess pore water pressure. In fact, in this study the soil samples are not saturated and are prepared as either dry or moist samples. In a constant volume direct simple shear test, it is assumed that the change in applied vertical stress as the specimen height is maintained constant during shear is equal to the excess pore pressure which would have been measured in a truly undrained test with constant total vertical stress (Bjerrum and Landva 1966). Other researchers have verified this pore pressure assumption. For example, Vuceti and Lacasse (1983) carried out a series of triaxial tests on normally consolidated Drammen clay and simple shear tests on the same clay at different OCR's. The anisotropically consolidated clay specimens were sheared under both truly undrained conditions and constant volume conditions and the pore pressure-axial strain curves for undrained and constant volume tests were identical in compression and in extension. Another example is the work performed by Dyvik et al. (1987) on normally consolidated Drammen clay. They carried out a series of truly undrained direct simple

shear tests on an NGI-type device that allows for pore pressure measurements. Results were compared with those of traditional constant volume direct simple shear tests. The stress-strain behavior and effective stress plots obtained with the two different methods were identical, which confirms that the change in vertical stress required to maintain a constant volume throughout the test are equal to the measured pore pressures in a truly undrained direct simple shear test. To the knowledge of the author, no findings have been reported about the validity of the constant-volume versus true undrained conditions assumption for cyclic direct simple shear tests on sand specimens.

In a direct simple shear test, the stacked-rings prohibit lateral deformation on the soil sample, so only the sample height must be kept constant to achieve constant volume conditions. This can be done in two different ways. One way would be physically fixing the specimen height with a rigid cross member (and measuring the change in vertical load) as suggested by Finn and Vaid (1977), or by adjusting the vertical load to maintain a constant height.

4.6 Cyclic Triaxial Apparatus

As stated previously, the cyclic triaxial tests were carried out at the Kirk Laboratory at the URI Main Campus with an Instron® Load Frame capable of applying cyclic loads. This load frame is controlled by a software package called Instron Wavematrix, which has an auto tuning wizard function that is used to determine the Proportional, Integral and Derivative (PID) settings required for sample testing. A detailed description of this system and the laboratory testing program can be found in Dobling (2013).

Monterey and Cabo Rojo sand samples were prepared at different densities by using the Modified Moist Tamping procedure developed at URI by Bradshaw and Baxter (2006). This sample preparation method is a modification of Ladd's (1976) moist tamping technique and is based on constant energy rather than constant height. Samples were prepared at a saturation of 55% and were isotropically consolidated up to 100 kPa, with shear wave velocity measurements made at the end of consolidation. The bender element system used for the cyclic triaxial testing was developed by Guadalupe (2013) and built by Mr. Fred Pease of the Ocean Engineering department. Shear wave velocity travel time was determined by peak to peak as the arrival time of the shear wave, as suggested by Lee and Santamarina (2005) and Kumar and Madhusudhan (2010). The system delay was removed by touching the bender elements and measuring the time delay. Samples were sheared under stress-controlled conditions at a frequency of 1 Hz at different cyclic stress ratios.

CHAPTER 5 RESULTS OF LABORATORY TESTING

5.1 Introduction

This chapter presents the results of the laboratory testing program. Testing included cyclic direct simple shear (CDSS) and cyclic triaxial tests with shear wave velocity measurements on samples of Monterey and Cabo Rojo sands. Procedures are described in Chapter 4. This chapter focuses primarily on the cyclic resistance on the Cabo Rojo sand and specifically the relationship between cyclic resistance and shear wave velocity. This chapter also includes the following:

- an evaluation of particle crushing of the Cabo Rojo sand during shear;
- a comparison of cyclic simple shear and cyclic triaxial results;
- an evaluation of the effect of preparing samples by dry pluviation and moist tamping on cyclic resistance; and
- an evaluation of varying carbonate content on cyclic resistance;

This chapter concludes with a brief summary and conclusions drawn from this part of the study.

5.2 Cyclic Resistance of the Cabo Rojo Sand

The cyclic resistance of the Cabo Rojo sand was evaluated by means of a series of K_o constant volume (undrained) cyclic direct simple shear tests and isotropically

consolidated undrained cyclic triaxial tests also with shear wave velocity measurements. Results are presented in the following subsections.

5.2.1 Shear wave velocity measurements

The shear wave velocity was determined in the time domain by identifying the “first deflection” of the shear wave (Lee and Santamarina, 2005). Measurements of the time difference between the first peaks of the transmitted and received signals was also evaluated and the arrival times were typically with 1 μ s of the “first deflection” approach. The system delay was measured by putting both bender elements together (transmitter and receiver) and that time delay was subtracted from the measured signals in soils.

Figure 5.1. Illustration of the choice of first arrival time for an input sine wave of 20 Volts peak-to-peak, $f = 20$ kHz and σ'_v 100 kPa. [Test ID: CYC_2014_CS_G1D_02, Dense Cabo Rojo sand specimen].

Two different waveforms were used to generate shear waves: (1) a single sine wave and (2) a square wave. Both signals were generated with an amplitude of 20 Volts peak-to-peak. The frequency of the transmitted sine wave was varied between 15 and 25 kHz,

whereas the frequency of the square wave was 30 Hz. Frequencies were selected by trial-and-error and from experience with taller samples (i.e. triaxial specimens). Based on the amplitude and clarity of the received signals, a single sine wave at a frequency of 20 kHz was used for this study. A typical transmitted signal at a frequency of 20 kHz and the resulting received signal is shown in Figure 5.1.

Samples were prepared and consolidated in a split mold with a small vacuum of approximately 30 kPa to stretch the membrane against stacked rings. This vacuum was removed at a vertical consolidation stress of 80 kPa and the shear waves velocity was measured before and after its removal. Snapshots before and after vacuum removal were taken to ensure there was no change in travel time. This is shown in Figure 5.2 using a square wave as the transmitted signal.

Figure 5.2. First arrival time (a) before and (b) after vacuum removal at an effective vertical stress of 80 kPa. [Test ID: CYC_2014_114_MS, Loose, Monterey Specimen].

The shear wave velocity was measured at the end of consolidation using a Tektronic TDS 2014B four-channel digital oscilloscope, as shown in Figure 4.17. Shear waves were measured by using the cursor function on the oscilloscope and measuring the travel time from the first deflection of the input signal to the first deflection of the received signal.

Shear waves were calculated using the tip-to-tip distance from the top of the bottom bender to the bottom of the top bender element using the following equation:

$$V_s = \frac{L}{\Delta t} \quad (5.1)$$

where Δt is the travel time (with the time delay taken into account), L is the tip-to-tip distance between bender elements, corrected for change in height during consolidation. This change in height was subtracted from the tip-to-tip distance.

To validate our new bender element system, measured values of shear wave velocity were compared to resonant column tests performed by Cataño and Pando (2010) as well as tests performed at the University of Texas at Austin (UTA) specifically for this study on the Cabo Rojo sand. The tests carried out by Cataño and Pando (2010) were performed at the Geotechnical Engineering Laboratory of the University of Notre Dame (UND). The tests specimens were prepared at loose ($D_r = 21\%$) and dense ($D_r = 91\%$) conditions using the dry pluviation technique. Samples were consolidated at effective confining pressures of 50, 100, 300 and 500 kPa. The tests at the University of Texas, at Austin were prepared to different relative densities (e.g. $D_r = 42.3\%$, 53.8% , 75.8% , 78.2% and 78.7%) using the moist tamping technique. Samples were consolidated at effective confining pressures that varied from 16 kPa to 496 kPa. For comparison purposes, the following figures only show the results for the loosest and densest samples.

Figure 5.3. Comparison of V_s measurement for the Cabo Rojo sand performed by Cataño and Pando (2010), Resonant Column Tests performed at University of Texas and CDSS performed at URI for this investigation.

Figure 5.3 shows a comparison of shear wave velocity measurements for the Cabo Rojo sand obtained by means of resonant column and bender element testing. Results are plotted in a log-log scale. The lowest boundary can be defined by the loose dry pluviated samples and shear wave velocities measured with bender elements. These results follow the trend exhibited by the loose dry pluviated samples and shear wave velocities measured with the resonant column device at the University of Notre Dame. This is not surprising given the fact that samples were prepared using the same sample preparation method and the specimens are loose. The highest boundary is defined by the moist tamped medium-dense and dense specimens and shear wave velocities measured with the resonant column device at the University of Texas at Austin. This behavior is also expected given that moist tamping is known to produce specimens with stronger fabrics or soil structure.

Figure 5.4 shows a comparison of shear wave velocity measurements by means of resonant column testing and bender elements on both cyclic triaxial and cyclic direct simple shear tests and samples prepared by dry pluviation, moist tamping and modified moist tamping. As shown in this figure, the shear waves measured by means of bender element testing on the cyclic triaxial apparatus plots to the right of the trends exhibited for both the resonant column test specimens prepared either by dry pluviation or moist tamping and the cyclic direct simple shear test specimens prepared using the dry pluviation method. This behavior might be attributed to the fact that the actual void ratio (i.e. relative density) at the end of consolidation is unknown and seems to have been calculated erratically (i.e. wrong measurement of the volume change at the end of consolidation from the pressure panel used to applied the cell and sample pressure). Therefore an accurate comparison cannot be made. The direct simple shear tests were consolidated to a vertical effective stress of 100 kPa, which corresponds to a mean effective stress of 57 kPa assuming a value of K_o equal to 0.36. At the same stress level (~60 kPa) it can be observed that the shear wave velocities obtained for the dry pluviated specimens in the University of Notre Dame resonant column device and the cyclic simple shear apparatus ranged from 241 m/s to 257 m/s, for a range of relative densities between 40% to 91%. This seems to be reasonable given that the samples are very different in terms of sample height and boundary conditions (i.e. isotropic consolidation versus K_o -consolidation). Also, shear wave velocity seems to be more sensitive to changes in effective stress rather than void ratio. The spread in the shear wave velocities for the modified moist tamped specimens tested under cyclic direct simple shear conditions seems to be much higher ($V_s = 203$ m/s at $D_r = 67.6\%$ and $V_s = 305$ m/s at $D_r = 104.9\%$).

Figure 5.4. (a) Comparison of V_s measurement for the Cabo Rojo sand at different relative densities under different boundary conditions up to 400 kPa of vertical effective stress and (b) 200 kPa of vertical effective stress.

5.2.2 *Cyclic direct simple shear tests*

This section presents the cyclic simple shear results on samples of Cabo Rojo and Monterey sand. Samples were prepared using two different methods: dry pluviation (DP) and modified moist tamping (MMT) in which the molding water content corresponded to a degree of saturation of 55%. Sample dimensions were 63.5 mm in diameter and approximately 25.4 mm in height. This diameter measurement included the correction for the membrane thickness used to contain the sample. All samples were subjected to a vertical effective consolidation stress of 100 kPa and the shear wave velocity was measured at the end of consolidation. Samples were sheared under constant volume conditions and subjected to a sinusoidal cyclic load at a frequency of 0.5 Hz. Liquefaction was defined at a double amplitude strain of 3.75%.

Cabo Rojo Sand – The results of a single test for a loose sample of Cabo Rojo sand sample are shown in Figure 5.5. The figures on the left shows the applied stress, shear strain and pore pressure ratio versus the number of cycles of loading. The figures on the right show the stress-strain behavior, the reduction in vertical stress in σ' - τ space, and the reduction in vertical effective stress versus shear strain. Failure was defined as when the double amplitude strain reached 3.75%. For this particular test, the sample reached failure after 38 cycles of loading. Figure 5.6 shows typical test results for a dense sample of Cabo Rojo sand prepared by dry pluviation.

Figure 5.5. Typical results of a CDSS test on a loose sample of Cabo Rojo sand prepared by dry pluviation.

Figure 5.6. Typical results of a CDSS test on a dense sample of Cabo Rojo sand prepared by dry pluviation.

Results for the all the cyclic direct simple shear tests performed on the Cabo Rojo sand are summarized in Tables 5.1 and 5.2. These tables shows the void ratios before (e_i) and after consolidation (e_c), relative density at the end of consolidation (D_{rc}), shear wave velocity (V_s) at the end of consolidation ($\sigma_{vcon} = 100$ kPa), applied cyclic stress ratio (CSR), and the number of cycles to cause liquefaction (N_f).

Table 5.1. Summary of results for the cyclic direct simple shear tests carried out on the Cabo Rojo sands, prepared using the dry pluviation technique

<i>Loose, Dry Pluviation</i>						
Test No. (-)	e_i (-)	e_c (-)	D_{rc} (%)	N_f (-)	CSR @ $DA=3.75\%$ ⁽¹⁾ (-)	V_s (m/s)
1	1.65	1.591	38.8	14	0.136	248
2	1.65	1.582	41.0	19	0.12	234
3	1.66	1.575	42.7	38	0.099	245
4	1.66	1.587	39.8	243	0.07	235
<i>average</i>		1.584				241
<i>stdev</i>		0.007				7.05
<i>Dense, Dry Pluviation</i>						
1	1.42	1.38	90.24	19	0.138	257
2	1.44	1.407	83.66	97	0.119	257
4	1.43	1.413	82.20	13	0.156	259
5	1.41	1.388	88.29	289	0.1	254
<i>average</i>		1.397				257
<i>stdev</i>		0.016				2.06
<i>Very Dense, Dry Pluviation</i>						
1	1.311	1.224	128.3	26	0.139	243
2	1.316	1.259	119.8	12	0.159	257
3	1.318	1.269	117.3	15	0.168	250
4	1.321	1.269	117.3	13	0.187	249
5	1.319	1.266	118.0	6	0.207	255
<i>average</i>		1.257				251
<i>stdev</i>		0.019				5.50

⁽¹⁾ $CSR = \tau_{cyc}/\sigma_{vcon}$

Table 5.2. Summary of results for the cyclic direct simple shear tests carried out on the Cabo Rojo sands, prepared using the modified moist tamping technique.

<i>Loose, Modified Moist Tamping</i>						
Test No. (-)	e_i (-)	e_c (-)	D_{rc} (%)	N_f (-)	CSR @ DA=3.75% (-)	V_s (m/s)
1	1.583	1.389	88.05	24	0.117	198
2	1.597	1.346	98.54	42	0.098	184
3	1.606	1.51	58.54	207	0.080	204
5	1.583	1.473	67.56	11	0.134	215
19	1.622	1.495	62.20	65	0.099	210
20	1.621	1.478	66.34	21	0.117	201
21	1.634	1.494	62.44	86	0.079	187
22	1.692	1.596	37.56	10	0.14	221
<i>average</i>		<i>1.473</i>				<i>203</i>
<i>stdev</i>		<i>0.076</i>				<i>12.88</i>
<i>Dense Modified Moist Tamping</i>						
16	1.393	1.298	110.2	2000	0.08	315
17	1.434	1.358	95.6	152	0.120	307
18	1.397	1.356	96.1	44	0.178	328
23	1.397	1.266	118.0	3	0.220	264
<i>average</i>		<i>1.320</i>				<i>304</i>
<i>stdev</i>		<i>0.045</i>				<i>27.72</i>

Figures 5.7 and 5.8 shows the results summarized in Tables 5.1 and 5.2. A dashed line is drawn at $N=15$ cycles, which represents an earthquake magnitude of 7.5. This figure also shows the average void ratios, relative densities, and shear wave velocity measurements at the end of consolidation. Figure 5.7 shows the cyclic resistance of samples prepared by dry pluviation. As expected, the loosest samples ($e = 1.584$, $D_r = 40.1\%$, $V_s = 241$ m/s) yielded the lowest cyclic resistance. The densest set of samples, however, with average void ratios of $e = 1.397$ and $e = 1.257$ (relative densities of 86.1% and 120%), showed almost the same cyclic resistance. The average shear wave velocity for these two sets of tests was also almost the same (257 m/s and 251 m/s), with the denser samples actually having slightly lower values of shear wave velocity.

Figure 5.8 shows the cyclic resistance for samples prepared by modified moist tamping. For these tests there is a consistent trend of increasing cyclic resistance with both increasing density and shear wave velocity.

Figure 5.7. CSR_{CDSS} vs. Number of cycles to failure for loose to dense specimens of Cabo Rojo sands prepared using dry pluviation.

Figure 5.8. CSR_{CDSS} vs. Number of cycles to failure for medium-dense and dense specimens of Cabo Rojo sands prepared using modified moist tamping.

Figure 5.9 is a combined plot of cyclic resistance with samples prepared by both dry pluviation and moist tamping. Studies from the literature (e.g., Mulilis et al. 1977) have shown that different sample preparation methods results in different cyclic resistances, and it is generally understood that moist tamping produces stiffer samples than other sample preparation methods (i.e., dry pluviation, slurry deposition). This was not seen for the Cabo Rojo sand. In fact, there is not a consistent trend of increasing shear wave velocity and cyclic resistance. One possible explanation for this is that the range of cyclic resistances is quite low (approximately 0.13 to 0.18 at 15 cycles of loading) over a wide range of relative densities and shear wave velocities. For example, cyclic resistance at 15 cycles of loading from the literature (e.g. Figure 2.3) shows a much broader range of values (0.14 to 0.26) for shaking table tests.

Figure 5.9. Cyclic resistance of the Cabo Rojo sand obtained by dry pluviation and modified moist tamping.

Monterey Sand – Figure 5.10 shows typical test results for a loose sample of Monterey sand prepared by dry pluviation, and a summary of all the CDSS tests on Monterey sand are included in Tables 5.3 and 5.4 Again, these tables show the void ratios before (e_i) and after consolidation (e_c), relative densities after consolidation, shear wave velocity (V_s), applied cyclic stress ratio (CSR), and the number of cycles to cause liquefaction (N_f).

Figure 5.10. Typical results of a cyclic DSS test on a loose sample of Monterey sand prepared by dry pluviation.

Table 5.3. Summary of results for the cyclic direct simple shear tests carried out on samples Monterey sand, prepared using the dry pluviation technique.

<i>Loose, Dry Pluviation</i>						
Test No. (-)	e_i (-)	e_c (-)	D_{rc} (%)	N_f (-)	CSR @ DA=3.75% (-)	V_s (m/s)
114	0.752	0.706	46.6	15	0.098	263
115	0.762	0.723	38.8	35	0.08	251
116	0.76	0.729	36.1	113	0.061	249
117	0.75	0.719	40.6	6	0.109	258
125	0.746	0.708	45.7	8	0.103	256
1	0.753	0.713	43.4	4	0.12	251
2	0.749	0.72	40.2	17	0.096	262
3	0.755	0.72	40.2	58	0.07	263
<i>average</i>		0.717				257
<i>stdev</i>		0.008				5.780
<i>Dense Dry Pluviation</i>						
1	0.632	0.613	89.0	9	0.155	287
2	0.629	0.621	85.4	60	0.078	320
3	0.629	0.607	91.8	10	0.117	291
4	0.621	0.586	101.4	6	0.113	279
5	0.638	0.612	89.5	14	0.097	274
6	0.629	0.616	87.7	10	0.119	291
7	0.624	0.611	90.0	45	0.085	297
8	0.633	0.607	91.8	8	0.115	289
9	0.633	0.618	86.8	7	0.135	285
10	0.633	0.61	90.4	37	0.08	269
11	0.639	0.61	90.4	4	0.127	275
12	0.635	0.613	89.0	3	0.116	292
13	0.634	0.603	93.6	10	0.096	284
<i>average</i>		0.610				287
<i>stdev</i>		0.009				12.792
<i>Very Dense Dry Pluviation</i>						
118	0.574	0.545	120.1	8	0.115	272
119	0.592	0.552	116.9	19	0.098	274
120	0.586	0.552	116.9	39	0.081	271
121	0.598	0.566	110.5	46	0.059	261
122	0.586	0.572	107.8	118	0.06	293
123	0.598	0.598	95.9	10	0.11	275
124	0.577	0.548	118.7	69	0.07	283
14	0.569	0.55	117.8	7	0.133	303
15	0.574	0.546	119.6	25	0.098	295
<i>average</i>		0.559				281
<i>stdev</i>		0.017				13.664

Table 5.4. Summary of results for the cyclic direct simple shear tests carried out on samples of Monterey sand, prepared using the Modified Moist Tamping technique.

<i>Monterey Sand, Medium Dense, Modified Moist Tamping</i>						
Test No. (-)	e_i (-)	e_c (-)	D_{rc} (%)	N_f (-)	CSR (-)	V_s (m/s)
1	0.65	0.588	100.5	4	0.101	213
2	0.663	0.596	96.8	8	0.093	215
3	0.66	0.573	107.3	24	0.079	223
4	0.667	0.59	99.5	77	0.059	217
<i>average</i>		<i>0.587</i>				<i>217</i>
<i>stdev</i>		<i>0.009</i>				<i>4.320</i>
<i>Monterey Sand, Dense, Modified Moist Tamping</i>						
5	0.602	0.516	133.33	5	0.106	191
6	0.601	0.532	126.028	8	0.0936	207
7	0.605	0.538	123.29	22	0.0778	211
9	0.605	0.539	122.83	24	0.0783	200
<i>average</i>		<i>0.531</i>				<i>202</i>
<i>stdev</i>		<i>0.0106</i>				<i>8.770</i>

Figures 5.11 and 5.12 shows the results summarized in Tables 5.3 and 5.4 with a red dashed line at $N=15$ cycles representing an earthquake magnitude of 7.5.

Figure 5.11 shows no clear trend between either density or shear wave velocity and the cyclic resistance. This is not reasonable, however no clear explanation for these results could be found. There is a slight trend between shear wave velocity and cyclic resistance. Tests on the loose dry pluviated specimen ($e = 0.717$, $D_r = 41.6\%$) yielded the lowest shear wave velocities, equal to 257 m/s. The shear wave velocities obtained for both dense specimens, $e = 0.61$ ($D_r = 90.4\%$) and $e = 0.559$ ($D_r = 113.7\%$), were very similar and in the order of 287 m/s and 281 m/s. Given that there is barely a difference in the cyclic strength for the dense specimens, the fact that the shear wave velocities are very similar is predictable. Identical trends were observed for the tests performed on moist tamped specimens (Figure 5.12).

Figure 5.11. CSR_{CDSS} vs. Number of cycles to failure for loose and dense specimens of Monterey sand prepared using dry pluviation.

Figure 5.12. CSR_{CDSS} vs. Number of cycles to failure for loose and dense specimens of Monterey sand prepared using the modified moist tamping.

Relationship Between Cyclic Resistance and Shear Wave Velocity – Figure 5.13 shows the CRR- V_s data at $N=15$ (corresponding to the equivalent number of cycles of loading for a $M_w=7.5$ earthquake) for both sands (Cabo Rojo and Monterey) prepared using the dry pluviation and the modified moist tamping technique. This figure suggests very valuable information. It can be observed that the cyclic resistance at 15 cycles to failure appears to be independent of sample preparation methods and also appears to be soil-specific.

Figure 5.13. CRR- V_s relationship for samples of Cabo Rojo and Monterey sand prepared using the dry pluviation and modified moist tamping techniques.

These results are consistent with the findings reported by Tokimatsu et al. (1986) and Baxter et al. (2008), as shown in Figure 5.14. This figure presents CRR- V_s results for Niigata sand, Providence silts, Cabo Rojo sand and Monterey sand. All these samples were prepared by different methods such as air or dry pluviation, preshearing, overconsolidation, modified moist tamping and an intact sample carved from a block. All samples were consolidated to 100 kPa vertical effective stress. The Niigata sand and Providence silt specimens were tested under isotropically consolidated cyclic triaxial conditions whereas the sands used in this study shown in this figure were tested under K_0 -conditions cyclic direct simple shear tests.

One striking difference between the results of this study and other published CRR- V_s relationships is the insensitivity of the cyclic resistance to a wide range of shear wave velocities (i.e. the flatness of the curves). A possible explanation for the flatness of the CSR- V_s relationship for the soils tested for this study is the lack of significant dilation in the denser (higher V_s) samples during shear. Some dilation can be observed, such as the “banana shaped” stress-strain loops in Figure 5.6, but it is clearly not enough to mobilize significant cyclic resistance. It is not clear why more cyclic resistance was not mobilized for the high shear wave velocity samples.

Figure 5.14. Variation of cyclic resistance and shear wave velocity for four soils illustrating that the CRR- V_s relationship appears to be both soil specific and independent of sample preparation method (adapted from Tokimatsu et al. 1986 and Baxter et al. 2008).

5.2.3 Cyclic triaxial tests

A total of 29 isotropically consolidated undrained cyclic triaxial tests were carried out for this study by Michael Dabling as part of his Master's thesis in Ocean Engineering (Dabling 2013). Eleven tests were performed on Monterey sand and 18 tests were performed on Cabo Rojo sand. All samples were prepared using the modified moist tamping method developed by Bradshaw and Baxter (2006). Samples had an initial height of 14.2 cm and a diameter of 7.14 cm. This diameter measurement included a correction for the membrane thickness used to enclose the specimen. Samples were inundated and backpressure saturated up to 240 kPa, using an effective stress of 30 kPa. All samples were subjected to a 100 kPa isotropic consolidation stress for 15 minutes and

the volume change was recorded, in order to accurately calculate the void ratio after consolidation for the samples. The shear wave velocity was measured, at the end of consolidation, using a frequency of 5 kHz. A 1-Hz sinusoidal cyclic load was applied until failure, which was defined as when the pore pressure ratio reached unity ($r_u = 1$). Details of the testing procedures as well as all the results can be found in Dobling (2013).

An example of typical results for a Monterey sand specimen is shown in Figures 5.15 and 5.16. Figure 5.15 shows the applied deviator stress, pore pressure ratio, axial strain and double amplitude strain as a function of the number of cycles of loading. The pore pressure ratio becomes unity (i.e., failure) after 107 cycles of loading. The stress-strain data shown in Figure 5.16 shows that there is preferential strain in extension, suggesting the sample is weaker in extension. This behavior was exhibited by all the samples tested in this study.

Figure 5.15. Typical results from cyclic testing on Monterey sand showing deviator stress, pore pressure ratio, and axial strain as a function of number of cycles of loading (Dobling, 2013).

Figure 5.16. Deviator stress vs. axial strain for the cyclic triaxial test on a sample of Monterey sand shown in Figure 5.15.

The results for the 11 cyclic triaxial tests for the Monterey sand are summarized on Table 5.5. This table includes: (1) void ratio before and after consolidation, (2) shear wave velocity (V_s), applied cyclic stress ratio (CSR), and (4) the number of cycles to liquefaction (N_f). Figure 5.17 is a plot from the data found on Table 5.5. The dashed line indicates the location of $N_f = 15$ cycles, which represents an earthquake magnitude of 7.5. Figure 5.13 shows the CRR- V_s data for a $M_w = 7.5$.

Table 5.5. Summary of results for the cyclic triaxial tests performed on the Monterey sand specimens.

Test No. (-)	e_i (-)	e_c (-)	V_s (m/s)	CSR (-)	N_f (-)
43	0.76	0.76	226	0.223	32
45	0.77	0.76	220	0.176	107
46	0.77	0.76	220	0.247	6
47	0.77	0.76	227	0.230	18
<i>average</i>		0.76	223		
<i>stdev</i>		0	3.775		
51	0.69	0.69	239	0.264	13
53	0.68	0.68	237	0.223	62
54	0.69	0.68	242	0.198	83
56	0.69	0.69	232	0.174	125
<i>average</i>		0.685	238		
<i>stdev</i>		0.006	4.203		
57	0.66	0.66	262	0.247	49
58	0.65	0.64	258	0.309	14
60	0.66	0.65	248	0.198	119
<i>average</i>		0.65	256		
<i>stdev</i>		0.010	7.211

Figure 5.17 CSR_{TX} vs N_f for Monterey samples (Results by Dobling 2013).

Typical results for a cyclic triaxial test on a sample of Cabo Rojo sand are shown in Figure 5.18 and 5.19. Figure 5.18 shows the applied deviator stress, pore pressure ratio, axial strain and double amplitude strain as a function of the number of cycles to failure. From the pore pressure ratio data, failure occurred after 42 cycles of loading. The stress-strain data shown in Figure 5.19 shows that the samples of Cabo Rojo sand were also weaker in extension than in compression.

Figure 5.18. Typical results from cyclic testing on Cabo Rojo sand showing deviator stress, pore pressure ratio, and axial strain as a function of number of cycles of loading (Dobling, 2013).

Figure 5.19. Deviator stress vs. axial strain for the cyclic triaxial test on a sample of Cabo Rojo sand shown in Figure 5.18.

The results for the 18 cyclic triaxial tests carried out on the Cabo Rojo sand are summarized on Table 5.6. This table includes: (1) void ratio before and after consolidation, (2) shear wave velocity (V_s), applied cyclic stress ratio (CSR), and (4) the number of cycles to liquefaction (N_f). Figure 5.20 is a plot from the data found on Table 5.6. The dashed line indicates the location of $N_f = 15$ cycles, which represents an earthquake magnitude of 7.5.

In the opinion of the author, the void ratios listed in Table 5.6 are unreasonably high. Triaxial samples were prepared to be loose, medium dense, and dense, with average void ratios of 1.601, 1.772, and 1.855. However, using values of $e_{\max} = 1.755$ and $e_{\min} = 1.273$, those void ratios listed in Table 5.6 correspond to relative densities of 28.76%, -3.28%, and -20.47%. As the same sand was used for both the cyclic direct simple shear tests and the cyclic triaxial tests, it is believed that these values are incorrect. The values of shear

wave velocity and cyclic resistance are considered to be valid, so only those values will be reported moving forward.

Table 5.6. Summary of results for the cyclic triaxial tests performed on the Calcareous sand specimens.

Test No. (-)	e_o (-)	e_c (-)	V_s (m/s)	CSR (-)	N_f (-)
12	1.64	1.63	258	0.252	42
13	1.62	1.6	263	0.274	30
14	1.61	1.59	266	0.298	26
16	1.62	1.61	268	0.323	20
17	1.62	1.59	257	0.343	10
18	1.61	1.6	193	0.203	155
19	1.61	1.6	262	0.324	12
34	1.61	1.59	252	0.231	44
<i>average</i>		<i>1.601</i>	<i>252</i>		
<i>stdev</i>		<i>0.014</i>	<i>24.535</i>		
20	1.77	1.75	217	0.198	37
22	1.8	1.78	217	0.239	13
23	1.79	1.77	221	0.222	24
24	1.79	1.77	222	0.176	112
25	1.8	1.79	219	0.251	8
26	1.79	1.77	223	0.152	400
<i>average</i>		<i>1.772</i>	<i>220</i>		
<i>stdev</i>		<i>0.013</i>	<i>2.563</i>		
38	1.89	1.85	205	0.193	14
39	1.89	1.86	210	0.151	80
40	1.87	1.85	201	0.201	12
41	1.88	1.86	208	0.131	185
<i>average</i>		<i>1.534</i>	<i>174</i>		
<i>stdev</i>		<i>0.746</i>	<i>84.419</i>		

As shown in Figure 5.20, the calcareous sands with the highest shear wave velocity ($V_s = 252$ m/s) showed the highest resistance to liquefaction. Similar behavior was observed for the calcareous sand tested under CDSS conditions.

Figure 5.20. CSR_{TX} vs N_f for samples of Cabo Rojo sand (Red dash line indicates a $M_w = 7.5$ based on the concept of equivalent numbers of cycles).

5.2.4 Comparison of cyclic simple shear and cyclic triaxial tests results

The cyclic resistance of sands is often measured in the laboratory using cyclic triaxial tests. However, it is known that the cyclic direct simple shear test reproduces horizontal shear stresses imposed by an earthquake more accurately than the cyclic triaxial tests. Both tests impose very different loading conditions on the specimens generating different cyclic stress ratios that cannot be compared directly.

Previous studies on Fraser River Delta sand carried out by Vaid and Sivathayalan (1996) showed that the cyclic strength of specimens tested under cyclic simple shear conditions is significantly lower than of those tested under cyclic triaxial tests. This concept is illustrated on Figure 5.21. It can be observed that at the same relative state parameter, the

differences in terms of CSR between cyclic triaxial and cyclic DSS can be as little as 15% and as much as 60%.

Figure 5.21. Illustration of the difference in cyclic resistance obtained from cyclic simple shear tests and cyclic triaxial tests (Vaid and Sivathayalan, 1996 as reported by Idriss and Boulanger, 2008).

These differences in the cyclic resistance obtained by means of triaxial or simple shear tests are attributed to the different states of consolidation stress and stress-induced anisotropy. For example, a normally consolidated sand specimen that is one

dimensionally consolidated in a simple shear device will have an at-rest earth pressure coefficient (K_o) of approximately 0.5, whereas if consolidated in a typical isotropically consolidated triaxial device K_o will be equal to one.

Ishihara et al. (1977, 1985) carried out a series of cyclic torsional shear tests with different K_o values and showed that the CRR for anisotropically consolidated samples, $K_o \neq 1$ (i.e. comparable to simple shear tests) can be related to the CRR for isotropically consolidated samples, $K_o = 1$ (i.e. isotropically consolidated triaxial tests), with the following equation:

$$CRR_{K_o \neq 1} = \frac{1 + 2K_o}{3} CRR_{K_o = 1} \quad (5.2)$$

Similarly, the CRR from simple shear tests can be related to the CRR of isotropically consolidated undrained triaxial tests with the following relationship:

$$CRR_{SS} = (0.63 - 0.67) CRR_{TX} \quad (5.3)$$

Several other researchers (e.g. Seed and Peacock 1971, Finn et al. 1971, Ishibashi and Sherif 1974, Castro 1975, Seed 1979) have shown similar relationships between the CRR_{SS} and CRR_{TX} . All of their results seem to be in good agreement with those found by Ishihara et al. (1977, 1985). For example, Seed and Peacock suggested a correction factor of 0.63 while Castro suggested a slightly higher correction factor of 0.69.

This section shows a comparison of the cyclic strength of moist tamped samples of Cabo Rojo and Monterey sand, tested under cyclic triaxial and cyclic direct simple shear tests conditions. A summary of the results is shown in Figures 5.22 and 5.23, for the Cabo Rojo and Monterey sand specimens tested under cyclic triaxial and cyclic direct simple shear tests, respectively.

Figure 5.22. Cyclic DSS vs. Cyclic Triaxial for Cabo Rojo sand samples.

As shown in Figure 5.22, the Cabo Rojo sand samples tested under isotropically consolidated undrained cyclic triaxial conditions exhibited an overall higher resistance to liquefaction than those tested under K_0 -consolidated constant volume cyclic direct simple shear tests. As for the shear wave velocities, the approximate 25.4 mm (1 in) tall, cyclic DSS samples showed significantly higher shear wave velocities. When comparing similar

relative densities for both tests, the “densest” condition tested under cyclic triaxial tests corresponds to a void ratio of 1.6 and a relative density of 32.2%, which yielded a shear wave velocity of 231 m/s whereas for the cyclic direct simple shear tests, the closest condition would be for a void ratio of 1.473 and a relative density of 67.6%, which at the same time resulted in a shear wave velocity of 203 m/s. This difference in shear wave velocity is not considered to be significantly higher given the differences in void ratio, sample height and boundary conditions. Looking at the two specimens with similar shear wave velocities: the Cabo Rojo sand tested under CDSS (solid black circles) and the Monterey sand tested under cyclic triaxial tests (open squares), the difference between the obtained CSR is in the order of 44%, which falls between the range obtained for the Fraser Delta sand shown in Figure 5.21.

Figure 5.23. Cyclic DSS vs cyclic Triaxial for Monterey sand samples.

The same behavior in terms of the cyclic strength is observed in Figure 5.23 for the Monterey sand samples. On the contrary, the shear wave velocities for the CDSS specimens were significantly lower than those for the cyclic triaxial specimens, even though the CDSS samples were tested at relative densities higher than 100%.

In order to investigate if the suggested correction factors mentioned before fit the Cabo Rojo sand selected for this study, two samples that yielded very similar shear wave velocities were compared. This is illustrated on Figure 5.24 for two Calcareous specimens and on Figure 5.25 on two Monterey sand samples.

Figure 5.24. Correction factor for Cyclic Triaxial to Cyclic DSS for two Cabo Rojo samples with similar shear wave velocities.

Figure 5.25. Correction factor for Cyclic Triaxial to Cyclic DSS for two Monterey samples with similar shear wave velocities.

Figure 5.24 shows that even though the void ratios at the end of consolidation for the triaxial and simple shear specimens are very different; the shear wave velocities are identical. By trial and error, the triaxial data was adjusted based on the very broad suggested range for c_r between 0.5 to 1. As the figure indicates, the correction factor that best fits the data is equal to 0.52, which is on the lower boundary of the suggested range. As a method to check if this value makes sense for this sand, the internal friction angle was back calculated. This value was found to be 46° , which might seem high at first, but is not that far off from the high internal friction angle values that Cabo Rojo sand tend to exhibit, around 40° - 42° , given this sand's angularity. If we estimate the CRR at $N=15$ based on the actual cyclic DSS data and the corrected cyclic triaxial data, a 4.4% of

difference is found between them. A low percentage of difference suggested good agreement with the data. The same concept was applied to two Monterey specimens with similar shear wave velocity values and the correction factor was found to be equal to 0.35, as shown in Figure 5.25. This value is far below the suggested limit.

5.2.5 Comparison with published data

Calcareous sands are difficult to study given their unique characteristics such as: (1) unusual high void ratios, (2) high intra particle porosity, (3) high specific gravity, (4) low density and (5) chemical composition. Their properties can vary greatly from samples taken from the same location, let alone from samples taken from locations where climate is very different. Table 5.7 shows measured index properties of samples of the Cabo Rojo sand as part of three separate studies.

Table 5.7. Comparison of index properties of Cabo Rojo sand samples obtained from the same beach in 2006 (Sandoval and Pando, 2012) and in 2012 for this study (as well as Dobling 2013).

Index Property		Cabo Rojo Sand (Sandoval and Pando)	Cabo Rojo Sand (Dobling)	Cabo Rojo Sand (Morales)	ASTM
D ₁₀	mm	0.24	0.24	0.24	ASTM D 422-63 (98)
D ₃₀	mm	0.3	0.3	0.3	
D ₅₀	mm	0.37	0.37	0.37	
D ₆₀	mm	0.41	0.42	0.42	
C _u		1.75	1.75	1.75	
C _c		0.94	0.89	0.89	ASTM D 854-06
G _s		2.84	2.87	2.87	
γ _{min}	kN/m ³	9.1	10.2	10.2	ASTM D 4254-00
e _{max}		2.07	1.76	1.75	
γ _{max}	kN/m ³	11.1	12.39	12	Morales: ASTM D 4253-00, Sandoval and Pando: Alternative Method
e _{min}		1.51	1.27	1.34	

The grain size distribution and values of specific gravity are very similar, however there are significant differences between the minimum and maximum void ratios. Although both studies followed the procedures suggested by the ASTM standards to perform the tests, it is believed that the differences in results are due mostly to variability in the tests rather than in the soil. For this reason, comparison of cyclic resistance between these studies is done based on both void ratio and relative density.

The results of the cyclic direct simple shear tests performed on the moist tamped Cabo Rojo sand from this study are shown in Figure 5.26 and compared to the cyclic triaxial testing results performed by Sandoval and Pando (2010) and Dobling (2013).

Sandoval and Pando (2010) prepared the samples using Ladd's (1976) moist tamping, undercompaction method. Samples were prepared in five layers with a final height of 102 mm and diameter of 51 mm. Dobling (2013) prepared his triaxial specimens using the Modified Moist Tamping method (MMT) developed by Bradshaw et al. (2008), also with undercompaction. Samples were prepared in eight layers with a final height of 142 mm and diameter of 71.4 mm. The cyclic direct simple shear samples were also prepared using the MMT method. Samples were prepared in one single layer with final approximate height of 25.4 mm and 63.5 mm in diameter. One important difference between sample preparation methods between this study and Dobling (2013) and Sandoval and Pando (2010) is the molding water contents. For this study and Dobling (2013) molding water contents were calculated keeping a constant degree of saturation of 55%, as the MMT method suggests. The molding water content for this study and

Dobling (2013) ranged from 27.3% to 31.2% and 29% to 33.5%, respectively, decreasing with increasing density; while for Sandoval and Pando (2012) the range vary from 4% - 25%, increasing with increasing density. Ladd's (1976) undercompaction method does not specify if the water content should increase or decrease with density. It only suggests saturation levels between 20% to 70% and that samples with low fines content will need lesser degrees of saturation. Another difference between these studies is the failure criterion. For the cyclic triaxial tests, failure was defined when the pore pressure ratio reached unity whereas for the cyclic simple shear tests failure was defines at a double amplitude strain of 3.75%. Figure 5.27 is shown below to illustrate how variable the cyclic resistance against liquefaction for calcareous sands of different origins can be. The loose and medium dense calcareous sample from Cabo Rojo, PR exhibited the highest resistance to liquefaction. This behavior can be attributed to the angularity of the grains and the higher specific gravity of the sand.

Figure 5.26. Comparison of the cyclic strength obtained by means of cyclic triaxial tests (Dobling 2013, Sandoval and Pando 2012) and CDSS (this study) for the Cabo Rojo sand.

The results of the cyclic direct simple shear tests performed on the dry pluviated Calcareous sands from this study are shown in Figure 5.28 and compared to the cyclic direct simple shear results performed by Brandes (2011). Brandes (2011) also dry pluviated his Calcareous specimens, by tamping and vibrating them as necessary. One difference between these studies was the confining method that provided K_0 -consolidated conditions. For this study a set of Teflon coated stacked rings were used, whereas Brandes (2011) used a wired reinforced membrane manufactured by NGI.

Figure 5.27. Comparison of cyclic resistance against liquefaction for calcareous sands at similar relative densities (all samples consolidated to a vertical effective stress of 100 kPa).

Even though the relative densities for the silica and quartz based sands (Nevada and Monterey) are different, their behavior is identical. Clearly the cyclic resistance of the calcareous sands is higher than those of the Nevada and Monterey sand. The specimens

that showed the higher resistance to liquefaction were the Calcareous sands collected at the study site in Cabo Rojo, PR.

Figure 5.28. CSR_{DSS} vs N_f for the Calcareous sands tested for this study and from Hawaii.

5.2.6 Evaluation of particle crushing

Susceptibility to crushing is a very important consideration for granular soils since it highly influences its geotechnical properties. Particle crushing is especially known to be an issue when dealing with calcareous deposits, given this material's high intraparticle voids and brittle mineralogy. To evaluate the crushing potential of the Cabo Rojo sand, grain size analyses were performed before and after several CDSS tests were performed. The crushing of particles was measured by comparing the grain size distribution curves obtained before and after the tests. Figure 5.29 and 5.30 shows the grain size distribution

curves before and after each CDSS and cyclic triaxial tests, for the Cabo Rojo sand, and it indicates that there was no crushing at the stress levels used in this study. This does not mean that the calcareous sands from Cabo Rojo are not susceptible to crushing, as it will clearly depend on the stress levels and boundary conditions of each test. Special attention or care must take place when dealing with such unique soils.

Figure 5.29. Grain size distribution curves before and after a CDSS test.

Figure 5.30. Grain size distribution curves before and after a cyclic triaxial test.

5.3 Effect of varying the carbonate content on the cyclic resistance of the calcareous sands

A series of K_0 consolidated constant volume cyclic direct simple shear tests were carried out to investigate the effect of varying the carbonate content ($\text{CaCO}_3\%$) on the liquefaction resistance of dry pluviated, loose sand samples collected during the geotechnical site investigation.

As described in Chapter 4, two Standard Penetration Tests (B1 and B2) were performed at the beach site in Cabo Rojo, PR with the main purpose of collecting enough material to be tested in the laboratory. The carbonate content was measured by combustion (loss on ignition) at the South Laboratory located at the URI, Bay Campus. Tests results revealed CaCO_3 contents in the range of 15-97%, with decreasing carbonate contents with depth.

Figure 5.31 shows the variation of carbonate content down hole with the SPT N-values obtained at the site.

Standard soil classification tests (grain size, maximum density, minimum density, specific gravity) were carried out on the soil samples collected from B1 and B2. Figure 5.32 shows the grain size distribution for the soils selected for this part of the study and Table 5.8 shows a summary of the results from the soil classification. This table includes minimum and maximum void ratio, specific gravity, grain diameters corresponding to 10%, 30% and 60% passing and the coefficients of uniformity and gradation. The minimum and maximum void ratios for groups 2, 3, 4 and 5 were obtained using an alternative method. Given the fact that not enough material was collected from the site investigation, the standard $1/30 \text{ ft}^3$ mold suggested by ASMT D 4253 could not be used to determine the maximum void ratio. A smaller mold (6.3 cm diameter, 15.1 cm height) was designed and built by Robin Freeland at the URI Bay Campus. The vibratory table, overburden stress, amplitude, frequency and duration of shaking suggested by this standard were kept as suggested by the standard.

According to the USCS, all the soil from Groups 1 through 6, classify as poorly graded sands. Soil gradation is very important in geotechnical engineering. It is well known that the behavior of soils is primarily governed by effective stresses and grain size.

Table 5.9 shows a summary of the soil characterization and CDSS results. In terms of the soil classification, this table includes the soil type, depths at which samples were

collected, carbonate content, specific gravity, void ratio at the end of consolidation and the corresponding relative density. Also the cyclic resistance ratio at 15 cycles to failure and the measured shear wave velocities can be found in this table.

Figure 5.31. Variation of CaCO_3 content downhole and SPT N-values for B1 and B2.

Figure 5.32. Grain size distribution for the surface soils, Monterey sand and the soils collected down hole at the site investigation.

Table 5.8. Results for minimum and maximum void ratio, specific gravity and grain size analysis for the soils used for this part of the study.

Group (-)	Soil Type (-)	$e_{\min}$	$e_{\max}$	G_s (-)	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_u (-)	C_c (-)
1	Calcareous	1.34	1.75	2.87	0.23	0.3	0.43	1.87	0.91
2	Calcareous	1.37	1.75	2.87	0.13	0.24	0.43	3.31	1.03
3	Calcareous	1.43	2.12	2.9	0.75	0.2	0.43	0.57	0.12
4	Calcareous	1.42	1.95	2.84	0.75	0.18	0.35	0.47	0.12
5	Calcareous	1.2	1.89	2.86	0.75	0.19	0.32	0.43	0.15
6	Monterey	0.56	0.808	2.66	0.32	0.45	0.58	1.81	1.09

Table 5.9. Summary of geotechnical parameters from the soils collected from the geotechnical site investigation at Cabo Rojo, PR.

Group (-)	Soil Type (-)	Sample ID (-)	Depth (m)	CaCO ₃ (%)	G _s (-)	e _c (-)	D _r (%)	CRR ₁₅ (-)	V _s (m/s)
1 L	Calcareous	Surface	0	> 95	2.87	1.584	40.5	0.128	241
1 D	Calcareous	Surface	0	> 95	2.87	1.397	86.1	0.149	257
2-3	Calcareous	B1S2, B1S3, B2S2, B1S4, B1S5, B1S7 B1S8,	0.46-3.2	70-95	2.89	1.624	56	0.148	247
4-5	Calcareous	B2S8, B1S9, B1S10, B1S11, B2S9	3.2-5	15-48	2.85	1.546	68	0.159	214
6	Monterey	N/A	N/A	0	2.66	0.717	41.2	0.094	259

Figure 5.33 shows the results shown in Table 5.9 in a CSR-N_f plot. The data in blue corresponds to the standard Monterey #0/30 sand and is shown for comparison purposes.

Figure 5.33. CRR-Vs for the soils collected downhole at the geotechnical site investigation in PR.

The cyclic resistance results obtained for groups 2 and 3 and 4 and 5 were grouped because the results followed a clear trend. From this figure we can observe that the loose, dry pluviated Monterey samples yielded the lowest cyclic resistance to liquefaction. The shear wave velocity values decreased with decreasing carbonate content for the Cabo Rojo sands (Calcareous) but the highest shear wave velocity measurement was obtained for the loose Monterey samples. No clear conclusions can be made about these tests results because there are many variables influencing the cyclic behavior of these soil samples such as grain size, mineralogy, relative density and shear wave velocity.

5.4 Comparison of cyclic resistance ratio and shear wave velocity relationships

The main objective of the experimental part of this investigation was to assess the relationship between the cyclic resistance and shear wave velocity for the Cabo Rojo sand. This data will be used in the following chapter to evaluate the applicability of field based approaches to estimate the liquefaction potential of calcareous sands using shear wave velocity.

Laboratory values of cyclic resistance were equated to field values by taking the cyclic resistance at 15 cycles of shaking. This is considered to be equivalent to liquefaction due to a magnitude 7.5 earthquake, as suggested by Seed and Idriss (1971).

The CRR obtained by means of cyclic DSS tests was converted to field conditions by applying a multidirectionality factor of 0.9, as suggested by Seed et al. (1975b). On the

other hand, the CRR obtained by means of cyclic triaxial testing were converted to in-situ conditions by applying the same multidirectionality factor plus a conversion factor from triaxial to simple shear conditions of equal to 0.63, as suggested by Seed and Peacock (1971). The values of V_s were corrected for overburden (100 kPa) and K_o conditions.

Figure 5.34 shows the final estimated field CRR- V_{s1} relationship for the soils tested in this study and by Dobling (2013) against the field-based approach suggested by Andrus and Stokoe (1997) and Kayen et al. (2013), as well as other relevant data found in the literature and previous research carried at URI on Rhode Island silts.

The biggest concern about the CDSS results of this study is that the CRR- V_{s1} relationships are significantly less sensitive to both void ratio and shear wave velocity (i.e. “flatter”). This is especially unusual at higher values of shear wave velocities (i.e. dense samples). This behavior is consistent with the absence of dilation spikes during the cyclic shear. It is still not clear why the cyclic resistance is not larger under these conditions.

As it can be observed from this figure, the curves obtained for the Cabo Rojo and Monterey sand, either by cyclic triaxial tests or cyclic DSS, are to the right of the field-based curves. This behavior suggests (1) the CRR- V_{s1} relationship is soil specific, (2) the use of field-based curves available in the literature for all soils may not be appropriate and (3) the use of the available field-based curves for the Cabo Rojo sand may be unconservative. In other words, the results of this study suggests that the liquefaction

resistance of the Cabo Rojo and Monterey sand is much lower than predicted by the field-based approaches.

Similar behavior can be observed for the Farmer's Markets Silts, which are non-plastic, dilatant silts found in Providence, RI and for the Kawaihae sand, which is a calcareous, uncemented sand from Hawaii. In fact, the behavior observed for the Calcareous sands from Hawaii is very similar to the Cabo Rojo sand from PR, even though their gran size distributions are very different. One interesting finding is that the Calcareous sands from Hawaii also did not show significant dilation spikes during shear.

Figure 5.34. CRR- V_{s1} for the soils tested in this study against the field-based approach developed by Andrus and Stokoe (1997) and Kayen et al. (2013).

5.5 Summary of Results

The objective of the laboratory program designed for this study was to develop the CRR- V_{s1} relationship for the Cabo Rojo sands collected at the study site in PR, as well as the standard silica based Monterey #0/30 sand for use in evaluating field-based liquefaction approaches. A series of K_o -consolidated constant volume CDSS tests and isotropically consolidated undrained cyclic triaxial tests, both with shear wave velocity measurements, were performed on the two sands.

Tests were carried out on samples prepared at different void ratios, to recreate loose and dense states, for each sand. To evaluate the effects of soil fabric on the cyclic resistance of soils, samples were prepared using two different sample preparation methods: dry pluviation and modified moist tamping.

The significant findings from this experimental study were as follows:

- The shear wave velocity measured on the Cabo Rojo sands by mean of resonant column tests at the University of Texas at Austin and the University of Notre Dame were in good agreement with the shear wave velocities measured with the new set of end caps and bender elements specifically designed and fabricated for this study. This validates our system.
- The Monterey sand tested under CDSS conditions showed very little sensitivity to void ratios for reasons that are yet to be understood.

- The $CRR-V_{s1}$ correlation for the Monterey and Cabo Rojo sand are in fact different.
- The $CRR-V_{s1}$ correlation for both sands appears to be independent of sample preparation methods.
- The $CRR-V_{s1}$ relationships developed in this study are less sensitive to changes in shear wave velocities (i.e. the curves are flatter) than those reported in previous studies from triaxial testing results. One study on calcareous sands from Hawaii and a silica sand tested in cyclic simple shear showed very similar “flat” $CRR-V_{s1}$ relationships.
- The correction factors to convert cyclic triaxial data to CDSS suggested in the literature seems to fit the results obtained for the Cabo Rojo sand, but not the Monterey sand. For this sand, the c_r is much lower than values suggested by Ishihara (1975, 1977).
- An attempt was made to evaluate the effect of calcareous sand content on liquefaction resistance by testing samples from Borings 1 and 2 that exhibited decreasing carbonate content with depth. Minimum and maximum void ratios were measured for each soil, and cyclic tests were performed. However, there was too much variation in the samples’ values of relative density to make definitive statements about the effect of carbonate content on cyclic resistance.
- Overall, the Cabo Rojo sand exhibited higher resistance to liquefaction than the Monterey sand. These results are in agreement with previous studies carried out by Brandes (2011) and Sandoval and Pando (2012) on both Calcareous and quarts/silica based sands.

The results of the laboratory testing program suggests that the liquefaction resistance of the Cabo Rojo and Monterey sands is much lower than predicted by the field-based approaches.

The last statement is the most significant finding of this experimental program. It suggests that the liquefaction resistance of the Cabo Rojo and Monterey sand is lower than predicted by the field-based approaches developed by Andrus and Stokoe (2000) and Kayen et al. (2013). If true, then using the field-based shear wave velocity approaches will significantly overestimate the liquefaction resistance of the Cabo Rojo and Monterey sand.

CHAPTER 6 EVALUATION OF FIELD-BASED LIQUEFACTION APPROACHES

6.1 Introduction

This chapter presents an evaluation of field-based liquefaction approaches (SPT, CPT, and DMT-based approaches) for the Cabo Rojo sand using the soil-specific cyclic resistance-shear wave velocity ($CRR-V_s$) relationship developed in the laboratory. A comparison of $CRR-V_s$ relationships found in the literature is shown. Also, a summary of the most relevant findings and some conclusions are presented herein.

6.2 Soil-Specific $CRR-V_s$ Relationship

As described in Chapter 5, a $CRR-V_s$ relationship was developed for the Cabo Rojo sand from a series of K_o -consolidated, constant volume cyclic direct simple shear tests and cyclic triaxial tests with shear wave velocity measurements on reconstituted samples. The laboratory determined cyclic shear strengths were converted to field strengths using corrections proposed by Seed (1979). The cyclic simple shear test results were corrected for multidirectional shaking (0.9) and the cyclic triaxial tests results were corrected for conversion from triaxial test to simple shear conditions ($c_r = 0.63$), multidirectional shaking (0.9) and K_o conditions. The measured shear wave velocities were normalized to 1 atm. This correlation was used to assess the insitu cyclic resistance at the site (based on in situ shear wave velocity measurements) as a baseline for comparing the cyclic resistance predicted using the CPT-based field approach. Figure 6.1 shows the soil

specific $CRR_{FIELD}-V_{s1}$ relationship developed in the laboratory for the Cabo Rojo sands by means of CDSS tests and cyclic triaxial tests.

With the $CRR-V_s$ relationship developed in the laboratory and the cyclic shear strengths converted to field strengths, the comparison between laboratory and field behavior using shear wave velocity as the link was the next step. This was achieved with the following steps:

- Select a V_{s1} value from the shear wave velocity profile obtained at the site investigation (either by means of sCPT or sDMT, see Figures 4.9 through 4.12).
- Enter Figure 6.1 with the value of V_{s1} , from the field data and select the corresponding CRR_{FIELD} based on either cyclic triaxial or cyclic simple shear conditions.
- The resulting value of CRR_{FIELD} can then be compared to the CRR from the different field-based approaches.

Given that the shear wave velocities obtained in the field were generally lower than those obtained in the laboratory by means of bender element testing, the $CRR_{field}-V_{s1}$ relationship was extrapolated in order to obtain values of CRR at lower shear wave velocities. This is shown in Figure 6.1 for the Cabo Rojo sand by the extended red dashed line.

Figure 6.1. Proposed in situ CRR- V_{s1} relationship for the Cabo Rojo sands compared to the field-based curves of Andrus and Stokoe (1997) and Kayen et al. (2013).

6.3 CRR- $(N_1)_{60}$ Relationship

A geotechnical investigation was performed at the test site located in Cabo Rojo, PR, which included conventional drilling and sampling, standard penetration tests (SPT) with energy measurements, cone penetration tests with shear wave velocity measurements (V_s) and flat dilatometer tests also with shear wave velocity measurements. This section focuses on the results obtained for the standard penetration tests.

The SPT-based approach was initially proposed independently by Seed and Idriss (1971) and Whitman (1971). The current standard-of-practice described in Youd et al. (2001) and Idriss and Boulanger (2008) utilizes the data and correlations of Seed et al. (1985)

with some modifications. The relationship proposed by Idriss and Boulanger is used in this study. Cyclic resistance ratio is correlated to $(N_1)_{60}$, defined as the Standard Penetration Test blow count corrected to an effective overburden stress of 1 atm (~100 kPa) and a hammer efficiency of 60%. This correlation was developed from field evidence of liquefaction (e.g. sand boils, settlements, ground cracking) observed at sites that had experienced an earthquake. Based on samples recovered, the calcareous sands were found up to approximately 3.5 to 4 meters, and so the prediction of cyclic resistance only extends to a depth of 4 meters. A blow count of zero was used for the CRR prediction in Eq. 6.1.

At the study site, soils were found to be in a very loose state, with blow counts ranging from weight-of-rod (i.e. 0) up to 19. In the calcareous sand layer, most of the N-values were either weight of rod or weight of hammer. This is very challenging when comparing the results with the field-based correlations because the blow counts and tip resistances are extremely low. The hammer efficiency for the CME 55 T Automatic SPT drill rig used for this study was found to be 81.6% with a standard deviation of 1.2. The CRR for a 7.5 magnitude earthquake for these soils was estimated using the following equation:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 * (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (6.1)$$

where $(N_1)_{60}$ is the raw N-value corrected for energy ratio, overburden, borehole diameter, rod length and samplers with or without liners. This equation only applies for $(N_1)_{60} < 30$. $(N_1)_{60}$ values higher than 30 are considered too dense to liquefy. Figure 6.2 shows the raw blow counts (N), the corrected blow counts $(N_1)_{60}$ and the cyclic resistance ratio estimated for the soils at the site, the soil-specific CRR- V_s relationship developed in

the laboratory and the shear wave velocity profile measured adjacent to the SPT locations. Based on the blow counts, the calcareous sand layer was found to be down up to approximate 5 meters for both boring logs. Based on the samples recovered, the calcareous sands were found up to approximately 3.5 to 4 meters. Below these depths, the CRR- V_{s1} relationship developed in the laboratory is not valid because the material changes to a clayey/silty layer.

Figure 6.2. SPT N and $(N_1)_{60}$ values and the estimated $CRR_{7.5}$.

As we can observe from this Figure, the soils are extremely loose (i.e. blow counts in the calcareous layer of interest of weight of rod and weight of hammer) therefore the estimated cyclic resistance obtained with the SPT-based approach is almost zero (~ 0.05). This SPT-based prediction is plotted against the soil-specific CRR- V_{s1} relationship obtained in the laboratory by means of cyclic triaxial tests and cyclic direct simple shear tests. It can be observed from this Figure that the cyclic resistance obtained with this

soil-specific relationship is greater for the Cabo Rojo sand when compared to the SPT-based approach. These results are reasonable given the low blow counts encountered in the field and the low shear wave velocities measured at the site. Also, the SPT-based approach is known to be a very conservative approach.

6.4 CRR- q_c Relationship

The procedure developed by Robertson and Wride (1998) forms the basis for evaluating liquefaction potential using the CPT (Youd et al. 2001). This approach correlates CRR with cone tip resistance normalized (q_{c1N}) to an effective overburden stress of 1 atm (~ 100 kPa), as described in section 2.4.

Two CPT soundings with shear wave velocity measurements were performed at the study site adjacent to Borings B-1 and B-2. The $CRR_{FIELD}-V_{s1}$ correlation developed in the laboratory was used with shear wave velocity measurements from the sCPT to estimate the cyclic resistance of the Cabo Rojo sand, and then the resistance was compared to the CPT-based field liquefaction approaches developed by Robertson and Wride (1998), Idriss and Boulanger (2008) and Robertson (2014).

Figure 6.3 shows the estimated CRR profiles for sCPT-1 and sCPT-2 locations using the soil-specific $CRR_{FIELD}-V_{s1}$ relationship. Based on the low values of skin friction in sCPT-1 and sCPT-2, the calcareous sand layer is up to 4 meters in depth. As shown in this figure, the soil-specific $CRR-V_s$ relationship developed by means of CDSS is in reasonable agreement with the cyclic resistance obtained with the field-based approaches developed by Idriss and Boulanger (2008) and Robertson and Wride (1998).

The Robertson (2014) yielded higher values of CRR than the other field-based approaches. Based on the data from sCPT-1, the field-based approaches all over-predicted the cyclic resistance slightly over the soil-specific cyclic resistances. Based on the data from sCPT-2, the agreement between the field-based approaches and the soil-specific was good. In fact, the Robertson (2014) method and the soil-specific method matched in the high velocity layers in the upper 1.5 m.

Figure 6.3. Cyclic resistance of the Cabo Rojo sand using the CPT field-based approach and a site-specific analysis.

6.5 CRR- K_D Relationship

The seismic dilatometer test can be used to assess the liquefaction potential of a soil deposit using two different parameters: the horizontal stress index (K_D) and shear wave velocity (V_s). The horizontal stress index is a potentially useful parameter to evaluate liquefaction potential given its sensitivity to stress history, prestraining, aging, cementation and structure amongst others. All of these factors are known to increase the

cyclic resistance (CRR) of soils. The use of shear wave for evaluating cyclic resistance is more established and both field based approaches and soil-specific correlations have been proposed in the literature.

Two DMT soundings with shear wave velocity measurements were performed at the study site in Cabo Rojo, PR. The $CRR_{FIELD}-V_{s1}$ correlation developed in the laboratory was used to assess the in situ cyclic resistance at the site (based on in situ shear wave velocity measurements) and as a baseline for comparing the cyclic resistance predicted using the DMT-based field liquefaction approach developed by Monaco et al. (2005).

This $CRR-K_D$ relationship proposed by Monaco et al. (2005) was developed by combining existing $CRR-K_D$ curves along with experience incorporated in current available methods based on SPT and CPT data. This correlation was compared against field performance from different sites that experienced liquefaction after an earthquake, such as Loma Prieta 1989 ($M_w=7$), that also had been characterized using the DMT. This $CRR-K_D$ relationship only applied to $M_w = 7.5$ earthquakes and clean sands.

The cyclic resistance ratio can be estimated with the following equation:

$$CRR_{7.5} = 0.0107K_D^3 - 0.0741K_D^2 + 0.2169K_D - 0.1306 \quad (6.2)$$

where K_D is the horizontal stress index and can be estimated with the following equation:

$$K_D = \frac{p_o - u_o}{S'_{vo}} \quad (6.3)$$

where σ'_{vo} is the pre-insertion in situ overburden stress, u_o is the static pre-insertion pore water pressure and p_o is the corrected first reading, which at the same time can be calculated with the following equation:

$$p_o = 1.05(A - Z_m + \Delta A) - 0.05(B - Z_m - \Delta B) \quad (6.4)$$

where A and B are the uncorrected first and second pressure readings in the field, ΔA and ΔB are corrections determined by membrane calibration and Z_m is the gage reading when vented to atmospheric pressure. Figure 6.4 shows the CRR estimated with this field-based correlation developed by Monaco et al. (2005) with the CRR estimated using the laboratory CRR- V_s correlation developed by CDSS and cyclic triaxial tests. For DMT 1, it can be observed that the field-based procedures overestimated the cyclic resistance of the soils found at the site, at all depths. For DMT 2, at very shallow depths, from the surface down up to 0.5 meters, the cyclic resistance obtained with the field-based approach is very high. This DMT sounding was made near the parking lot area of the beach. An explanation to this high CRR value could be that soils were compacted by the cars and boats or that the calcareous sands were cemented. Also, the shear wave velocity measurement at this depth is abnormally high ($> 1,000$ m/s). Even though this value is definitely not expected in soils, and could be potentially attributed to a faulty reading, it verifies such a high cyclic resistance ratio based on the horizontal stress index. At greater depths, around 1 to 4 meters, where the calcareous layer is found there is very good agreement with the field-based approach and the correlation developed in the laboratory by means of CDSS and cyclic triaxial testing. Once we abandon the calcareous sand layer, the field-based approach overestimates the cyclic resistance of those soils. The cyclic resistance to liquefaction obtained by both methodologies is very low in the

calcareous layer. This is expected given the low horizontal stress index found in these sands. Low horizontal stress index indicated: (1) loose sands, (2) uncemented sands, (3) low K_o environment and (4) little to no stress history in the deposit. Sand with all of these conditions is expected to liquefy under cyclic loading. All of these factors were verified with data collected with the SPT, sampling and CPT tests.

Figure 6.4. CRR predicted by the DMT method compared with CRR predicted with the laboratory correlation.

Maugeri and Monaco (2006) compared the cyclic resistance ratio (CRR) predicted by both shear wave velocity (V_s) and horizontal stress index (K_D). In order to do this, the CRR- V_s method developed by Andrus and Stokoe (2004) and the CRR- K_D method by Monaco et al. (2005) were compared by constructing a relationship between V_{s1} and K_D

implied by the CRR- V_{sl} curve for sands with a fine content less than 5% and the CRR- K_D curve. This “equivalence” curve was obtained by combining the equations (and eliminating CRR) used to estimate CRR based on both methods. These equations are shown below:

$$CRR_{7.5} = 0.022 \left(\frac{K_{a1} V_{sl}^2}{100} + 2.8 \frac{1}{V_{sl}^* - K_{a1} V_{sl}} - \frac{1}{V_{sl}^*} K_{a2} \right) \quad (6.5)$$

$$CRR_{7.5} = 0.0107 K_D^3 - 0.0741 K_D^2 + 0.2169 K_D - 0.1306 \quad (6.6)$$

where V_{sl}^* is the limiting upper value of normalized shear wave velocity for liquefaction (215 m/s), K_{a1} and K_{a2} are factors to correct shear wave and CRR, respectively, for aging. For this case both factors were equal to one.

A possible advantage of this new V_{sl} - K_D relationship is that it provides a direct comparison of both field-based approaches to evaluate liquefaction without the need to calculate CSR or CRR. This allows for data from sites that have not been shaken by strong motions to be included and used to evaluate the methods. This is very helpful given the very few well-documented case histories with DMT data.

This new approach is shown in Figure 6.5 with the CRR-equivalence curve. This figure also includes V_{sl} - K_D data pairs collected by sDMT at different sites of mostly sand deposits. These datapoints shown are limited to a maximum depth of 15 meters, which is the usual depth range for liquefaction occurrence (Maugeri and Monaco, 2006). Also shown in the figure are the datapoints collected with the sDMT at the test site in Cabo Rojo, PR.

Figure 6.5. CRR-equivalence curve between Andrus et al. (2004) CRR- V_{s1} and Monaco et al. (2005) CRR- K_D relationships for clean sands and magnitude 7.5 earthquake and the Cabo Rojo soils.

6.6 Discussion and Summary

The significant findings from this chapter were as follows:

- The soils found at the site study were in a very loose state, with raw blow counts (N-values) ranging from weight of rod to approximate 20 and very low tip resistances (0 – 7 MPa).
- When evaluating the $CRR-(N_1)_{60}$ relationship for these sands, the values of cyclic resistance obtained were too low. As a matter of fact the CRR predicted by the SPT approach for the calcareous sand layer was almost zero.
- The $CRR_{FIELD}-V_{s1}$ relationship developed in the laboratory was used to assess the in situ cyclic resistance at the site using the in situ shear wave velocity measurements
- For sCPT-1, the $CRR_{FIELD}-V_{s1}$ relationship developed in the laboratory by means of CDSS was in reasonable agreement with the CRR estimated with the field-based approaches developed by Robertson and Wride (1998) and Idriss and Boulanger (2008) at the calcareous sand layer. The updated version of Roberson and Wride (1998), which can be found in Robertson (2014) yielded consistently higher values of CRR.
- For sCPT-2, the $CRR_{FIELD}-V_{s1}$ relationship from the CDSS tests was in good agreement with the field-based approaches available in the literature.

- A CRR- K_D relationship was also developed for the soils found at the Cabo Rojo, PR site. The cyclic resistance obtained with this field-based approach was compared to the CRR obtained with the laboratory soil specific relationship obtained for the Cabo Rojo sand. Overall, the DMT field-based approach overestimated the CRR of the soils found at the site.

CHAPTER 7 SUMMARY AND CONCLUSIONS

7.1 Introduction

The main objective of this research was to assess the applicability of current field-based liquefaction approaches for calcareous sands. This is of importance because there are many coastal communities in areas of high seismicity founded on calcareous sands (e.g. Haiti, Puerto Rico, Hawaii, Dubai, etc.), and the existing field-based approaches were developed almost entirely for case studies in silica sands. In addition, the review of existing research described above from laboratory studies shows contradictory results as to how the cyclic behavior of these unique soils compare to silica sands. It may be that the cyclic resistance of these soils is controlled by the unique mineralogy and grain characteristics and may be soil specific.

This was achieved by means of a combined field and laboratory investigation. The field program included Standard Penetration Tests with energy measurements and conventional sampling, Cone Penetration Tests with shear wave velocity measurements and Flat Dilatometer Tests also with shear wave velocity measurements. The laboratory program included K_0 -consolidated constant volume cyclic direct simple shear tests with shear wave velocity measurements and isotropically consolidated undrained cyclic triaxial tests also with shear wave velocity measurements on reconstituted samples. This chapter summarizes the work done to achieve the main objective, lists the major

conclusions found from this research and presents some recommendations for future work.

7.2 Summary

Chapter 1 presents the statement of the problem and a brief description of how calcareous sands are formed, where they are found and mention some of their very unique characteristics. The overall objective and scope of research is described in this chapter as well and an explanation of why Puerto Rico (PR) was selected as the study area is given.

Chapter 2 presents a detailed literature review, which includes an overview of the liquefaction phenomena and a description of the liquefaction fundamentals and the cyclic loading behavior of saturated sands. This chapter also describes the mechanism of pore pressure generation during cyclic loading for triaxial and simple shear conditions. Also included is a literature review of the most relevant existing research available in the literature on the cyclic behavior of Calcareous sands. It was found that there is no clear trend in terms of the magnitudes of the cyclic resistance of the Calcareous sands when compared to those reported for standard silica sands, which have been extensively used in laboratory testing. A description of the field-based approaches for the assessment of liquefaction potential (e.g. SPT, CPT, DMT, V_s) is also described in this chapter along with the applicability of shear wave velocity to link laboratory and in situ behavior.

Chapter 3 presents a description of the geotechnical site investigation program carried out for this research. An aerial map with the location of the borings and soundings performed

at the site is also shown and typical profiles of the soils found at the site with the most relevant geotechnical parameters (e.g. $(N_1)_{60}$, q_{c1N} , V_s) are included.

Chapter 4 describes the laboratory testing program designed for this study, the equipment used (Geocomp's CDSS machine and Instron's cyclic load frame apparatus) and the design and fabrication of the bender element end caps used to measure the time delay necessary to calculate the shear wave velocity on the short samples tested under simple shear conditions. A description of the soils tested for study is included. According to the Unified Soil Classification System (USCS), the Cabo Rojo sand classifies as a poorly graded sand (SP) with angular particles, fine to medium in size. The carbonate content of this sand is > 95% and the mineralogy studies confirmed the predominant mineral composing the sand matrix were calcite and aragonite. The latter being a heavier mineral, which is responsible for this sand's very high specific gravity ($G_s = 2.87$). The minimum and maximum void ratios for this sand can be up to 2 to 3 times higher when compared to those reported for standard silica sands. The silica sand used in this study for comparison purposes, Monterey #0/30, also classifies as poorly graded sand, with rounded, uniform and fine grains.

Chapter 5 describes the results of the laboratory testing program designed for this study. A summary of the results obtained from the cyclic simple shear tests and cyclic triaxial tests performed on reconstituted specimens of the Cabo Rojo and Monterey sand using dry pluviation and modified moist tamping is included in this chapter as well as a comparison with published data. Also, a comparison of the cyclic resistance of the Cabo

Rojo sands obtained by means of cyclic simple shear tests and cyclic triaxial tests is shown. In addition, an evaluation of particle crushing for the Cabo Rojo sand and the effect of varying the carbonate content on the cyclic resistance of these sands collected downhole during the site investigation is included. Finally, this chapter ends with a comparison of the cyclic resistance ratio (CRR) and shear wave velocity (V_s) relationship for the soils selected for this study against the field-based approaches available in the literature and laboratory data also found in the literature for Calcareous sands from Hawaii, tested under the same boundary conditions and stress levels. It was found that the Cabo Rojo sand tested for this study that the CRR- V_{s1} relationship was less sensitive to changes in shear wave velocity than those reported in previous studies. Also, it was found that the shear wave velocity for the Cabo Rojo sand is significantly higher than those of standard silica sands.

Chapter 6 presents an evaluation of the CRR of the Cabo Rojo sand using a SPT-based, CPT-based, DMT-based approach and the results of a soil-specific CRR- V_{s1} relationship developed in the laboratory.

7.3 Conclusions

The significant results from this research were as follows:

- The shear wave velocity measured on the Cabo Rojo sand by means of resonant column tests at the University of Texas at Austin and the University of Notre Dame were in good agreement with the shear waves measured with the new set of end caps and bender elements specifically designed and fabricated for this study.

- The Monterey sand tested under CDSS conditions showed very little sensitivity to void ratios for reasons that are yet to be understood.
- The $CRR-V_{s1}$ relationships for the Monterey and Cabo Rojo sand are in fact different and appears to be independent of sample preparation methods.
- The $CRR-V_{s1}$ relationship developed in this study is less sensitive to changes in shear wave velocities (i.e. the curves are flatter) than those reported in previous studies. In addition, the shear wave velocities obtained for the Cabo Rojo sand are significantly higher than those of standard silica sands.
- The correction factors to convert cyclic triaxial data to CDSS suggested in the literature seems to fit the results obtained for the Cabo Rojo sand, but not the Monterey sand. For this sand, the c_r values are lower than those suggested by Ishihara (1975, 1977).
- An attempt was made to evaluate the effect of calcareous sand content on liquefaction resistance by testing samples from Borings 1 and 2 that exhibited decreasing carbonate content with depth. Minimum and maximum void ratios were measured for each soil, and cyclic tests were performed. However, there was too much variation in the samples' values of relative density to make definitive statements about the effect of carbonate content on cyclic resistance.

- Overall, the Cabo Rojo sand exhibited higher resistance to liquefaction than the Monterey sand. This is in agreement with the results of previous research carried out on Calcareous sands from Hawaii by Morioka and Nicholson (2000) and Brandes (2011). Also, these results are similar to the results obtained by Sandoval et al. (2011) on the same Cabo Rojo sand tested under isotropically consolidated cyclic undrained triaxial tests.
- The comparison of the tests results for this study with existing field-based correlations suggests that the Monterey and Cabo Rojo sands lies outside the liquefaction resistance curves used in practice. Using these field-based correlations would significantly overestimate the liquefaction resistance of both the Monterey and Cabo Rojo sands.
- The soils found at the site study were in a very loose state, with raw blow counts (N-values) ranging from weight of rod to approximate 20 and very low tip resistances (0 – 7 MPa). When evaluating the $CRR-(N_1)_{60}$ relationship for these sands, the cyclic resistance obtained were too low. As a matter of fact the CRR obtained for the calcareous sand layer was almost zero.
- The $CRR-V_{s1}$ relationship developed in the laboratory was used to assess the in situ cyclic resistance at the site, based on in situ shear wave velocity measurements, and as a baseline for comparing the cyclic resistance predicted

using the CPT-based field liquefaction approach developed by Robertson and Wride (1998), Idriss and Boulanger (2008) and Robertson (2014).

- For sCPT-1, the $CRR-V_s$ relationship developed in the laboratory by means of CDSS was in reasonable agreement with the CRR estimated with the field-based approaches developed by Robertson and Wride (1998) and Idriss and Boulanger (2008) at the calcareous sand layer. The updated version of Robertson and Wride (1998), which can be found on Robertson (2014) overestimated the CRR of the Cabo Rojo sand. The $CRR-V_s$ relationship obtained by means of cyclic triaxial testing gave the lowest values of cyclic resistance, which suggests that the current field-based approaches developed for silica sands may overestimate the CRR of the Cabo Rojo sand. For CPT 2, the $CRR-V_s$ relationship obtained by means of CDSS was in good agreement with the field-based approaches available in the literature.
- A $CRR-K_D$ relationship was also developed for the soils found at the Cabo Rojo, PR site. The cyclic resistance obtained with this field-based approach was compared to the CRR obtained with the laboratory soil specific relationship obtained for the calcareous sands. Overall, the DMT field-based approach overestimates the CRR of the soils found at the site.

- There was good agreement between the shear wave velocities measured by the seismic CPT and DMT. The CRR estimated by both field-based approaches (DMT and CPT) overall, overestimated the cyclic resistance of the Cabo Rojo sand when compared to the soil-specific CRR- V_s relationship. This study highlights the need for more study on assessing the liquefaction potential of calcareous sands.

In summary, the most significant finding of this experimental program suggests that the liquefaction resistance of the Cabo Rojo and Monterey sand is lower than those predicted by the field-based approaches developed by Andrus and Stokoe (2000) and Kayen et al. (2013). This means that using these suggested and well accepted field based shear wave velocity approaches will significantly overestimate the liquefaction resistance of the Cabo Rojo and Monterey sand.

7.4 Recommendations for future work

For further continuation of this project, the author recommends the following:

- It is assumed that the frequency at which the samples are sheared doesn't play an important role when dealing with cohesionless soils. Investigating the influence of frequency on the CRR of these unique soils should be taken into consideration.
- The soils at the site selected for this study were too loose and therefore no matter which field-based approach was used to assess the liquefaction potential at the site, the values of CRR were very low. Also, recreating such loose reconstituted

specimens in the laboratory is very difficult, especially when dealing with such short samples (~ 1 in, 25.4 mm in height). Ideally, another site with denser and deeper a deeper Calcareous sand layer should be found and more field and laboratory testing should be performed in order to link the laboratory and field behavior more accurately.

- This research only studied the cyclic capacity at the site. More research could be done evaluating the cyclic demand of this site.
- All the tests performed for this research were stress-controlled tests. Further research should be carried out on the Cabo Rojo sands tested under strain-controlled tests and the influence of their mineralogy on the shear modulus could be evaluated.
- It is known that the mechanical behavior of sands might be influenced by its particle morphology (i.e. particle size and shape). For this study, particle size was assessed only by sieve analysis and the particle shapes were qualitatively described. A more complex analysis including state-of-the-art particle imagery on these sands with very unique particle characteristics (angular grains, shell fragments of different sizes and porous structure) should be considered.

APPENDIX 1 X-RAY DIFFRACTION

Figure A1.1. Soil ID: Bucket Depth: 0 ft. (surface) Samples: A through H

Figure A1.2. Soil ID: Boring 1, Sample 1 Depth: 0 - 1.5 ft., 0 - 0.46 m. Samples: A through H

Figure A1.3. Soil ID: Boring 1, Sample 2 Depth: 1.5 - 3 ft., 0.46 – 0.91 m. Samples: A through H

Figure A1.4. Soil ID: Boring 1, Sample 3 Depth: 3 - 4.5 ft., 0.91 – 1.37 m. Samples: A through H

Figure A1.5. Soil ID: Boring 1, Sample 4 Depth: 4.5 - 6 ft., 1.37 - 1.83 m. Samples: A through H

Figure A1.6. Soil ID: Boring 1, Sample 5 Depth: 6 - 7.5 ft., 1.83 - 2.29 m. Samples: A through H

Figure A1.7. Soil ID: Boring 1, Sample 7 Depth: 9 - 10.5 ft., 2.74 - 3.2 m. Samples: A through H

Figure A1.8. Soil ID: Boring, 2 Sample 1 Depth: 0 - 1.5 ft., 0 - 0.46 m Samples: A through H

Figure A1.9. Soil ID: Boring 2, Sample 2 Depth: 1.5 - 3 ft., 0.46 - 0.91 m. Samples: A through H

Figure A1.10. Soil ID: Boring 2, Sample 6 Depth: 7.5 - 9 ft., 2.29 - 2.74 m. Samples: A through H

Figure A1.11. Soil ID: Boring 2, Sample 8 Depth: 10.5 - 12 ft., 3.2 - 3.66 m. Samples: A through H

APPENDIX 2 FIELD TEST PLOTS

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Project		Job N°	Ground Elevation	Boring ID
Date Start	Water Level Depth	Date	Time	General Remarks
Date Finish	Water Level Depth	Date	Time	Drilling Method
				Foreman

Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks	Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks
0		12					5						
1		6					6						
2		4	①	15"	10		7						
3		3	3	11"	5		8						
4		2	2	12"	5		9		2				
5		3	2	12"	5		0		3	⑭	13"	8	
6		2	2	8"	1		1						
7		wh	wh	1"	1		2						
8							3						
9							4						
0		wh	wh	4"	wh		5						
1		wh	wh	15"	wh		6						
2		wh	wh	14"	wh		7						
3		wh	wh	12"	wh		8						
4		wh	wh	10"	wh		9						
5		wh	wh	10"	wh		0						
6		wh	wh	10"	wh		1						
7							2						
8							3						
9		3					4						
0		4					5						
1		4					6						
2							7						
3							8						
4							9						
5		wh	wh	10"	7		0						

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Revised: 08-2003

Created by: CJRM

Revised by: ARC

Figure A2.1. Boring log, Cabo Rojo, PR: p1 of 5

Project		Job N°		Ground Elevation		Boring ID	
216 to 051048's						B-1 (P2)	
Date Start	Water Level	Date	Time	General Remarks		Drilling Method	
8/3/13	Depth 3'	8/3/13				CME-SS	
Date Finish	Water Level	Date	Time			Foreman	
8/3/13	Depth					C. ROSS	

Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks	Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks
0							5						
1		① 15"	7				6						
2		② 15"	7				7						
3		③ 10"	2				8						
4		④ WH	X	WH		no sample recovered	9						
5		⑤ WH	X	WH		no sample recovered	0						
6		⑥ WH	X	WH		no sample recovered	1						
7		⑦ WH	X	WH			2						
8		⑧ WH	X	WH			3						
9		⑨ WH	X	WH			4						
0		⑩ WH	X	WH			5						
1		⑪ WH	X	WH		12" weight of soil	6						
2		⑫ WH	X	WH			7						
3		⑬ WH	X	WH			8						
4		⑭ WH	X	WH		harder soil, harder to penetrate	9						
5		⑮ WH	X	WH		continuous sand	0						
6		⑯ WH	X	WH			1						
7		⑰ WH	X	WH			2						
8		⑱ WH	X	WH			3						
9		⑲ WH	X	WH		10' 18" c 18.5'	4						
0		⑳ WH	X	WH		Foot 2 to 4 muffled	5						
1		㉑ WH	X	WH			6						
2		㉒ WH	X	WH			7						
3		㉓ WH	X	WH			8						
4		㉔ WH	X	WH			9						
5		㉕ WH	X	WH			0						

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Figure A2.2. Boring log, Cabo Rojo, PR: p2 of 5

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Project UR1/UNCC/UPRM		Job N°	Ground Elevation	Boring ID B3 next to ramp
Date Start March 11/13	Water Level Depth	Date	Time	General Remarks energy measurements by Dr. B. Anderson
Date Finish March 11/13	Water Level Depth	Date	Time	Drilling Method Hollow Stem Auger with Foreman Wichay

Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks	Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks
0							5						
1	S1	13 5	18"	9			6						
2		4					7						
3	S2	4 3	18"	6			8						
4	S3	3 3	8"	5			9	S13	2 2	18"	7		CLAY E2
5		1					10		5				
6	S4	1 1	9"				1	S14	1 4	18"	10		E3
7	S5	1 1	X	WH		NO RECOVERY	2	S15	6 9	18"	22		F4 Normal = 27
8	S6	4 WH	4.5	WH			3		11				
9		WH					4	S16	12 14	18"	20		E5
10	S7	WH	2	WH			5						
1		WH					6						
2	S8	WH	15	WH			7						
3	S9	WH	14	WH			8						
4	S10	WH	15	WH		SE empuja a ver problemas de inclinación ir de S a N	9						
5							10						
6							1						
7							2						
8							3						
9	S11	3 3	18"	8			4						
10		5					5						
1							6						
2							7						
3							8						
4	S12	2 3	18"	7		Empujando energía mecánica CLAY E1	9						
5		4					10						

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Created by: CJRM

Revised by: ARC

Figure A2.3. Boring log, Cabo Rojo, PR: p3 of 5

GEOCONSULT

Project UR11 UNCC/UPRN		Job N°	Ground Elevation	Boring ID B4
Date Start March 12 13	Water Level Depth	Date	Time	General Remarks
Date Finish March 12 13	Water Level Depth	Date	Time	
				Drilling Method Hollow Stem Auger with
				Foreman

Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks	Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks
0							5						
1	S1	5 4 3	15"	7			6						
2							7						
3	S2	3 2 3	10"	5			8						
4	S3	3 2 2					9						
5							0						
6	S4	2 WH 1	3"				1						
7	S5	WH 1 1	4"	2			2						
8							3						
9	S6	1 WH WH	2"				4						
0	S7	WH WH WH	2"				5						
1							6						
2	S8	WH WH WH	0.5"			Material aluvial tipo el "zapato" y no encaja bien en la cámara	7						
3	S9	WH WH WH					8						
4							9						
5	S10	WH 2 3		5			0						
6							1						
7							2						
8							3						
9							4						
0							5						
1							6						
2							7						
3							8						
4							9						
5							0						

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Revised: 08-2003

Created by: CJRM

Revised by: ARC

Figure A2.4. Boring log, Cabo Rojo, PR: p4 of 5

GEOCONSULT

Project <u>UR1/UNCC/UPRM</u>		Job N°	Ground Elevation	Boring ID <u>B5</u>
Date Start <u>March 13</u>	Water Level	Date	Time	General Remarks
Date Finish <u>March 13</u>	Depth	Date	Time	
				Drilling Method <u>Hollow Stem Auger</u>
				Foreman

Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks	Depth feet	Sample Type	Blows 6-12-18" Sample N°	Sample Recover	N Value	Graph	Soil Description and Remarks
0							5						
1	S1	7		18"	9		6						
2		5					7						
2 3/4		4					8						
3		S2	3		7		9						
4	S3	4			3		0						
5		2					1						
6		1					2						
7	S4	1	15"				3						
8		WH	WH				4						
9	S5	WH	WH		WH		5						
0		WH	WH				6						
1		WH	WH				7						
2		WH	WH				8						
3	S6	1	1"		WH		9						
4		WH	WH				0						
5		WH	WH				1						
6	S7	WH	WH		2" WH		2						
7		WH	WH				3						
8		WH	WH				4						
9	S8	WH	WH		0.5" WH		5						
0		WH	WH				6						
1		WH	WH				7						
2	S9	WH	WH		18" WH	se mece a ver medio limo pulcro	8						
3		WH	WH			basicamente rodad	9						
4		WH	WH			veredera por en	0						
5		WH	WH			chubasco	1						
6	S11	WH	WH		18"	Bottom 6" era clay pero con un poco de sand por encima	2						
7		WH	WH				3						
8	S12	2			4		4						
9		2					5						
0							6						
1							7						
2							8						
3							9						
4							0						
5							1						

Created: 08-2003

Revised: 08-2003

Created by: CJRM

Revised by: ARC

Figure A2.5. Boring log, Cabo Rojo, PR: p5 of 5

APPENDIX 3 RESONANT COLUMN TEST RESULTS

Sample ID: YW02

Soil Type: Cabo Rojo sand (surface)

Total Unit Weight = 1.47 g/cm³, Dry Unit Weight = 1.18 g/cm³

Water Content = 24.5%, Degree of Saturation = 49.3%

Void Ratio = 1.43, Relative Density = 78.2%

Diameter = 5.08 cm, Height = 10.18 cm

Table 3.1. Variation in shear wave velocity, shear modulus, damping ratio and estimated void ratio against isotropic confining pressure from RC tests on sample YW02.

Isotropic Confining Pressure, σ'_0			Low-Amplitude Shear Modulus, G_{max}		Low-Amplitude Shear Wave Velocity, V_s	Low-Amplitude Material Damping Ratio, D_{min}	Estimated Void Ratio, e
(psi)	(psf)	(kPa)	(ksf)	(MPa)	(fps)	(%)	
2.25	324	16	1640	78.6	758	0.50	1.428
4.5	648	31	1999	95.8	837	0.43	1.428
9	1296	62	2493	119.5	935	0.41	1.429
18	2592	124	3081	147.7	1039	0.30	1.427
36	5184	248	3958	189.7	1177	0.22	1.423
72	10368	496	5241	251.3	1353	0.17	1.415

Figure A3.1. Shear wave velocity versus isotropic confining pressure for sample YW02.

Sample ID: YW03

Soil Type: Cabo Rojo sand (surface)

Total Unit Weight = 1.47 g/cm³, Dry Unit Weight = 1.18 g/cm³

Water Content = 24.7%, Degree of Saturation = 49.2%

Void Ratio = 1.44, Relative Density = 75.8%

Diameter = 5.10 cm, Height = 10.14 cm

Table A3.2. Variation in shear wave velocity, shear modulus, damping ratio and estimated void ratio against isotropic confining pressure from RC tests on sample YW03.

Isotropic Confining Pressure, σ'_0			Low-Amplitude Shear Modulus, G_{max}		Low-Amplitude Shear Wave Velocity, V_s	Low-Amplitude Material Damping Ratio, D_{min}	Estimated Void Ratio, e
(psi)	(psf)	(kPa)	(ksf)	(MPa)	(fps)	(%)	
2.25	324	16	1588	76.1	747	0.47	1.438
4.5	648	31	1975	94.7	833	0.60	1.440
9	1296	62	2485	119.1	935	0.35	1.439
18	2592	124	3169	151.9	1055	0.33	1.437
36	5184	248	4030	193.2	1189	0.24	1.433
72	10368	496	5345	256.2	1368	0.16	1.426

Figure A3.2. Shear wave velocity versus isotropic confining pressure for sample YW03.

Sample ID: YW05

Soil Type: Cabo Rojo sand (surface)

Total Unit Weight = 1.44 g/cm³, Dry Unit Weight = 1.13 g/cm³

Water Content = 26.8%, Degree of Saturation = 50.3%

Void Ratio = 1.53, Relative Density = 53.8%

Diameter = 5.05 cm, Height = 10.16 cm

Table A3.3. Variation in shear wave velocity, shear modulus, damping ratio and estimated void ratio against isotropic confining pressure from RC tests on sample YW05.

Isotropic Confining Pressure, σ_o'			Low-Amplitude Shear Modulus, G_{max}		Low-Amplitude Shear Wave Velocity, V_s	Low-Amplitude Material Damping Ratio, D_{min}	Estimated Void Ratio, e
(psi)	(psf)	(kPa)	(ksf)	(MPa)	(fps)	(%)	
2.25	324	16	1282	61.4	678	0.62	1.528
4.5	648	31	1539	73.7	743	0.40	1.527
9	1296	62	1937	92.7	833	0.26	1.526
18	2592	124	2503	119.8	946	0.18	1.523
36	5184	248	3379	161.8	1098	0.14	1.516
72	10368	496	4768	228.3	1300	0.12	1.499

Figure A3.3. Shear wave velocity versus isotropic confining pressure for sample YW05.

Sample ID: **YW07**

Soil Type: Cabo Rojo sand (surface)

Total Unit Weight = 1.47 g/cm³, Dry Unit Weight = 1.18 g/cm³

Water Content = 24.3%, Degree of Saturation = 48.9%

Void Ratio = 1.43, Relative Density = 78.7%

Diameter = 5.06 cm, Height = 10.30 cm

Table A3.4. Variation in shear wave velocity, shear modulus, damping ratio and estimated void ratio against isotropic confining pressure from RC tests on sample YW07.

Isotropic Confining Pressure, σ_o'			Low-Amplitude Shear Modulus, G_{max}		Low-Amplitude Shear Wave Velocity, V_s	Low-Amplitude Material Damping Ratio, D_{min}	Estimated Void Ratio, e
(psi)	(psf)	(kPa)	(ksf)	(MPa)	(fps)	(%)	
2.25	324	16	1539	73.7	735	0.55	1.426
4.5	648	31	1838	88.0	803	0.43	1.426
9	1296	62	2335	111.8	905	0.30	1.424
18	2592	124	3001	143.7	1025	0.25	1.421
36	5184	248	4021	192.5	1185	0.19	1.415
72	10368	496	5350	256.2	1364	0.14	1.404

Figure A3.4. Shear wave velocity versus isotropic confining pressure for sample YW07.

Sample ID: **YW08**

Soil Type: Cabo Rojo sand (surface)

Total Unit Weight = 1.42 g/cm³, Dry Unit Weight = 1.11 g/cm³

Water Content = 27.9%, Degree of Saturation = 50.8%

Void Ratio = 1.58, Relative Density = 42.3%

Diameter = 5.06 cm, Height = 10.10 cm

Table A3.5. Variation in shear wave velocity, shear modulus, damping ratio and estimated void ratio against isotropic confining pressure from RC tests on sample YW08.

Isotropic Confining Pressure, σ_o'			Low-Amplitude Shear Modulus, G_{max}		Low-Amplitude Shear Wave Velocity, V_s	Low-Amplitude Material Damping Ratio, D_{min}	Estimated Void Ratio, e
(psi)	(psf)	(kPa)	(ksf)	(MPa)	(fps)	(%)	
2.25	324	16	1205	57.7	661	0.57	1.575
4.5	648	31	1437	68.8	721	0.44	1.574
9	1296	62	1848	88.5	818	0.25	1.573
18	2592	124	2434	116.5	938	0.18	1.569
36	5184	248	3280	157.0	1088	0.14	1.566
72	10368	496	4700	225.0	1297	0.12	1.547

Figure A3.5. Shear wave velocity versus isotropic confining pressure for sample YW08.

APPENDIX 4 CYCLIC DIRECT SIMPLE SHEAR TESTS

CABO ROJO SAND (BUCKET, $\text{CaCO}_3\% > 95\%$, $\sigma = 100$ kPa)
DRY PLUVIATION

Figure A4.1. CYC_2014_CS_G1_01, $e_c = 1.591$, $D_r = 38.8\%$, $V_{s1} = 248$ m/s

Figure A4.2. CYC_2014_CS_G1_02, $e_c = 1.582$, $D_r = 41.0\%$, $V_{s1} = 234$ m/s

Figure A4.3. CYC_2014_CS_G1_03, $e_c = 1.575$, $D_r = 42.7\%$, $V_{s1} = 245$ m/s

Figure A4.4. CYC_2014_CS_G1_04, $e_c = 1.587$, $D_r = 39.8\%$, $V_{s1} = 235$ m/s

Figure A4.5. CYC_2014_CS_G1D_01, $e_c = 1.380$, $D_r = 90.2\%$, $V_{s1} = 257$ m/s

Figure A4.6. CYC_2014_CS_G1D_02, $e_c = 1.407$, $D_r = 83.7\%$, $V_{s1} = 257$ m/s

Figure A4.7. CYC_2014_CS_G1D_04, $e_c = 1.430$, $D_r = 82.2\%$, $V_{s1} = 259$ m/s

Figure A4.8. CYC_2014_CS_G1D_05, $e_c = 1.388$, $D_r = 88.3\%$, $V_{s1} = 254$ m/s

Figure A4.9. CYC_2014_CS_G1D2_01, $e_c = 1.224$, $D_r = 128.3\%$, $V_{s1} = 243$ m/s

Figure A4.10. CYC_2014_CS_G1D2_02, $e_c = 1.259$, $D_r = 119.8\%$, $V_{s1} = 257$ m/s

Figure A4.11. CYC_2014_CS_G1D2_03, $e_c = 1.269$, $D_r = 117.3\%$, $V_{s1} = 250$ m/s

Figure A4.12. CYC_2014_CS_G1D2_04, $e_c = 1.269$, $D_r = 117.3\%$, $V_{s1} = 249$ m/s

Figure A4.13. CYC_2014_CS_G1D2_05, $e_c = 1.266$, $D_r = 118.0\%$, $V_{s1} = 255$ m/s

MODIFIED MOIST TAMPING

Figure A4.14. CYC_2014_CS_01, $e_c = 1.389$, $D_r = 88.1\%$, $V_{s1} = 198$ m/s

Figure A4.15. CYC_2014_CS_02, $e_c = 1.346$, $D_r = 98.5\%$, $V_{s1} = 184$ m/s

Figure A4.16. CYC_2014_CS_03, $e_c = 1.510$, $D_r = 58.5\%$, $V_{s1} = 204$ m/s

Figure A4.17. CYC_2014_CS_05, $e_c = 1.473$, $D_r = 67.6\%$, $V_{s1} = 215$ m/s

Figure A4.18. CYC_2014_CS_19, $e_c = 1.495$, $D_r = 62.2\%$, $V_{s1} = 210$ m/s

Figure A4.19. CYC_2014_CS_20, $e_c = 1.478$, $D_r = 66.3\%$, $V_{s1} = 201$ m/s

Figure A4.20. CYC_2014_CS_21, $e_c = 1.494$, $D_r = 66.5\%$, $V_{s1} = 187$ m/s

Figure A4.21. CYC_2014_CS_22, $e_c = 1.596$, $D_r = 37.6\%$, $V_{s1} = 221$ m/s

Figure A4.22. CYC_2014_CS_17, $e_c = 1.358$, $D_r = 95.6\%$, $V_{s1} = 307$ m/s

Figure A4.23. CYC_2014_CS_18, $e_c = 1.356$, $D_r = 96.1\%$, $V_{s1} = 328$ m/s

Figure A4.24. CYC_2014_CS_23, $e_c = 1.266$, $D_r = 118.1\%$, $V_{s1} = 264$ m/s

MONTEREY SAND $\sigma = 100$ kPa
DRY PLUVIATION

Figure A4.25. CYC_2014_114_MS, $e_c = 0.706$, $D_r = 46.6\%$, $V_{s1} = 263$ m/s

Figure A4.26. CYC_2014_115_MS, $e_c = 0.723$, $D_r = 38.8\%$, $V_{s1} = 251$ m/s

Figure A4.27. CYC_2014_116_MS, $e_c = 0.729$, $D_r = 36.1\%$, $V_{s1} = 249$ m/s

Figure A4.28. CYC_2014_117_MS, $e_c = 0.719$, $Dr = 40.6\%$, $Vs1 = 258$ m/s

Figure A4.29. CYC_2014_125_MS, $e_c = 0.708$, $Dr = 45.7\%$, $Vs1 = 256$ m/s

Figure A4.30. CYC_2014_MS_G6_01, $e_c = 0.713$, $Dr = 43.3\%$, $Vs1 = 251$ m/s

Figure A4.31. CYC_2014_MS_G6_02, $ec = 0.72$, $Dr = 40.2\%$, $Vs1 = 262$ m/s

Figure A4.32. CYC_2014_MS_G6_03, $ec = 0.72$, $Dr = 40.2\%$, $Vs1 = 263$ m/s

Figure A4.33. CYC_2014_MS_G6D_01, $ec = 0.613$, $Dr = 89.04\%$, $Vs1 = 287$ m/s

Figure A4.34. CYC_2014_MS_G6D_02, $ec = 0.621$, $Dr = 85.4\%$, $Vs1 = 320$ m/s

Figure A4.35. CYC_2014_MS_G6D_03, $ec = 0.607$, $Dr = 91.8\%$, $Vs1 = 291$ m/s

Figure A4.36. CYC_2014_MS_G6D_04, $ec = 0.586$, $Dr = 101.4\%$, $Vs1 = 279$ m/s

Figure A4.37. CYC_2014_MS_G6D_05, $ec = 0.638$, $Dr = 89.5\%$, $V_{s1} = 274$ m/s

Figure A4.38. CYC_2014_MS_G6D_06, $ec = 0.616$, $Dr = 87.7\%$, $V_{s1} = 291$ m/s

Figure A4.39. CYC_2014_MS_G6D_07, $ec = 0.611$, $Dr = 90.0\%$, $V_{s1} = 297$ m/s

Figure A4.40. CYC_2014_MS_G6D_08, $ec = 0.607$, $Dr = 91.8\%$, $Vs1 = 289$ m/s

Figure A4.41. CYC_2014_MS_G6D_09, $ec = 0.618$, $Dr = 86.8\%$, $Vs1 = 285$ m/s

Figure A4.42. CYC_2014_MS_G6D_10, $ec = 0.610$, $Dr = 90.4\%$, $Vs1 = 269$ m/s

Figure A4.43. CYC_2014_MS_G6D_11, $ec = 0.610$, $Dr = 90.4\%$, $Vs1 = 275$ m/s

Figure A4.44. CYC_2014_MS_G6D_12, $ec = 0.613$, $Dr = 89.04\%$, $Vs1 = 292$ m/s

Figure A4.45. CYC_2014_MS_G6D_13, $ec = 0.603$, $Dr = 93.6\%$, $Vs1 = 284$ m/s

Figure A4.46. CYC_2014_118_MS, $ec = 0.545$, $Dr = 120.1\%$, $Vs1 = 272$ m/s

Figure A4.47. CYC_2014_119_MS, $ec = 0.552$, $Dr = 116.9\%$, $Vs1 = 274$ m/s

Figure A4.48. CYC_2014_120_MS, $ec = 0.586$, $Dr = 116.9\%$, $Vs1 = 271$ m/s

Figure A4.49. CYC_2014_121_MS, $e_c = 0.566$, $Dr = 110.5\%$, $Vs1 = 261$ m/s

Figure A4.50. CYC_2014_122_MS, $e_c = 0.586$, $Dr = 107.8\%$, $Vs1 = 293$ m/s

Figure A4.51. CYC_2014_123_MS, $e_c = 0.598$, $Dr = 95.9\%$, $Vs1 = 275$ m/s

Figure A4.52. CYC_2014_124_MS, $ec = 0.548$, $Dr = 118.7\%$, $Vs1 = 283$ m/s

Figure A4.53. CYC_2014_MS_G6D_14, $ec = 0.550$, $Dr = 117.8\%$, $Vs1 = 303$ m/s

Figure A4.54. CYC_2014_MS_G6D_15, $ec = 0.550$, $Dr = 119.6\%$, $Vs1 = 295$ m/s

MODIFIED MOIST TAMPING

Figure A4.55. CYC_2014_MS_01, $ec = 0.588$, $Dr = 100.5\%$, $Vs1 = 213$ m/s

Figure A4.56. CYC_2014_MS_02, $ec = 0.596$, $Dr = 96.8\%$, $Vs1 = 215$ m/s

Figure A4.57. CYC_2014_MS_03, $ec = 0.573$, $Dr = 107.3\%$, $Vs1 = 223$ m/s

Figure A4.58. CYC_2014_MS_04, $ec = 0.590$, $Dr = 99.5\%$, $Vs1 = 217$ m/s

Figure A4.59. CYC_2014_MS_05, $ec = 0.516$, $Dr = 133.3\%$, $Vs1 = 191$ m/s

Figure A4.60. CYC_2014_MS_06, $ec = 0.532$, $Dr = 126.02\%$, $Vs1 = 207$ m/s

Figure A4.61. CYC_2014_MS_07, $ec = 0.538$, $Dr = 123.4\%$, $Vs1 = 211$ m/s

Figure A4.62. CYC_2014_MS_09, $ec = 0.539$, $Dr = 122.8\%$, $Vs1 = 200$ m/s

CABO ROJO SAND (DOWNHOLE, $CaCO_3\% \sim 79 - 95\%$, $\sigma = 100$ kPa)

Figure A4.63. CYC_2014_CS_G2_02, $ec = 1.527$, $Dr = 58.7\%$, $Vs1 = 264$ m/s

Figure A4.64. CYC_2014_CS_G2_03, $ec = 1.536$, $Dr = 56.3\%$, $Vs1 = 243$ m/s

Figure A4.65. CYC_2014_CS_G2_04, $ec = 1.533$, $Dr = 57.1\%$, $Vs1 = 234$ m/s

Figure A4.66. CYC_2014_CS_G2_05, $ec = 1.541$, $Dr = 55.0\%$, $Vs1 = 244$ m/s

Figure A4.67. CYC_2014_CS_G3_01, $e_c = 1.736$, $D_r = 55.7\%$, $V_{s1} = 244$ m/s

Figure A4.68. CYC_2014_CS_G3_02, $e_c = 1.748$, $D_r = 53.9\%$, $V_{s1} = 251$ m/s

Figure A4.69. CYC_2014_CS_G3_03, $e_c = 1.746$, $D_r = 54.2\%$, $V_{s1} = 250$ m/s

CABO ROJO SAND (DOWNHOLE, $\text{CaCO}_3\%$ ~ 15 - 48%, $\sigma = 100$ kPa)

Figure A4.70. CYC_2014_CS_G4_01, $ec = 1.591$, $Dr = 67.7\%$, $Vs1 = 213$ m/s

Figure A4.71. CYC_2014_CS_G4_02, $ec = 1.591$, $Dr = 67.7\%$, $Vs1 = 213$ m/s

Figure A4.72. CYC_2014_CS_G4_03, $ec = 1.60$, $Dr = 66.0\%$, $Vs1 = 218$ m/s

Figure A4.73. CYC_2014_CS_G5_01, $ec = 1.522$, $Dr = 53.3\%$, $Vs1 = 211$ m/s

Figure A4.74. CYC_2014_CS_G5_03, $ec = 1.488$, $Dr = 58.3\%$, $Vs1 = 213$ m/s

Figure A4.75. CYC_2014_CS_G5_04, $ec = 1.505$, $Dr = 56.0\%$, $Vs1 = 215$ m/s

Figure A4.76. CYC_2014_CS_G5_05, $ec = 1.525$, $Dr = 52.9\%$, $Vs1 = 218$ m/s

DRY PLUVIATION

Figure A4.77. CYC_2014_CS_01, $ec = 1.323$, $Dr = 104.2\%$

Figure A4.78. CYC_2014_CS_02, $ec = 1.329$, $Dr = 102.8\%$

Figure A4.79. CYC_2014_CS_03, $ec = 1.319$, $Dr = 105.0\%$

Figure A4.80. CYC_2014_CS_04, $ec = 1.331$, $Dr = 102.3\%$

Figure A4.81. CYC_2014_CS_15, $ec = 1.341$, $Dr = 99.7\%$

Figure A4.82. CYC_2014_CS_17, $ec = 1.280$, $Dr = 115.7\%$

Figure A4.83. CYC_2014_CS_06, $ec = 1.554$, $Dr = 47.8\%$

Figure A4.84. CYC_2014_CS_07, $ec = 1.541$, $Dr = 51.0\%$

Figure A4.85. CYC_2014_CS_08, $ec = 1.542$, $Dr = 50.7\%$

Figure A4.86. CYC_2014_CS_12, $ec = 1.536$, $Dr = 52.1\%$

Figure A4.87. CYC_2014_CS_13, $ec = 1.536$, $Dr = 52.3\%$

Figure A4.88. CYC_2014_CS_16, $ec = 1.530$, $Dr = 54.4\%$

MONTEREY SAND ($\sigma = 200$ kPa)
DRY PLUVIATION

Figure A4.89. CYC_2014_44_M, $ec = 0.66$, $Dr = 68.2\%$

Figure A4.90. CYC_2014_45_M, $ec = 0.67$, $Dr = 65.5\%$

Figure A4.91. CYC_2014_46_M, $ec = 0.65$, $Dr = 70.3\%$

Figure A4.92. CYC_2014_47_M, $ec = 0.67$, $Dr = 64.6\%$

Figure A4.93. CYC_2014_48_M, $ec = 0.73$, $Dr = 34.7\%$

Figure A4.94. CYC_2014_49_M, $ec = 0.73$, $Dr = 36.0\%$

Figure A4.95. CYC_2014_52_M, $ec = 0.73$, $Dr = 36.4\%$

Figure A4.96. CYC_2014_53_M, $ec = 0.73$, $Dr = 37.3\%$

Figure A4.97. CYC_2014_54_M, $ec = 0.73$, $Dr = 37.6\%$

Figure A4.98. CYC_2014_55_M, $ec = 0.73$, $Dr = 37.1\%$

Figure A4.99. CYC_2014_56_M, $ec = 0.72$, $Dr = 39.3\%$

Figure A4.100. CYC_2014_57_M, $ec = 0.62$, $Dr = 86.5\%$

Figure A4.101. CYC_2014_58_M, $ec = 0.62$, $Dr = 84.0\%$

Figure A4.102. CYC_2014_59_M, $ec = 0.62$, $Dr = 84.9\%$

Figure A4.103. CYC_2014_60_M, $ec = 0.63$, $Dr = 83.0\%$

Figure A4.104. CYC_2014_61_M, $ec = 0.62$, $Dr = 84.5\%$

Figure A4.105. CYC_2014_62_M, $ec = 0.62$, $Dr = 84.0\%$

Figure A4.106. CYC_2014_63_M, $ec = 0.62$, $Dr = 87.6\%$

Figure A4.107. CYC_2014_64_M, $ec = 0.62$, $Dr = 86.7\%$

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