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Modeling the Static and Dynamic Properties of Uncemented, Natural Sands

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MODELING THE STATIC AND DYNAMIC PROPERTIES OF UNCEMENTED, NATURAL SANDS

A Dissertation
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Doctor of Philosophy
Civil Engineering

by
Michael P Esposito III
August 2015

Accepted by:
Dr. Ronald D. Andrus, Committee Chair
Dr. C. Hsein Juang
Dr. Nadarajah Ravichandran
Dr. Qiushi Chen
ABSTRACT

Research to quantify the influence of aging processes (or diagenesis) on the static peak shear strength, the dilatancy, and the small strain dynamic stiffness of uncemented predominantly quartz sands is presented in this dissertation. New equations are proposed to model the dilatancy and the static shear strength due to diagenesis in natural sands as functions of either age or measured to estimated velocity ratio (MEVR). New predictive relationships between small strain dynamic stiffness and age are also recommended based on laboratory and field test results in natural sands.

A laboratory investigation was performed to quantify the influence of age (or diagenesis) on the peak shear strength and the dilatancy of an uncemented Pleistocene age sand deposit at the Coastal Research and Education Center (CREC) near Charleston, South Carolina. Drained triaxial compression tests were performed on high quality intact specimens retrieved using the in situ freezing and frozen core sampling method, and on remolded specimens prepared to match the densities of the intact specimens. The stress-strain behavior of intact specimens was accompanied by dilation and a maximum or peak shear value, whereas remolded specimens generally contracted throughout shearing. The peak friction angle of intact specimens was found to be 3.0-8.6° higher than the peak friction angle of remolded specimens. A diagenesis-dilatancy term was added to the dilatancy index equation proposed by Bolton (1986) to account for the difference between intact and remolded peak friction angle. The resulting equation suggests that dilatancy caused by diagenesis and by density are both suppressed with increasing confining pressure, which has important implications for the design strength of natural
deposits under heavy surcharge loads. A profile of in situ peak friction angle with depth is established from the test results and compared with values estimated from empirical relationships.

The diagenesis-dilatancy term was generalized as a function of age based on a dataset of triaxial compression test results for ten different uncemented, predominantly quartz sands. Strong evidence was shown that dilatancy due to diagenesis increases with age, and that a model including age and confining pressure terms significantly improved predictions over a model with no age term. Therefore an age-dilatancy model was proposed. It was also shown that other properties such as density have little influence on dilatancy due to age. Because age of natural deposits is often difficult to accurately determine, a MEVR-dilatancy model was also proposed based on the framework of the age-dilatancy model.

The age-dilatancy and MEVR-dilatancy equations were recommended to estimate intact peak friction angle from remolded peak friction angle or for predicting loss of strength during a disturbance or under large surcharges provided reliable in situ peak friction angle estimates are available. General models for estimating peak strength are implied by the age dilatancy and MEVR dilatancy equations and can be used once the model is validated with the data presented in this study and the data compiled by Bolton (1986).

Relationships for predicting the change in small strain shear modulus ($G_{\text{max}}$) or shear wave velocity ($V_s$) with time are reviewed. The $G_{\text{max}}$-time relationship proposed by Afifi and Richart (1973) and the MEVR-time relationship proposed by Andrus et al.
(2009) are related using a term called velocity ratio \( VR \), which is the ratio of \( V_s \) at a given time relative to its value in a deposit of similar density at a selected reference age. VR datasets were established from laboratory tests conducted on remolded sands and from laboratory tests conducted on intact sands. The VR datasets were combined to propose a VR-time relationship intended for natural sands.

The proposed VR-time relationship based on laboratory results was compared with the VR-time relationship based on in situ VS and penetration resistance measurements implied by MEVR. The laboratory based relationship suggested a 3% change in VR for each ten fold change in age, while the field test based relationship suggested a 8% change with each ten fold change in age. It is found that much of the difference in the slope of the laboratory and field based VR-time relationships can be explained by the difference in fines content of the VR laboratory cases and VR field cases, which provides strong evidence for an influence of fines content on diagenesis. Much closer agreement between the VR-time relationships of field and laboratory cases with clean sands only is observed. The results indicate that field and laboratory based VR-time relationships can be used as indices for degree of diagenesis, provided the influence of fines content is accounted for.

The preliminary results of a numerical study to predict the response of a Pleistocene age natural sand deposit at the CREC site during an in situ liquefaction experiment involving one of the NEES@UTexas mobile field shakers are presented. A plasticity model intended for earthquake engineering applications, was used for the Pleistocene sand deposit. Calibration of the model required considerably adjusting one of
three main model inputs, called the contraction rate parameter, using the procedure recommended by Boulanger and Ziotopoulou (2015) due to the relatively low density and high predicted cyclic strength of the CREC sand.

The simulation predicted concentrations of cyclic shear strain, cyclic stress ratio, and excess pore pressure that were located near the corners of the mobile shaker base plate during loading, and tended to produce a biased accumulation of shear strain toward either side of the sensor area. Below the base plate and within the zone where liquefaction sensor were installed at CREC, the excess pores pressure ratio was predicted to reach a maximum value of 12% and 18% at respective depths of 2.7 m and 3.3 m in the Pleistocene deposit. The prediction of low excess pore pressure buildup agrees with the limited field observations that were available to the author at the writing of this dissertation.
DEDICATION

This dissertation is dedicated to my parents in recognition of their love, support, and inspiration

And to my fiancé, who demonstrated great patience and kept me motivated during my graduate studies
ACKNOWLEDGMENTS

I would like to express the utmost gratitude to my advisor Dr. Ronald Andrus for encouraging me to pursue and develop an appreciation for research when I was an undergraduate student and for providing me with the opportunity to continue conducting research as a graduate student. I could not have achieved my research and professional goals without his positivity, foresight, kindness, and insight. Most of all, I appreciate his help in maintaining my focus on the task at hand and instilling me with confidence when I was lacking it.

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Kenneth Stokoe, Brady Cox, and Farnyuh Menq of University of Texas at Austin for providing information about sensor locations during in situ liquefaction testing at Clemson’s Coastal Research and Education Center; and finally I would like to thank the Itasca Consulting Group, Inc. for providing me with a free two month lease of FLAC 7.0.

Finally, this research was supported in part by the National Science Foundation, under the NSF Grant No. 0751278 and Grant No. CMS–0556006 which is greatly appreciated. Any opinions, findings, conclusions, or recommendations are made by the author of this dissertation and do not necessarily reflect the views of the National Science Foundation. Additional support was provided by the Aniket Shrikhande Memorial Assistantship fund. I would like to thank the Aniket Shrikhande family for providing me this support.
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CHAPTER 1
INTRODUCTION

1.1 Background

Static and dynamic characteristics of uncedented sands used in stability and deformation analyses such as peak static shear strength, small strain dynamic stiffness, normalized shear modulus and damping relationships, and cyclic shear strength are commonly predicted from intrinsic soil properties, state variables, laboratory measurements and/or in situ measurements when they cannot be measured directly. The empirical equations used in these predictions are usually calibrated from the results of laboratory tests conducted on unaged or freshly deposited sands or from field tests conducted in engineered fills or Holocene deposits and are not always appropriate for older deposits. Therefore, correction factors or appropriately calibrated parameters should be applied to existing models for more accurate evaluation of static and dynamic properties in natural deposits and the influence of aging processes, known collectively as diagenesis.

Site specific or local diagenesis correction factors/calibrated parameters are rarely determined for uncedented natural sands in geotechnical engineering practice, due to the difficulty and expense involved in collecting intact samples. Instead, the effects of diagenesis can be estimated from deposit age. For example:

- Afifi and Richart (1973) proposed an age correction factor for the small strain shear modulus ($G_{\text{max}}$) of sands and clays from the results of resonant column tests performed on specimens aged in the laboratory. The results were

- Seed (1979) proposed an age correction factor for the cyclic shear strength of sands, which were subsequently updated by Arango et al. (2000), Hayati and Andrus (2008), and Hayati and Andrus (2009), from the cyclic shear strengths of intact natural sands and from field-based estimates of cyclic shear strength. Hayati and Andrus (2008) termed this age or diagenesis correction factor as $K_{DR}$.

- Kulhawy and Mayne (1990) proposed an age correction factor for the standard penetration resistance of sands from blow counts in natural sands.

- Zhang et al. (2005) recommended predictive equations for the normalized shear modulus and damping relationships of Quaternary soils, as well as for Tertiary and older soils, from results of resonant column and torsional shear tests on intact natural specimens.

Usually, considerable judgment or local expertise is needed to determine the age of natural deposits and to determine if previous seismic events induced liquefaction, which could cause older deposits to lose some or all of their static and dynamic resistance associated with diagenesis. The ratio of measured shear wave velocity ($V_S$) to estimated $V_S$ computed from penetration resistances with relationships for recently deposited sands (MEVR) proposed by Andrus et al. (2009) is a promising proxy variable to quantify the degree of aging or diagenesis because (1) MEVR is sensitive to prior strain history, (2) it
is based on common engineering measurements rather than inferred from site stratigraphy, and (3) it is site specific. MEVR has been successfully applied in previous studies as a proxy for age in estimating $K_{DR}$ (Andrus et al. 2009, Hayati and Andrus 2009, Heidari and Andrus 2012).

1.2 Purpose of this research

The main purpose of the research presented in this dissertation is to develop a better understanding of the static peak strength, the small strain dynamic stiffness, and the in situ dynamic response of natural uncemented predominantly quartz sands. The motivation for the research is summarized in this section.

Presently, relatively little is known about the importance of diagenesis in predicting the peak shear strength (or peak friction angle) and dilatancy of natural sands, compared to freshly deposited sands. It has been speculated that the effects of diagenesis on other characteristics of natural sands (i.e., small strain dynamic stiffness, static and dynamic stress-strain response, cyclic shear strength) are at least in part caused by mechanisms that enhance frictional resistance and dilatancy such as enhanced interlocking, stress homogenization, overconsolidation, and low amplitude cyclic loading (Mesri et al. 1990; Schmertmann 1991; Yudhbir and Rahim 2001). The hypothesis that there is a relationship between peak shear strength and age or MEVR is investigated in this dissertation.

The predictive $G_{max}$-time relationship first proposed by Afifi and Woods (1971) concerned specimens that were held under isotropic confining pressures for durations of
1-6 months, and are thus best suited to predicting changes in $G_{\text{max}}$ occurring over relatively short time scales for which the dominant mechanism of diagenesis is secondary compression (i.e. the lifetime of an engineering structure). There has previously been no comprehensive attempt to extend the relationship to natural sand deposits.

The MEVR-time relationship proposed by Andrus et al. (2009) quantifies the increase in $V_s$ due to diagenesis, however there has presently been no attempt to compare the $G_{\text{max}}$-time relationship of Afifi and Woods (1971) and MEVR-time relationship.

The correction factor $K_{\text{DR}}$ used to account for the influence of diagenesis on liquefaction resistance is based on the cyclic resistance ratio of intact specimens measured in the laboratory cyclic triaxial compression tests. However, relatively little is known regarding the in situ large strain dynamic response of natural soils.

1.3 Objectives

The specific objectives of this dissertation are to:

1. Present the results of monotonic triaxial compression tests conducted on intact specimens retrieved from a Pleistocene age uncemented natural sand deposit and remolded specimens prepared with matching densities, quantify any differences in strength-dilatancy behavior, and compare the results with selected empirical methods for estimating the peak strength of sands.

2. Establish a dataset of triaxial compression test results in which the peak strengths of high quality intact specimens and corresponding remolded specimens are compared.
3. Propose a predictive relationship for estimating the peak strength or dilatancy due to diagenesis as a function of age or MEVR.

4. Establish a dataset of $V_s$ or $G_{\text{max}}$ age correction factors obtained from (a) laboratory aging studies and (b) intact natural sand specimens and remolded specimens prepared with matching densities.

5. Propose a predictive $V_s$-time relationship for uncemented natural sands and compare it with the predictive relationship for MEVR.

6. Present preliminary results of a numerical study performed to predict the response of a Pleistocene age uncemented sand deposit during an in situ liquefaction test.

1.4 Organization

This dissertation is organized into eight chapters. The introduction is presented in the current chapter, Chapter 1. Presented in Chapter 2 is a review of investigations concerning the change in peak shear strength and small strain dynamic stiffness of sands with time. Chapter 3 presents results of triaxial compression tests performed on intact Pleistocene-age sand specimens and remolded specimens prepared to matching densities. Based on the results a relationship for estimating dilatancy of the intact specimens due to diagenesis is proposed. Chapter 4 extends the relationship proposed in Chapter 3 to a larger set of triaxial compression test results conducted on intact and remolded sand specimens and proposes general equations for dilatancy due to diagenesis based on age and MEVR. Chapter 5 relates the age and MEVR dependent diagenesis dilatancy relationships to corresponding relationships for static peak shear strength. Chapter 6
compares the age correction factors for small strain dynamic stiffness obtained from laboratory aging studies, intact natural specimens, and the MEVR-time relationship. Chapter 7 presents the preliminary results of a numerical study performed to predict the response of a Pleistocene age uncedented sand deposit during an in situ liquefaction test. Finally, the major conclusions of this dissertation and the recommendations for future research are summarized in Chapter 8.
CHAPTER 2

REVIEW OF INVESTIGATIONS ON THE CHANGE IN STATIC PEAK STRENGTH AND SMALL STRAIN DYNAMIC STIFFNESS OF SANDS WITH TIME

2.1 Introduction

Presented in this chapter is a review of research performed to investigate and quantify the influence of age on the peak static strength and small strain dynamic shear modulus (or shear wave velocity) in uncemented natural sand deposits. The results largely pertain to clean predominantly silica sands, however results pertaining to sands containing different mineralogies or fines are referenced when needed.

2.2 Peak strength of natural sands

It is widely known (e.g., Taylor 1948; Bishop 1954; Rowe 1962; Bolton 1986) that the peak shear strength of freshly deposited granular soils consists of two components: 1) the critical state component, which is associated with the shear stress needed to overcome interparticle friction, particle rearrangement, and grain crushing; and 2) the dilatancy component, which is associated with the shear stress needed to expand the soil enough to accommodate shearing. The critical state strength is a function of intrinsic soil properties (e.g., mineralogy, grading, shape, and texture of component particles), whereas dilatancy is a function of intrinsic and state properties (e.g., density, confining pressure, and soil fabric).
The dilatancy of granular soils is governed by the number and nature of grain-to-grain contacts in an assembly of particles and distribution of stresses at the interparticle contacts (Rowe 1962). Density is recognized as the most important influence on the state of packing and dilatancy; however particle shape, overconsolidation ratio, depositional method, and strain history are other factors that influence dilation for a given density (Rowe 1962; Oda 1972; Yudhir and Rahim 2001; Guo and Su 2007). Confining pressure suppresses the dilatancy caused by these factors because soil particles are slightly deformable and can crush at the particle asperities.

Research to quantify the influence of diagenesis on peak shear strength of sands has been limited because it is very difficult to obtain undisturbed samples of uncemented sands from natural deposits and because peak shear strength is usually interpreted from in situ field test measurements (e.g., penetration resistance, flat plate dilatometer horizontal stress index).

Daramola (1980) compared the monotonic triaxial compression behavior of a dense unaged isotropically consolidated Ham River sand specimen with equally dense specimens that were held under a constant isotropic confining pressure for 10, 30, and 150 days (5 months). The specimens generally exhibited a stiffer stress-strain response, stronger tendency towards dilation, and slight increase in peak friction angle with increasing age. Presented in Fig. 2.1(a) and Fig. 2.1(b) are plots of the peak secant friction angle ($\phi'$) and the peak secant shear strength ($\tau_p$) evaluated from the stress-strain plots of the four specimens presented by Daramola (1980) with time since deposition ($t$), assuming an initial age of 3 days (time required for back pressure saturation.
Figure 2.1: Values of (a) $\phi'$ and (b) $\tau_p$ for dense Ham River sand after consolidation periods of 3 days, 13 days, 33 days, and 153 days (assuming a 3 day period of back pressure saturation). Data from Daramola (1980).
and consolidation). Therefore, the other $\phi'_p$ and $\tau_p$ values are plotted at adjusted times of 13, 33 and 153 days. The $\phi'_p$ and $\tau_p$ values generally increased due to an increasing duration of isotropic consolidation from initial values of $38.6^\circ$ and 518 kPa to values of $39.2^\circ$ and 532 kPa after 153 days. The best fit relationships between the $\phi'_p$ and $\tau_p$ and $(t)$ are also shown in Figs 2.1(a) and 2.1(b), which indicate average increases of about $0.4^\circ$ and 10 kPa for every ten fold change in age, or average percentage increases of about 1% and 2% for every ten fold change in age with respect to their initial values of $38.6^\circ$ and 518 kPa.

Results of similar laboratory aging studies conducted on quartz sand specimens with isotropic consolidation periods ranging from one week to one month (Human 1992; Lam 2000; Yudhibir and Rahim 2001) also indicate increases in strength and the tendency towards dilation, and 0 or < 1% increase in peak friction angle relative to their initial values after consolidation.

Results of triaxial compression tests performed on high quality intact specimens retrieved from natural deposits and remolded specimens prepared with matching densities provide conflicting evidence regarding the influence of deposit age on peak strength and dilatancy of granular soils. For example, Ghionna and Porcino (2003) found that a recently deposited marine silica sand and corresponding remolded specimens exhibited < 1° difference in peak secant friction angles interpreted from modified Mohr Coulomb $(q - p')$ failure envelopes. Christoffersen and Lunne (1982) observed a 3-5° difference between the peak secant friction angle values of intact specimens retrieved from a
Holocene fluvial deposit and corresponding remolded specimens prepared with matching densities (initial relative density within 5% of intact specimen). Mimura (2003) found that two Holocene age silica sand specimens exhibited \( \approx 2-4^\circ \) greater peak secant friction angle and proportionally greater maximum angle of dilation than corresponding remolded specimens but also found that two other Holocene sands predominantly composed of igneous rock fragments exhibited the same peak strength and dilatancy as remolded specimens. The results of these studies provide evidence of gradual improvement in the peak shear strength and dilatancy of natural silica sands, but also suggest that the improvement is influenced by mineralogy.

Tests conducted on Pleistocene and Tertiary locked sands provide strong evidence of the eventual influence of diagenetic processes on the microfabric, peak strength, and dilatancy of quartz sands, but are not directly comparable to remolded sands because they are too dense to be reproduced in the laboratory (Dusseault and Morgerstern 1979; Palmer and Barton 1987; Cresswell and Powrie 2004). The peak friction angles of locked sand specimens are much greater than values typically reported for freshly deposited sands due to both interlocking and greater maximum dilatancy at failure (Cresswell and Powrie 2004).
2.3 Small strain stiffness or shear wave velocity of natural sands

The dynamic small-strain shear modulus \( G_{\max} \) of natural sands can be reliably evaluated in situ from the measured shear wave velocity \( V_s \) of low amplitude seismic waves at shearing strains (\( \gamma \)) around \( 10^{-6} \) via field testing (e.g. seismic crosshole, seismic downhole, spectral analysis of surface waves, suspension logger, etc.). \( G_{\max} \) is also commonly evaluated from: (1) laboratory tests (e.g. resonant column, or bender elements) on remolded clean sands, (2) empirical relationships between \( G_{\max} \) and state variables based on remolded clean sands (e.g. Hardin and Richart 1963, Jamiolkowski et al. 1995), or (3) empirical relationships between penetration resistance (\( q \), or \( N_{60} \)) and \( V_s \) (e.g. Fear and Robertson 1995, Hagazy and Mayne 1995, Andrus and Stokoe 2000, Andrus et al. 2004). Applying \( G_{\max} \) or \( V_s \) determination of freshly remolded sands to field conditions via the former two approaches without adjusting for the effects of aging processes commonly results in underprediction of field measured \( V_s \) (Yokota et al. 1985, Yasuda and Yamaguchi 1985). Similarly \( V_s \) estimated from penetration-\( V_s \) relationships have been found to underpredict measured shear wave velocity in Pleistocene and Tertiary sediments (Andrus et al. 2009). Therefore, procedures to estimate the effects of aging processes on \( V_s \) of remolded sands or \( V_s \) estimated from penetration-\( V_s \) relationships have been proposed. These procedures are reviewed in this section.

2.3.1 Age effects on \( G_{\max} \) and \( V_s \) of remolded sands

Results of resonant column tests published by Afifi and Woods (1971) demonstrated an increase in \( G_{\max} \) of dry sands with time under a constant confining
pressure during aging periods of up to 430 days. Negligible changes in void ratio occurred during the aging period, implying an increase in $G_{\text{max}}$ due to age.

The increase in $G_{\text{max}}$ of dense Ottawa sand with time since deposition ($t$), under an isotropic confining stress of 207 kPa is reproduced from Afifi and Woods (1971) in Fig. 2.2. It is seen that $G_{\text{max}}$ increases from about 176 to 184 MPa. Based on similar results for cohesive and cohesionless soils, Afifi and Richart (1973) suggested that the age-corrected value of $G_{\text{max}}$ ($G_t$) could be estimated from $G_{\text{max}}$ when $t = 1000$ minutes ($G_{1000}$) and the slope of the relationship between $G_{\text{max}}$ and $\log t$, leading to the introduction of the following $G_{\text{max}}$-time relationship:

$$
\frac{G_t}{G_{1000}} = 1 + N_G \log_{10} \left( \frac{t}{t_{1000}} \right)
$$

(2.1)

where $G_t/G_{1000}$ is an age correction factor, $N_G$ is the increase in $G_t/G_{1000}$ for each ten fold change in $t$ (typically reported as a percentage value), and $t_{1000}$ is an assigned reference age of 1000 minutes. In Fig. 2.2, $G_{1000}$ of 180 MPa is indicated. Therefore, $N_G$ is the slope of the line passing through points for which $t > t_{1000}$, divided by the reference value of 180 MPa. As seen in Fig. 2.2, such a fitting predicts a $N_G$ value of 1.04%.

It had been hypothesized (Mesri et al. 1990) and later confirmed experimentally through both laboratory and discrete element studies (Wang and Tsui 2009, Gao et al. 2013) that secondary compression or creep effects (i.e., contact-force homogenization) account for the change in $G_{\text{max}}$ and $V_s$ of cohesionless soils observed due to a prolonged period of confinement.
Mesri et al. (1990) suggested that soil compression indices (i.e., compression index \( C_c \) and swell index \( C_s \)) influence the magnitude of secondary compression effects observed with time. Thus, a typical range of \( N_G \) for clean silica sands is 1-5% (Mitchell 2008) whereas \( N_G \) values for sands with compressible particles (e.g. carbonate, micaceous, and silty sands) have been found to exhibit larger \( N_G \) values than silica sands (Anderson and Stokoe 1978, Høeg et al. 2000, Jamiolkowski et al. 2003).

Anderson and Stokoe (1978) reported that, in actuality, \( V_s \) increases linearly with time whereas \( G_{\text{max}} \) increases in a slightly nonlinear fashion, indicating the following \( V_s \)-time relationship,

\[
\frac{V_{s,t}}{V_{s,1000}} = 1 + N_{VS} \log_{10}(t/t_{1000})
\]

where \( V_{s,t} \) and \( V_{s,1000} \) are values of \( V_s \) measured at times \( t > t_{1000} \) and 1000 minutes, respectively, and \( N_{VS} \) is the increase in \( V_{s,t}/V_{s,1000} \) for each ten fold change in \( t \). Figure 2.3 illustrates the relationship between \( V_s \) with time interpreted from the data in Fig. 2.2. A better coefficient of correlation \( (r^2) \) is obtained with \( V_s \) than with \( G_{\text{max}} \). Error due to the linear approximation of \( G_s/G_{1000} \) is not highly significant for \( N_G < 25\% \). The \( N_{VS} \) value obtained from the data plotted in Fig. 2.3 based on Eq. 2.2 is 0.52\% or half of the corresponding \( N_G \) value.
Figure 2.2: Change in $G_{\text{max}}$ of dense Ottawa sand with time since deposition under a constant confining pressure of 207 kPa (reproduced from Afifi and Woods, 1971).

Figure 2.3: Change in $V_s$ of dense Ottawa sand with time since deposition under a constant confining pressure of 207 kPa (after Afifi and Woods, 1971).
The terms $N_{VS}$ and $N_G$ are related for a given reference age ($t_R$). For example, given $t_R = 1000$ minutes and using the following relationship between $G_{\text{max}}$ and $V_S$,

$$G = \rho V_S^2$$

(2.3)

(where $\rho$ is soil density) a relationship between $N_G$ and $N_{VS}$ can be derived, which is given by,

$$N_{VS} = \frac{-1+\sqrt{1+11N_G}}{11} \quad (\text{for } t_R = 1000 \text{ minutes})$$

(2.4)

The derivation assumes that the change in density for sands is very small between times $t_{1000}$ and $t$. Afifi and Woods (1971) and Anderson and Stokoe (1978) showed that reduction in density of Ottawa sand during confinement accounted for no change in $G_{\text{max}}$ for an aging period of 70 to 430 days, therefore the assumption used to derive Eq. 2.4 is reasonable.

Comparisons of $G_{\text{max}}$ of carefully collected intact samples with $G_{\text{max}}$ of remolded samples prepared with matching densities indicate that Eq. 2.1 and a typical $N_G$ value of 2% provide reasonable age correction factors for engineered fills (Yamashita et al. 1997, Cha and Cho 2007) but underestimate the age correction factors in older natural sediments (Tokimatsu et al. 1986, Fahey 1998, Kiyota et al. 2009b). The extrapolation of the estimating procedure described above to natural deposits is considered in Chapter 6.

2.3.2 Age effects on penetration resistance-$V_S$ relationships

Andrus et al. (2009) investigated a quantity they called measured to estimated velocity ratio (MEVR): the ratio of measured clean sand equivalent, overburden stress
corrected shear wave velocity \([ (V_{s1})_{cs} ]\) to \( (V_{s1})_{cs} \) estimated from \( q_c-V_S \) or \( N_{60}-V_S \) relationships developed for Holocene age soils by Andrus et al. (2004) and Andrus et al. (2007).

Presented in Figure 2.5 is the variation of MEVR with time from Andrus et al. (2009) based on a database of 91 sets of penetration resistance \((q_c \text{ or } N_{60})\), \(V_S\), and age (i.e., the time since deposition or last critical disturbance) from Holocene, Pleistocene, and Tertiary sands deposits and the \(q_c-V_S\) or \(N_{60}-V_S\) relationships developed by Andrus et al. (2004). It is seen that the MEVR values correlate strongly with age. The empirical relationship between MEVR and age \((t)\) was expressed by (Andrus et al. 2009):

\[
\text{MEVR} = 0.0820 \log(t) + 0.935 \tag{2.5}
\]

where \(t\) is in years.

Based on Eqn. 2.5, values of measured \(V_S\) increase by about 8\% per log cycle of time relative to estimated \(V_S\) and \(\text{MEVR} = 1\) when \(t = 6.2\) years. Therefore the \(q_c-V_S\) or \(N_{60}-V_S\) relationships developed by Andrus et al. (2004) generally underpredict measured \(V_S\) for ages greater than 6.2 years.

MEVR or related terms have primarily been thought of as an indication of aging processes (Schnaid and Yu 2007, Andrus et al. 2009, Hayati and Andrus 2009, Heidari and Andrus 2012). MEVR is also a factor that can be used to correct \(q_c-V_S\) or \(N_{60}-V_S\) relationships intended for Holocene age soils. The latter use implies that Eq. 2.5 roughly quantifies the increase in \(V_S\) with age relative to a sand deposit of 6.2 years. This definition of MEVR can be expressed in a form similar to Eq. 2.2. That is,
Figure 2.4: Variation of measured to estimated velocity ratio with time for sands based on the relationships by Andrus et al. (2004a) as presented by Andrus et al. (2009).
MEVR = \frac{\left( V_s \right)_{t}}{\left( V_s \right)_{6\text{years}}} = 1 + 0.082 \log \frac{t}{6.2 \text{ years}} \tag{2.6}

2.3.3 General \( V_s \)-time relationship

Eqs. 2.2 and 2.6 represent two \( V_s \)-time relationships that have been applied to sands. A general \( V_s \)-time relationship can be expressed as,

\[ \frac{V_{s,t}}{V_{s,R}} = 1 + N_{VS} \log \frac{t}{t_R} \tag{2.7} \]

where \( V_{s,t} \) is shear wave velocity at time \( t > t_R \), \( V_{s,R} \) is shear wave velocity at a reference age of \( t_R \), \( N_{VS} \) is the rate increase of \( \frac{V_{s,t}}{V_{s,R}} \), and \( t \) is time since deposition or last critical disturbance.

From this point the ratio \( \frac{V_{s,t}}{V_{s,R}} \) will be called the velocity ratio (VR) indicating that it is the factor used to correct \( V_s \) for the influence of aging processes.

In summary, the equations for VR based on \( V_s \) of freshly remolded sands that are 1000 min or 0.002 years old (VR\(_{0.002}\)) and on penetration resistance-\( V_s \) relationships that are for 6.2 year old deposits (VR\(_{6.2}\)) are given by,

\[ \text{VR}_{0.002} = \frac{V_{s,t}}{V_{s0.002}} = 1 + N_{VS} \log \frac{t}{0.002 \text{ years}} \tag{2.8a} \]

and

\[ \text{VR}_{6.2} = \frac{V_{s,t}}{V_{s6.2}} = 1 + N_{VS} \log \frac{t}{6.2 \text{ years}} \tag{2.8b} \]
respectively. It is noted that a middle of range $N_{VS}$ for Eq. 2.8a is about 1% and for Eq 2.8b is about 8%. The difference between $N_{VS}$ interpreted from laboratory and field measurements will be investigated in Chapter 6.

2.4 Summary

This chapter summarized previous investigations to quantify the influence of age on the static peak strength and dynamic small strain stiffness of predominantly clean silica sands.

Investigations concerning lab-fabricated specimens held under a constant confining pressure indicate that peak friction angle increases with time. Direct comparisons between the peak shear strength of intact specimens and remolded specimens prepared with equal densities provided evidence of gradual improvement in the peak shear strength and dilatancy of natural silica sands with time, but also suggest that the improvement is influenced by mineralogy.

The development of the $G_{\text{max}}$-time relationship proposed by Afifi and Richart (1973) for sands and clays was reviewed. A new relationship for the increase in $V_s$ with time corresponding to an increase in $G_{\text{max}}$ with time was introduced. Based on the new equation, it was found that the rate at which $G_{\text{max}}$ and $V_s$ increase with time is related for a given reference age. The term VR, or the ratio of shear wave velocity at a given time relative to its value at a reference time was introduced. It was shown that both the $V_s$-time relationship established from laboratory cases and MEVR-time relationship
established from field cases can be expressed with the same general equation, while reflecting their different normalization approaches.
CHAPTER 3
PEAK STRENGTH AND DILATANCY OF A PLEISTOCENE AGE SAND*

3.1 Introduction

This chapter summarizes laboratory investigations to characterize the monotonic peak strength and dilatancy of intact sand specimens retrieved from an un cemented Pleistocene-age deposit at the Coastal Research and Education Center (CREC), near Charleston, South Carolina. The results of 22 triaxial compression tests, 11 performed on intact specimens recovered with the in situ freezing and frozen core sampling method, and 11 performed on remolded specimens are summarized. The stress-strain response, peak shear strength, and dilatancy of the intact and remolded specimens are compared for confining pressures ranging from 17-138 kPa.

The hypothesis investigated through this research is that there is a quantifiable difference in the strength of natural and freshly deposited sands due to aging processes. The main objectives of the investigations are to characterize the peak strength and dilatancy of intact and remolded CREC sand specimens, to quantify any differences in strength-dilatancy behavior, and to compare the results with selected empirical methods for estimating the peak shear strength of sands.

*A similar form of this chapter has been submitted for publication in ASCE Journal of Geotechnical and Geoenvironmental Engineering; Esposito III, M. P. and Andrus, R. D. (2015). “Peak strength and dilatancy of a Pleistocene age sand”.
3.2 Site description

The CREC is an agricultural research facility owned by Clemson University. A map showing the locations of field testing and sampling at the CREC geotechnical investigation site is presented in Fig. 3.1. The field investigations included cone penetration tests with pore pressure measurements (CPTu), cone penetration tests with pore pressure and shear wave travel measurements (SCPTu), standard penetration tests (SPT), crosshole shear wave velocity tests, dilatometer tests (DMT), in situ liquefaction and dynamic modulus tests using a mobile field shaker, and in situ freezing and frozen ground sampling. Detailed descriptions of the field investigations, except for the mobile field shaker tests, are presented by Boller (2008), Hossain et al. (2014), and Esposito et al. (2014).

The surficial deposits at the CREC site generally consist of 0.6 m of silty sand overlying clean sand that extends to depths of about 4.4 m. The clean sand layer is most likely part of the upper sand facies of the 70,000 to 130,000 year old Wando Formation (McCarten et al. 1984; Weems et al. 2014), which is a prominent surficial deposit in the area. The groundwater table is located at 0.9-1.9 m below the ground surface and is generally at its highest in early spring and lowest in late summer.

The CREC site lies in an area where little to no surface manifestations of liquefaction were observed during the 1886 Charleston earthquake. The two nearest sites of major surface manifestations of liquefaction in 1886 are located 5 km and 8 km from the CREC site based on ground failure maps originally published by Dutton (1889) and later compared with surface geology in Heidari and Andrus (2012).
Figure 3.1: Map of the CREC geotechnical investigation site showing locations of field tests and frozen ground sampling (modified from Esposito et al. 2014).
3.3 Ground freezing and frozen core sampling

In situ ground freezing and frozen core sampling was employed at the location indicated in Fig. 3.1 to retrieve high quality intact sand cores between depths of 1.8 and 3.8 m (Esposito et al. 2014). Freezing prior to coring aids in the preservation of soil fabric by temporarily cementing the sand particles, thereby minimizing sample disturbance developed during drilling, retrieval, transportation, and handling.

A 50-mm diameter steel pipe installed at the center of the frozen ground sampling array was continuously filled with liquid nitrogen to radially freeze the surrounding soil during a 270-hr (13-day) period. Frozen sand was cored from the mass of frozen soil with a 76-mm inside diameter ice coring auger at the locations labeled S1-S5 in the enlarged view of the sampling location indicated in Fig. 3.1. Photographs of the ground freezing and sampling array and a frozen sample retrieved from the site are presented in Figs. 3.2 and 3.3, respectively. The length of frozen sand cores ranged from 200-500 mm. Retrieved cores were identified and measured, and then placed in coolers filled with dry ice to prevent melting. Later, the frozen cores were carefully wrapped with mylar to prevent sublimation and with bubble wrap to reduce vibration during transportation. A more detailed account of the frozen ground sampling procedure is given in Esposito et al. (2014).

The frozen core samples retrieved between depths of 2.0 and 3.6 m from sample holes S1 and S3 were used for the drained triaxial compression tests described in this study.
Figure 3.2: Photograph of the as-built ground freezing and sampling array. Frozen ground sampling locations are labelled S1-S5 (Esposito et al. 2014).
Figure 3.3: Frozen sample retrieved from an ice coring auger (Esposito et al. 2014).
3.4 Characteristics of CREC sand

The in situ characteristics of the sand between the depths of 1.5 to 4.0 m are summarized in Fig. 3.4. Depicted in Figs. 3.4(a)-3.4(c) are profiles of porewater pressure-corrected piezocone tip resistance ($q_t$), energy-corrected standard penetration resistance ($N_{60}$), and small-strain shear wave velocity ($V_S$) at the locations closest to the ground freezing location. The higher resolution $q_t$ profiles in Fig. 3.4(a) indicate significant variability with an overall trend of decreasing resistance with depth in the sand layer.

Presented in Fig 3.4(d) is a profile of measured to estimated shear wave velocity ratio (MEVR), which is an index for degree of aging processes or diagenesis in soil deposits (Andrus et al. 2009). Measured shear wave velocity is based on the $V_S$ plotted in Fig. 3.4(c). Estimated shear wave velocity is based on the cone tip resistance plotted in Fig. 3.4(a) for C5 using the CPT-$V_S$ relationship proposed by Andrus et al. (2004). MEVR ranges from about 1.1 to 1.6 and averages 1.4 within the depths of the frozen ground sampling zone. Based on the MEVR-time relationship derived by Andrus et al. (2009), a MEVR of 1.4 roughly corresponds to a 500,000 year old deposit; and MEVR of 1.1 roughly corresponds to a deposit that is 100 years old. These MEVR values provide additional evidence that at least the top 3.2 m of sand did not liquefy during the 1886 Charleston earthquake. They also suggest that there may have been cyclic strain accumulation and pore pressure build up sufficient to degrade the small-strain stiffness below the depth of about 3.2 m.

The variability exhibited in the $q_t$, $N_{60}$, and $V_S$ profiles shown in Fig. 3.4(a)-3.4(c) is similar to the variability exhibited in the plot of initial void ratio ($e_i$) of frozen
specimens presented in Fig. 3.4(e). The values of \( e_0 \) for the intact specimens were determined from frozen and dry weight measurements. Range and average in situ physical properties of the sand between the depths of 2.0 and 3.6 m are presented in Table 3.1.

Grain size distribution curves determined for 10 intact specimens are presented in Fig. 3.5. All 10 gradation curves indicate poorly graded fine sand (SP).

A qualitative investigation of the mineral composition of the sand was conducted by x-ray diffraction (XRD) on samples from depths of 2 and 3 m. The dominant mineral phase of both samples was found to be crystalline quartz. Plagioclase feldspar was identified as a possible, albeit minor, constituent mineral phase. Trace amounts of mica were also visually observed, but were not detected in the XRD tests.

A complete summary of quantitative and qualitative tests conducted with samples of CREC sand and results including minimum and maximum void ratio, specific gravity, and XRD are summarized in Appendix A.

3.5 Initial tests on Ottawa sand

A series of drained triaxial compression tests was performed on Ottawa sand, a common reference sand in geotechnical engineering research, prior to the series of tests performed on intact and remolded CREC sand. The tests were conducted for three reasons: (1) to practice with the triaxial testing equipment and procedures, (2) to obtain results comparable with published data on Ottawa sand; and (3) to compare results obtained using two different triaxial chamber sizes (1042 cm\(^3\) and 2725 cm\(^3\)). Results of the tests are summarized in Appendix B.
Figure 3.4: Profiles of (a) cone tip resistance, (b) energy-corrected standard penetration test blow count, (c) shear wave velocity, (d) measured to estimated shear wave velocity ratio, and (e) initial void ratio of the clean sand layer at CREC. (SPT blow count and shear wave velocity data from Boller 2008 and Hossain et al. 2014).
In situ physical properties of CREC sand at depths of 2.0-3.6 m.

<table>
<thead>
<tr>
<th>Characteristic/Property</th>
<th>Range</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical effective stress $\sigma'_0$ (kPa)</td>
<td>34-47</td>
<td>40</td>
</tr>
<tr>
<td>Initial void ratio, $e_0$</td>
<td>0.75-1.06</td>
<td>0.87</td>
</tr>
<tr>
<td>Cone tip stress, $q_t$ (MPa)$^a$</td>
<td>3.2-12.1</td>
<td>8.6</td>
</tr>
<tr>
<td>Shear wave velocity, $V_S$ (m/s)$^b$</td>
<td>172-239</td>
<td>211</td>
</tr>
<tr>
<td>Measured to estimated velocity ratio, MEVR$^c$</td>
<td>1.1-1.6</td>
<td>1.4</td>
</tr>
<tr>
<td>Coefficient of uniformity, $C_u$</td>
<td>1.7-2.1</td>
<td>1.9</td>
</tr>
<tr>
<td>Coefficient of curvature, $C_c$</td>
<td>1.2-1.8</td>
<td>1.6</td>
</tr>
<tr>
<td>Mean grain size, $D_{50}$ (mm)</td>
<td>0.18-0.19</td>
<td>0.19</td>
</tr>
<tr>
<td>Fines content, $F_C$ (%)</td>
<td>2-5</td>
<td>3</td>
</tr>
<tr>
<td>Specific gravity of soil solids, $G_s$</td>
<td>2.67-2.69</td>
<td>2.68</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{min}$</td>
<td>0.67-0.69</td>
<td>0.68</td>
</tr>
<tr>
<td>Maximum void ratio, $e_{max}$</td>
<td>1.14-1.16</td>
<td>1.15</td>
</tr>
</tbody>
</table>

$^a$CPTs C5 and RB5; corrected for pore water pressure acting behind the cone tip.

$^b$Based on seismic crosshole travel time measurements between B1-B2, and B2-B3 (Hossain et al. 2014).

$^c$Measured, stress-corrected shear wave velocity ($V_{S1}$) divided by estimated $V_{S1}$ based on normalized cone tip resistance ($q_{tN}$) and the relationship by Andrus et al. (2004): $V_{S1} = 62.6(q_{tN})^{0.231}$

**Figure 3.5:** Grain size distributions of ten frozen core specimens.
3.6 Drained triaxial compression testing

A series of drained triaxial compression tests were performed on eleven intact specimens (A1, A2, …D2) and eleven remolded specimens (RA1, RA2, …RD2) prepared to similar densities from trimmings of the intact specimens. The triaxial tests were conducted under effective confining pressures ($\sigma'_c$) of 17, 35, 69, and 138 kPa (2.5, 5.0, 10, and 20 psi). Table 3.2 summarizes the results of the triaxial tests, including values of relative density ($D_R$) for the test specimens. Qualitatively four intact specimens classify as loose with $D_R = 20$-38%, four classify as medium dense with $D_R = 64$-73%, and three classify as dense with $D_R = 82$-85%.

Intact specimens were prepared by sawing 142-mm long segments from frozen sand cores and then trimming to a diameter of 71-mm by allowing a thin outer layer of the sawed cores to thaw. Once in the triaxial cell, a thawing period was required to allow the pore ice to melt and the specimen to equilibrate with stress conditions imposed during thawing. Thawing was performed under isotropic temperatures and effective stresses smaller than the effective confining stresses applied during the consolidation stage ($\sigma'_c$) under drained conditions.

The change in height of five of the eleven intact specimens (A1, A2, B2, D1, and D2) was measured during the thawing period under effective confining stresses of 10, 21, or 35 kPa to estimate change in void ratio, because sample disturbance can occur if the in situ effective stress and static pore water pressure are larger than the effective stress and back pressure applied as the specimen thaws (Hofmann 1997). Height change measurements were not performed for every intact specimen because lowering the load
piston down upon the specimen cap can also risk disturbance. Under a thawing stress of 10 kPa, specimen A1 exhibited a large change in void ratio ($\Delta e_{\text{thaw}} = 0.02$) and specimen A2 exhibited a small change ($\Delta e_{\text{thaw}} < 0.01$). Thawing disturbance was also small ($\Delta e_{\text{thaw}} < 0.01$) for the specimens B2, D1 and D2, because the isotropic effective stresses applied during thawing (21-35 kPa) were close to the estimated in situ mean effective stresses (20-30 kPa) and because the static pore water pressure is low at the sampling depths. From these observations, the strains associated with thawing were not expected to influence large strain static properties.

Nearly all remolded specimens were prepared to the approximate intact specimen density by moist tamping in 3 to 6 layers. Moist tamping was used because it is well suited to preparing specimens over a wide range of relative densities. A limitation of moist tamping, however, is that it creates a more or less random orientation of paricles (Oda 1972), whereas natural sands tend to have particles arranged with the long axis oriented with a horizontal plane.

Studies indicate that remolded specimens prepared with preferred orientation towards a horizontal plane can be stiffer than specimens with randomly orientated particles, but exhibit similar peak strengths (Oda 1972; Wanatowski and Chu 2008). Moist tamping was considered appropriate for this study because peak shear strength is the main physical property of interest.

Frost and Park (2003) discouraged the use of moist tamping for dense specimens consolidated under low confining pressures. Thus, one dense remolded specimen was prepared by dry vibration in five layers on a shaking table (RA2), because the stress
required for moist tamping was believed to be much higher than the effective confining stress of 17 kPa applied during consolidation and shearing.

Both intact and remolded specimens were saturated by incrementally raising the pore pressure and effective cell pressure until the pore pressure coefficient $B$ was greater than 0.95 to facilitate accurate volume change measurements during consolidation and shearing. After back pressure saturation, the specimens were isotropically consolidated for 100 minutes and sheared in strain controlled monotonic loading at a rate of 0.07% per minute to a total axial strain of 20% under drained conditions. Volume change was measured by monitoring the amount of water expelled from the specimens with a graduated burette for up to 10-12% axial strain. A photograph of the automated triaxial test system used in this study is presented in Fig. 3.6.
<table>
<thead>
<tr>
<th>Specimen Name&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Depth (m)</th>
<th>Initial void ratio, $e_0$&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Initial relative density, $D_{R,0}$&lt;sup&gt;b&lt;/sup&gt; (%)</th>
<th>Axial strain at failure, $\varepsilon_a$ (%)</th>
<th>Volumetric strain at failure, $\varepsilon_v$ (%)</th>
<th>Effective confining pressure, $\sigma'_c$ (kPa)</th>
<th>Peak principal stress difference $(\sigma_1 - \sigma_3)_p$ (kPa)</th>
<th>Peak secant friction angle $\phi'_{p}$ (degrees)</th>
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</thead>
<tbody>
<tr>
<td>A1</td>
<td>2.21</td>
<td>0.83</td>
<td>69</td>
<td>5.8</td>
<td>-2.0</td>
<td>17.2</td>
<td>83.1</td>
<td>44.9</td>
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<tr>
<td>RA1</td>
<td></td>
<td>0.83</td>
<td>69</td>
<td>15.9</td>
<td>NA&lt;sup&gt;d&lt;/sup&gt;</td>
<td>17.2</td>
<td>53.1</td>
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<tr>
<td>A2</td>
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<td>85</td>
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<td>84.0</td>
<td>45.2</td>
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<tr>
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<td></td>
<td>0.72</td>
<td>91</td>
<td>7.3</td>
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<td>71.7</td>
<td>42.3</td>
</tr>
<tr>
<td>B1</td>
<td>2.04</td>
<td>0.75</td>
<td>85</td>
<td>7.0</td>
<td>-1.8</td>
<td>35.3</td>
<td>144.9</td>
<td>42.3</td>
</tr>
<tr>
<td>RB1</td>
<td></td>
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<td>85</td>
<td>9.3</td>
<td>-0.8</td>
<td>34.2</td>
<td>99.9</td>
<td>36.4</td>
</tr>
<tr>
<td>B2</td>
<td>2.53</td>
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<td>35.4</td>
<td>144.2</td>
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<td>67</td>
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<td>69.0</td>
<td>238.7</td>
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<td>249.9</td>
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<td>18.5</td>
<td>NA</td>
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<td>166.8</td>
<td>33.2</td>
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</tbody>
</table>

Table 3.2: Summary results of drained triaxial compression tests on intact and remolded CREC specimens.
Table 3.2: (continued) Summary results of drained triaxial compression tests on intact and remolded CREC specimens.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Depth (m)</th>
<th>Initial void ratio, $e_0$</th>
<th>Initial relative density, $D_{R,0}$</th>
<th>Axial strain at failure, $\varepsilon_a$</th>
<th>Volumetric strain at failure, $\varepsilon_v$</th>
<th>Effective confining pressure, $\sigma'_c$</th>
<th>Peak principal stress difference $\left(\sigma_1 - \sigma_3\right)_p$</th>
<th>Peak secant friction angle $\phi'_p$</th>
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<td>1.03</td>
<td>25</td>
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<td>0.1</td>
<td>69.1</td>
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<tr>
<td>RC4</td>
<td>1.04</td>
<td>23</td>
<td>19.0</td>
<td>NA</td>
<td>NA</td>
<td>68.4</td>
<td>172.6</td>
<td>33.9</td>
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<td>0.84</td>
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<td>69.7</td>
<td>247.9</td>
<td>39.8</td>
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<td>0.77</td>
<td>82</td>
<td>8.3</td>
<td>-1.1</td>
<td>137.8</td>
<td>457.8</td>
<td>38.6</td>
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<td>0.78</td>
<td>78</td>
<td>17.4</td>
<td>NA</td>
<td>NA</td>
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<tr>
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<td>136.7</td>
<td>333.7</td>
<td>33.3</td>
</tr>
</tbody>
</table>

\(^a\)Intact specimen names begin with A, B, C, and D; and corresponding remolded specimen names begin with RA, RB, RC, and RD.

\(^b\)Before thawing and consolidation.

\(^c\)Assuming no cohesion intercept (i.e., $c' = 0$).

\(^d\)Not available because volume change was not measured after the first 10-12% axial strain.
Figure 3.6: Photograph of the automated triaxial test system used in this study during the shearing stage of a test performed on remolded CREC sand.
3.7 Stress-strain response*

Presented in Fig. 3.7 are typical principal stress ratio ($\sigma'_{1}/\sigma'_{3}$) and volumetric strain ($\varepsilon_v$) versus axial strain ($\varepsilon_a$) relationships for intact specimens (closed symbols) and remolded specimens (open symbols) consolidated under $\sigma'_{c}$ of 69 kPa. The intact specimens behave as if they were denser than the corresponding remolded specimens, exhibiting higher peak stress ratios at lower values of $\varepsilon_a$ and a greater tendency towards dilation. It is evident from the plot of $\varepsilon_v$ versus $\varepsilon_a$ in Fig. 3.7(b) that the onset of dilation occurs at lower values of $\varepsilon_a$ for intact specimens than remolded specimens, which is mirrored by large differences in the stress-strain behavior in Fig. 3.6(a) after the onset of dilation. Fig. 3.7(a) also depicts a clear increase in the peak strength of both intact and remolded sand with increasing density, with both increasing at similar rates.

Compared in Fig. 3.8 are $\sigma'_{1}/\sigma'_{3}$ and $\varepsilon_v$ versus $\varepsilon_a$ relationships of intact and remolded dense CREC sand at $\sigma'_{c}$ of 35 and 138 kPa. For both intact and remolded specimens, stiffness, peak strength and dilatancy decrease under higher $\sigma'_{c}$. For the intact specimens the tendency towards dilation is diminished under $\sigma'_{c}$ of 138 kPa, but not fully suppressed. For the remolded specimens, the tendency towards dilation is fully suppressed. Fig. 3.8(a) also indicates that the difference between the peak strength of the intact and remolded specimens is greater under $\sigma'_{c}$ of 35 kPa than under $\sigma'_{c}$ of 138 kPa.

*Selected stress-strain results are presented in section 3.7. A complete set of stress-strain curves is presented in Appendix C.
Figure 3.7: Relationships of (a) stress ratio-axial strain and (b) volumetric strain-axial strain relationships measured in drained triaxial compression for intact and remolded specimens at an effective confining pressure of 69 kPa.
Figure 3.8: Relationships of (a) stress ratio-axial strain and (b) volumetric strain-axial strain for dense intact and remolded specimens under effective confining pressures of 35 and 138 kPa.
Compared in Fig. 3.9 are the values of $\sigma'_1/\sigma'_3$ at the beginning of dilation for all eleven intact specimens and seven remolded specimens plotted versus axial strain. The four other remolded specimens are plotted with the $\sigma'_1/\sigma'_3$ values observed at 16-20% strain because they contracted throughout shearing. It can be seen that intact and remolded specimens sustained average principal stress ratios of 3.57 and 3.40, respectively at the beginning of dilation. This observation indicates that only slightly larger stress ratios were required to initiate volumetric expansion of intact specimens. The difference in $\sigma'_1/\sigma'_3$ values at the beginning of dilation is small however, compared to results reported for natural sand specimens with locked or cemented granular fabrics (Cuccovillo and Coop 1999; Cresswell and Powrie 2004), implying that the strength of intact CREC sand is associated with dilation rather than interlocking or bonding.

### 3.8 Stress dilatancy relationship

The relationship between mobilized friction angle ($\phi'$), dilation angle ($\psi$), and the critical state friction angle ($\phi'_{\text{crit}}$) is useful for evaluating the peak strength of soils and as the general framework of a flow rule in constitutive modeling of soil plasticity (Bolton 1986; Yang and Li 2004). Based on Rowe’s (1962) stress-dilatancy relation, Bolton (1986) approximated a linear relationship between the peak friction angle $\phi'_p$, the maximum dilation angle ($\psi_{\text{max}}$), and $\phi'_{\text{crit}}$ with a general form given by

$$\phi'_p = \phi'_{\text{crit}} + A\psi_{\text{max}}$$

(3.1)
Figure 3.9: Mobilized principal stress ratio and axial strain level at the beginning of dilation.
where the term $A \approx 0.6$-$0.8$ for plane strain loading (Bolton 1986; Chakraborty and Salgado 2010) and $\approx 0.4$-$0.7$ for axisymmetric loading (Vaid and Sasitharan 1992; Yang and Li 2004; Frydman et al. 2007; Guo and Su 2007; Chakraborty and Salgado 2010).

The values of $\psi$ reported in this study are based on the definition proposed by Vermeer and de Borst (1984) for axisymmetric and plane strain loading:

$$\sin \psi = \frac{d\varepsilon_v/d\varepsilon_1}{2 - d\varepsilon_v/d\varepsilon_1}$$  \hspace{1cm} (3.2)

where $d\varepsilon_1$ and $d\varepsilon_v$ are the incremental changes in axial strain and volumetric strain, respectively.

Fig. 3.10 presents the relationship between $\phi'$ and $\psi$ for intact and remolded CREC sand. The $\phi'$ and $\psi$ values plotted with circles correspond to peak $\sigma'_{1}/\sigma'_{3}$ for specimens which failed during the first 10% axial strain. The $\phi'$ and $\psi$ values plotted with triangles correspond to $\sigma'_{1}/\sigma'_{3}$ values measured at 5% strain for specimens that had not yet reached a peak $\sigma'_{1}/\sigma'_{3}$ value. Shown for comparison in Fig. 3.10 are the relationships between $\phi'_{p}$ and $\psi_{\text{max}}$ interpreted from Bolton (1986) for clean silica sands and Guo and Su (2007) for Ottawa sand using $\phi' = \phi'_{p}$, and values of $A$ of 0.5 and 0.63, respectively.

It is observed in Fig 3.10 that the results of intact and remolded CREC sand follow the same stress-dilation relationship. This finding is in good agreement with the stress-dilation behavior of intact and remolded uncemented Thanet sand and locked Reigate Silver sand characterized by Ventouras and Coop (2009) and Cresswell and
Powrie (2004), respectively. The \( \psi \) values of the intact specimens are concentrated between 5-15\(^\circ\), whereas the \( \psi \) values of remolded specimens are generally less than 5\(^\circ\). Thus, the intact specimens are characterized by much stronger dilation at or prior to the peak shearing resistance.

The values of \( A \) and \( \phi'_{crit} \) predicted from the linear regression line in Fig. 3.10 are 0.730 and 33.4\(^\circ\), respectively. If the intact and remolded data are fitted independently, a small difference in the predicted value of \( A \) is observed and the predicted \( \phi'_{crit} \) values are found to be 33.5\(^\circ\) and 33.2\(^\circ\), respectively. These predicted values of \( \phi'_{crit} \) are typical for predominantly quartz sand and agree well with the angle of repose observed by slowly lifting a funnel filled with CREC sand (32\(^\circ\)-34\(^\circ\) in three trials). The predicted value of \( A \) is slightly higher than values proposed for clean silica sands. This may be explained by the larger confining stresses (Bolton 1986, \( \sigma'_{nf} = 1/3(\sigma'_{1f} + 2\sigma'_{3f}) = 150-600 \text{ kPa}; \) Guo and Su 2007, \( \sigma'_c = 100-500 \text{ kPa} \) used in determining \( A \) in prior studies. Linear regression of the CREC sand data for specimens consolidated at \( \sigma'_{c} > 35 \text{ kPa} \) predict a value of \( A = 0.68 \) and \( \phi'_{crit} = 33.2\(^\circ\), which agrees more closely with the results of Guo and Su (2007).
Figure 3.10: Variation of friction angle with the angle of dilation for intact and remolded specimens.
3.9 Peak friction angle

The peak secant friction angle of freshly deposited silica sand and its dependency on density and confining pressure were characterized by the dilatancy index \( I_R \), proposed by Bolton (1986) in an attempt to estimate \( \phi'_p \) from \( \phi'_{crit} \). The \( I_R \)–based equation originally proposed by Bolton (1986) for axisymmetric loading is rearranged in this study to give the following general expression for peak friction angle:

\[
\phi'_p - \phi'_{crit} = 0.5\psi_{max} = aI_R = \frac{aD_R}{100} \ln \left( \frac{\sigma'_R}{\sigma'_{mf}} \right) + b
\]

(3.3)

where \( D_R \) is relative density expressed as a percentage value, \( \sigma'_{mf} \) is mean effective stress at the peak principal stress ratio, \( a \) and \( b \) are empirical fitting constants, and \( \sigma'_R \) is a semi-empirical fitting constant related to compressibility of the constituent soil particles. Bolton (1986) predicted values of \( a = 3, \sigma'_R = 22,000 \) kPa, and \( b = -3 \) for Eq. 3.3 based on the \( \phi'_p \) values of 17 varieties of clean, predominantly silica sands with a typical estimating error of 1-2°. Within the dilatancy index framework, it is assumed that the constants \( a, \sigma'_R, \) and \( b \) are intrinsic soil properties (Salgado et al. 2000).

In Eq. 3.3 the term \( aD_R \) accounts for dilatancy due to density and the term \( \ln \left( \frac{\sigma'_R}{\sigma'_{mf}} \right) \) accounts for suppression of the dilatancy under high confining stresses. The constant \( b \) has a functional role of allowing \( \phi'_p - \phi'_{crit} = 0 \) at different combinations of \( D_R \) and \( \sigma'_{mf} \) as opposed to forcing a common intercept at \( D_R = 0 \) for any value of
\( \sigma'_{mf} \). Thus, \( \phi'_p = \phi'_{crit} \) should be taken when Eq. 3.3 returns a negative value, and \( \phi'_p = 45^\circ \) can be taken as a practical upper bound value (Bolton 1986).

The influence of \( \sigma'_{mf} \) on \( \phi'_p \) of loose, medium dense, and dense CREC specimens are compared in Fig 3.11. The peak friction angle values of intact specimens (\( \phi'_I \)) seen in Fig. 3.11 are 3.0-8.6\(^\circ \) greater than peak friction angles (\( \phi'_R \)) of corresponding remolded specimens, with an average difference (\( \phi'_I - \phi'_R \)) of 5.3\(^\circ \). It is also seen in Fig. 3.11 that the variation in \( \phi'_p \) with \( \sigma'_{mf} \) differs between intact and remolded specimens. The \( \phi'_I \) values decrease with \( \sigma'_{mf} \) at greater rates than the \( \phi'_R \) values, depicting a general trend of decreasing \( \phi'_I - \phi'_R \) with increasing \( \sigma'_{mf} \).

Plotted for each grouping in Fig. 3.11 is the best fit linear relationship between \( \phi'_p \) and \( \sigma'_{mf} \) assuming constant values of \( \sigma'_R \) and \( b \) as in Eq. 3.3, while allowing the term \( aD_R \) to vary for each group. The \( aD_R \) values derived for loose, medium dense, and dense remolded specimens are 0.9, 1.4, and 2.3, respectively, depicting an increase in dilatancy with \( D_R \). The remolded \( aD_R \) values are in good agreement with the values estimated by Eq. 3.3 for silica sands with \( a = 3 \) and average \( D_R \) values of 30%, 70%, and 85%, which are 0.9, 1.8, 2.6, respectively. As seen in Fig. 3.11, the \( aD_R \) values derived for loose, medium dense, and dense intact specimens are about 1.9 and 1.7 greater than the \( aD_R \) values of remolded specimens and estimated using Eq. 3.3 and \( a = 3 \),
Figure 3.11: Variation of peak secant friction angle with confining pressure (log scale) for intact and remolded specimens, grouped by relative density.
respectively, for a given density. Thus, the intact specimens are characterized by a
difference in $aD_R$ of about 1.8 when compared to freshly deposited clean silica sands.

3.10 Diagenesis dilatancy relationship

Figs. 3.10 and 3.11 illustrate a significant difference between the peak strength
and dilatancy of intact and remolded specimens, making Eq. 3.3 a poor predictor of $\phi'_p$
when both datasets are fitted with a common value of $a$. Such a fitting returns a
coefficient of determination ($R^2$) of 0.432 and a root mean square error (RMSE) of
2.71°. The $\phi'_p$ values are better predicted when different values of $a$ are used for intact
and remolded specimens ($R^2 = 0.868$, RMSE = 1.31°). However, this would imply that
the value of $a$ is not constant for a given soil and that the difference in $aD_R$ terms
changes with $D_R$, whereas the $aD_R$ terms in Fig. 3.11 indicate a constant difference.

Instead, Eq. 3.3 can be improved by adding an diagenesis-dilatancy constant $C_DZ$
to the term $aD_R$. That is,

$$
\phi'_p - \phi'_{cri} = \left(\frac{aD_R}{100} + C_DZ\right) \ln \left(\frac{\sigma'_R}{\sigma'_{mf}}\right) + b
$$

(3.4)

where $C_D$ is a term that approximates the dilatancy contributed by diagenesis, and $Z$ is a
variable that takes a value of 1 for intact specimens and 0 for remolded specimens.

Multiple linear regression of Eq. 3.4 with all values of $\phi'_p$ and $\phi'_{cri} = 33.4^\circ$
predicts values of $a = 2.41$, $\sigma'_R = 1510$ kPa, $b = -1.76$, and $C_D = 1.95^\circ$. The error in the
predicted \( \phi'_p \) values \( (R^2 = 0.946, \text{RMSE} = 0.8^\circ) \), and the subsets of values \( \phi'_I \) \( (R^2 = 0.973, \text{RMSE} = 0.5^\circ) \) and \( \phi'_R \) \( (R^2 = 0.814, \text{RMSE} = 1.1^\circ) \) are reasonable when compared to the database compiled by Bolton (1986).

Eq 3.4 implies that \( \phi'_I - \phi'_R \) is given by the expression:

\[
\phi'_I - \phi'_R = C_D \ln \left( \frac{\sigma'^R}{\sigma'^{mf}} \right)
\]  

(3.5)

Eq 3.5 is a term which approximates the dilatancy due to diagenesis and suppression of diagenesis-dilatancy with increasing confining pressure. The results indicate that both the \( C_D \) and \( aD_R \) are suppressed under a similar range of confining pressures. The relationship also implies that diagenesis effects may be completely diminished at \( \sigma'^{mf} > 1,500 \text{kPa} \).

Values of \( \phi'_I - \phi'_R \) determined in this study are plotted versus \( \sigma'^{mf} \), based on the average \( \sigma'^{mf} \) for each intact/remolded pair, and compared with Eq. 3.5 in Fig. 3.12. Small adjustments \((< 1^\circ)\) are made to the \( \phi'_R \) values to account for differences between intact and remolded \( D_R \) using Eq. 3.4. It is seen that ten of the eleven \( \phi'_I - \phi'_R \) values follow the relationship predicted by Eq. 3.5. One value, determined from specimen pair A2/RA2, plots significantly lower than the prediction of Eq. 3.5. The \( \phi'_I - \phi'_R \) values do not depict significant variation with relative density, besides the low \( \phi'_I - \phi'_R \) value observed for A2/RA2.
A profile of $C_D$ values back-calculated from Eq. 3.5 with the values of $\phi'_i - \phi'_R$, $\sigma'_{mf}$, and $\sigma'_R = 1510$ kPa is compared with the MEVR profile (Fig. 3.2d) for the CREC site in Fig 3.13. In Fig. 3.13, an $C_D$ value of zero corresponds to freshly deposited soil with no diagenesis-dilatancy and an MEVR of 0.7 roughly corresponds to an age of 1 day based on the MEVR-time relationships of Andrus et al. (2009). Therefore, the $C_D$ scale beginning at zero and MEVR scale beginning at 0.5 are based on similar reference values.

Reasonable agreement between the $C_D$ and MEVR profiles is observed in Fig. 3.13. The profiles of $C_D$ and MEVR both depict a range in which the values decrease slightly with depth, and a location within the profile wherein a minimum value is reached. It is also observed however, that $C_D$ reaches a minimum value at a shallower depth before increasing between depths of 3.0-3.5 m. The MEVR profile is not observed to increase with depth beyond the region with minimum MEVR. Thus, the difference in the $C_D$ and MEVR profiles likely reflects variability in the deposit at CREC.

From the results shown in Figs. 3.10-3.12 and Eqs. 3.4 and 3.5, it is concluded that the stress-dilatancy of sand can be characterized when the influence of diagenesis on the magnitude of peak strength and dilation is considered for a given state. Furthermore, the results shown in Fig. 3.13 indicate that dilatancy due to diagenesis can be estimated from MEVR. The observed mechanical behavior of the intact CREC specimens does not suggest a bonded or strongly locked fabric. Physical mechanisms such as the reorientation and the sliding of particles into more stable positions, resulting in light
Figure 3.12: Variation in $\phi'_f - \phi'_R$ with confining pressure (log scale), grouped by relative density.
Figure 3.13: Profile of $C_D$ values back-calculated with Eq. 3.5 compared with MEVR.
interlocking and a more stable distribution of contact stresses, better explain the stress-strain behavior of CREC sand.

3.11 Friction angle from empirical relationships

Presented in Fig. 3.14 is the profile of $\phi'_p$ determined from triaxial compression test results on intact specimens compared with profiles of $\phi'_p$ estimated from the cone tip resistances in Fig. 3.2(a), the shear wave velocities shown in Fig 3.2(c), and Eq. 3.4. Estimates of $\phi'_p$ from standard penetration test results are not shown because there are only two available blowcounts in the 1.8-3.8 m depth range. The values of $\phi'_p$ from intact specimens are adjusted to correspond to the estimated in situ $\sigma'_mf$ based on the Eq. (3.4), where a value $2 \times \sigma'_{vo}$ is used to approximate $\sigma'_mf$ as suggested by Mayne (2006). It is seen in Fig. 3.14 that the adjusted values of $\phi'_p$ range from 39.0° to 44.9° and decrease slightly with depth.

The estimated values of $\phi'_p$ based on cone and $V_S$ measurements are calculated from the average normalized cone tip resistance ($q_{c1N}$) at C5 and RB3, and the average effective overburden stress corrected shear wave velocity ($V_{S1}$) at B1-B2 and B2-B3 within ±0.15 m of the retrieval depth for each intact specimen. The $q_{c1N}$- and $V_{S1}$-based empirical equations by Uzielli et al. (2013), i.e., $\phi' = 25.0^\circ q_{c1N}^{0.10}$ and $\phi' = 3.9 V_{S1}^{0.44}$ (where $q_{c1N}$ is dimensionless, $V_{S1}$ is in m/s) are used for estimating $\phi'_p$ because they were derived from triaxial compression test results on high quality intact specimens retrieved mainly from natural silica sand deposits.
Values of $\phi'_p$ computed from Eq. 3.4 are based on $a = 2.41$, $\sigma'_k = 1,510$ kPa, $b = -1.76$, and $C_D = 1.95$ determined from multiple linear regression and $\phi'_{crit} = 33.4^\circ$. A value $2\times \sigma'_{vo}$ as an approximation of $\sigma'_{mf}$ and relative densities of the intact CREC specimens are also used to evaluate Eq. 3.4. Based on these inputs, the $\phi'_p$ values of the intact specimens are predicted with a RMSE of 0.5°.

As observed in Fig. 3.14, the values of $\phi'_p$ established from in situ frozen specimens are well predicted with $q_{c,IN}$, with a RMSE of 2.2°. The predictions based on $V_{S1}$ are considerably higher than the estimated values, with a RMSE of 4.5° in the top 3 m and 2.4° below 3 m. The values of $\phi'_p$ are likely over predicted by $V_{S1}$ because the small-strain properties of soil are more sensitive to degree of diagenesis than large strain properties and indexes, such as $\phi'_p$ and $q_{c,IN}$. As a result, $V_{S1} - \phi'_p$ relationships used in sands that are not recently deposited may require adjustment of $V_S$, which could be accomplished with a MEVR-based relationship (Andrus et al. 2009).

3.12 Summary

Results of a series of triaxial compression tests indicate that intact sand specimens sampled from a 70,000-130,000 year old deposit near Charleston possess greater peak frictional strength and dilatancy than freshly deposited specimens prepared at the same densities. The difference between intact and remolded peak strength decreases with increasing effective confining pressure and does not vary significantly with relative density. Based on mechanical behavior and visual observation of the retrieved specimens,
Figure 3.14: Profile of measured $\phi'_p$ adjusted using the in situ effective overburden stress, compared with profiles of estimated $\phi'_p$ based on relationships by Uzielli et al. (2013) and Bolton (1986).
increased dilatancy of the intact sand is likely due to light interlocking and a more stable distribution of contact stresses.

The influences of density and confining pressure on the peak friction angle and dilatancy of intact and remolded specimens were characterized using a revised version of the dilatancy index equation proposed by Bolton (1986) which incorporates a diagenesis-dilatancy term. The resulting model suggests that the dilatancy of freshly remolded specimens includes a density component that decreases with increasing confining pressure, whereas the dilatancy of intact natural specimens includes a density component and a diagenesis component which both decrease under increasing confining pressure. The results also suggest that natural uncemented sands may exhibit greater sensitivity to confining pressure than remolded sands due to gradual destruction of the soil fabric, and that diagenesis effects can be erased under large effective stresses.

The in situ peak friction angles of intact sand specimens determined from triaxial compression test results were compared with empirically predicted values based on relationships with cone tip resistance and small-strain shear wave velocity proposed by Uzielli et al. (2013). Cone tip resistance-based predictions were close to the in situ peak friction angles. The shear wave velocity-based predictions were generally much greater than the measured \( \phi' \) values, which is likely because small-strain properties are more sensitive to diagenesis than cone tip resistance or peak strength.
CHAPTER 4

PEAK STRENGTH AND DILATANCY OF UNCEMENTED, NATURAL SANDS

4.1 Introduction

A general equation for the diagenesis-dilatancy term proposed in Chapter 3 as a function of time since deposition \((t)\) or measured to estimated velocity ratio (MEVR), is investigated in this chapter. In Chapter 3 the difference between intact and remolded peak effective friction angles \((\phi'_I - \phi'_R)\) due to diagenesis was quantified with the equation:

\[
\phi'_I - \phi'_R = C_D \ln \left( \frac{\sigma'_R}{\sigma'_{mf}} \right)
\]

where \(\sigma'_R\) is an intrinsic property of a given soil, \(\sigma'_{mf}\) is mean effective confining stress at peak strength, and \(C_D\) is constant for a given set of aging processes occurring within a soil deposit over a specific duration of time. Therefore, the hypothesis to be tested is that the term \(C_D\) and \(t\) or MEVR are related (i.e., \(C_D = f(t)\) or \(f(\text{MEVR})\)).

The objectives of this chapter are to 1) establish a dataset of triaxial compression test results in which the peak friction angles of high quality intact specimens and corresponding remolded specimens are compared, 2) characterize the relationship between \(\phi'_I - \phi'_R\) and time or MEVR, and 3) generalize Eq. 4.1 as a continuous function of time or MEVR.
4.2 Dataset

A dataset of monotonic triaxial compression test results of aged and remolded sand specimens was established from 10 independent studies. Relevant information pertaining to each study, including location, site name, deposit type, age, and sampling method is summarized in Table 4.1. Eight studies compare the monotonic strength of specimens prepared from intact core samples that were obtained in natural and man-made deposits and remolded specimens consolidated for a standard length of time. Two supplementary studies by Daramola (1980) and Lam (2000) compare the monotonic strength of remolded specimens consolidated for a prolonged period of time and lab-fabricated specimens consolidated for a standard length of time.

Drained or undrained triaxial compression test results of intact and corresponding remolded specimens were directly compared in nine of the ten studies (Cases 1-2 and 5-10). Results pertaining to the Kidd site (Case 4) were compared with drained and undrained triaxial compression test results for remolded Massey specimens presented by Konrad and Pouliot (1997). It was deemed reasonable to use the results of remolded Massey specimens to compare with intact specimens retrieved from the Kidd site because the soil layers at both sites are clean sand deposits of the Fraser River with the same mineralogical composition, gradation, and index properties (Wride and Robertson 1999; Robertson et al. 2000).

Criteria were established for selecting results concerning intact specimens prepared from core samples to reduce uncertainty. These criteria include:
1. Intact sand cores were obtained by freezing in situ or were verified with accurate field measurements to maintain a similar density to that existing in situ.

2. The test specimens were saturated, clean (fines content, FC < 5%), predominantly quartzofeldspathic sand.

3. Remolded specimens were prepared with initial relative densities within ± 5% of the intact relative densities ($D_R$).

4. For drained tests, intact and remolded specimens were consolidated under the same mean confining stress ($\sigma_{mc}'$) before shearing.

5. For undrained tests, intact and remolded specimens achieved peak stress ratios under similar mean effective stresses ($\sigma_{mf}'$) (percent difference within 20%).

6. Age of specimens could be estimated to within ± 1 log cycle of time.

7. Repeat test results were averaged provided $t$, $D_R$, $\sigma_c'$, and $\sigma_{mf}'$ were the same.

Exceptions to Criterion 3 were permitted for four of the CREC specimens by adjusting $\phi_R'$ to account for differences in $D_R$, as discussed in Chapter 3. Exceptions to Criterion 5 were permitted for the results pertaining to the Massey site and Kidd site because results were available for a wide range of confining pressures and reasonable adjustments to peak friction angle values could be made to account for differences in $\sigma_{mf}'$. Adjustments were only made to $\phi_R'$ values and were not considered if there was a two-fold difference or more in $\sigma_{mf}'$.

Triaxial compression test results for clean, quartzofeldspathic sand cases are compiled in Table 4.2. Each entry in Table 4.2 summarizes the results of one intact
specimen and a corresponding remolded specimen. This compilation includes multiple results when numerous test results were available for a given case, resulting in a total of 30 intact-remolded pairs. Characteristics of each test including test type, sample preparation method (for remolded specimens), time since deposition or last major disturbance \( (t) \), fines content \( (F_c) \), mean grain size \( (D_{50}) \), relative density \( (D_R) \), estimated in situ mean effective stress \( (\sigma'_{m0}) \), minor principal effective stress during consolidation \( (\sigma'_{3c}) \), and mean effective stress at failure \( (\sigma'_{mf}) \) are presented in Table 4.2.

Further explanation regarding the cases and quantities comprising the database in Tables 4.1 and 4.2 are discussed in the following subsections.

4.2.1 Deposit age

The estimated time since initial deposition for each case study involving natural sands was used unless there was evidence of significant disturbance (i.e., severe liquefaction) in the region where it was sampled. For the Gioia Tauro site the time since a strong earthquake known to have caused liquefaction in the town of Gioia Tauro in 1793 (Facciorusso and Vannuchi 2003) was used. For the Kidd site the time since widespread liquefaction of the Fraser River Delta (Claque et al. 1997) was used.

The lower bound age of the Yodo River and Natori River deposits and the upper bound age of the Natori River deposit were not reported in the literature, as noted in Table 4.1. Lower bound estimates of 500 years were assumed for the Holocene-age Yodo River and Natori River deposits because the samples were obtained at considerable depth.
**Table 4.1:** Laboratory cases involving predominantly quartzofeldspathic sand used in this study.

<table>
<thead>
<tr>
<th>Study</th>
<th>Case #</th>
<th>Location</th>
<th>Site/Sand name</th>
<th>Deposit type</th>
<th>Deposit age, $t$ (years)</th>
<th>Method Sampled&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Daramola (1980)</td>
<td>1</td>
<td>Reference sand</td>
<td>Ham River sand</td>
<td>Fabricated</td>
<td>0.42</td>
<td>Lab</td>
</tr>
<tr>
<td>Christoffersen and Lunne (1982); Lunne et al. (2003)</td>
<td>2</td>
<td>Drammenford, Norway</td>
<td>Holmen</td>
<td>Fluvial</td>
<td>2k-3k</td>
<td>FP</td>
</tr>
<tr>
<td>Wride and Robertson (1999); Konrad and Pouliot (1997)</td>
<td>3</td>
<td>Fraser River delta, Vancouver</td>
<td>Massey</td>
<td>Deltaic</td>
<td>200</td>
<td>ISF</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Fraser River delta, Vancouver</td>
<td>Kidd</td>
<td>Deltaic</td>
<td>1.7k&lt;sup&gt;b&lt;/sup&gt;</td>
<td>ISF</td>
</tr>
<tr>
<td>Lee et al. (1999)</td>
<td>5</td>
<td>West Kowloon, China</td>
<td>West Kowloon reclamation</td>
<td>Hydraulic fill</td>
<td>0.083</td>
<td>M</td>
</tr>
<tr>
<td>Lam (2000)</td>
<td>6</td>
<td>Reference sand</td>
<td>Fraser River sand</td>
<td>Fabricated</td>
<td>0.02</td>
<td>Lab</td>
</tr>
<tr>
<td>Mimura (2003); Mimura and Suzaki (2001)</td>
<td>7</td>
<td>Yodo River, Japan</td>
<td>Yodo River</td>
<td>Fluvial</td>
<td>0.5k&lt;sup&gt;c&lt;/sup&gt;-3k</td>
<td>ISF</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Natori River, Japan</td>
<td>Natori River</td>
<td>Fluvial</td>
<td>0.5k&lt;sup&gt;c&lt;/sup&gt;-10k&lt;sup&gt;d&lt;/sup&gt;</td>
<td>ISF</td>
</tr>
<tr>
<td>Ghionna and Porcino (2003)</td>
<td>9</td>
<td>Gioia Tauro, Italy</td>
<td>Gioia Tauro</td>
<td>Marine</td>
<td>200&lt;sup&gt;e&lt;/sup&gt;</td>
<td>ISF</td>
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<td>This study</td>
<td>10</td>
<td>Charleston, South Carolina</td>
<td>Coastal Research and Education Center (CREC)</td>
<td>Beach to barrier island</td>
<td>100k</td>
<td>ISF</td>
</tr>
</tbody>
</table>

Notes:
<sup>a</sup>Lab = Laboratory fabricated, FP = Fixed piston sample, ISF = In situ ground freezing and core sampling, M = Mazier tube sample.
<sup>b</sup>Time since last liquefaction event (Claque et al. 1997).
<sup>c</sup>Estimated lower bound age considering depth of sampling and no record of recent deposition.
<sup>d</sup>Upper bound age for Holocene deposit.
<sup>e</sup>Time since last liquefaction event (Facciorusso and Vannuchi 2003).
Table 4.2: Test details, physical properties, and peak effective friction angle results of intact/remolded pairs for clean, quartzofeldspathic sand cases.

<table>
<thead>
<tr>
<th>Case #</th>
<th>Test type(^a)</th>
<th>Remolded sample preparation(^b)</th>
<th>(t) (years)</th>
<th>(F_C) (%)</th>
<th>(D_{50}) (mm)</th>
<th>(D_R) (%)</th>
<th>(\sigma_{\text{mo}}^1) (kPa)</th>
<th>(\sigma_{3c}^1) (kPa)</th>
<th>(\sigma_{mf}^1) (kPa)</th>
<th>Intact (\phi'_i) (deg)</th>
<th>Remolded (\phi'_e) (deg)</th>
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<td>51</td>
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<td>68(^c)</td>
<td>51(^c)</td>
<td>238(^c)</td>
<td>37.8(^d)</td>
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<td>74</td>
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<td>70(^c)</td>
<td>337(^c)</td>
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<td>40.0(^e)</td>
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<td>52</td>
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<td>118</td>
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<tr>
<td></td>
<td></td>
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<td>AP</td>
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Table 4.2: (continued) Test details, physical properties, and peak effective friction angle results of intact/remolded pairs for clean, quartzofeldspathic sand cases.

<table>
<thead>
<tr>
<th>Case #</th>
<th>Test type&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Remolded sample preparation&lt;sup&gt;b&lt;/sup&gt;</th>
<th>t (years)</th>
<th>$F_C$ (%)</th>
<th>$D_{50}$ (mm)</th>
<th>$D_R$ (%)</th>
<th>$\sigma'_{w0}$ (kPa)</th>
<th>$\sigma'_{sc}$ (kPa)</th>
<th>$\sigma'_{mf}$ (kPa)</th>
<th>Intact $\phi'_i$ (deg)</th>
<th>Remolded $\phi'_k$ (deg)</th>
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<tbody>
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<td>41.7&lt;sup&gt;f&lt;/sup&gt;</td>
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<tr>
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<tr>
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<td></td>
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<td>260</td>
<td>36.2</td>
<td>32.9&lt;sup&gt;f&lt;/sup&gt;</td>
<td></td>
</tr>
</tbody>
</table>

Notes:

<sup>a</sup> CD = Isotropically consolidated drained test, CU = Isotropically consolidated undrained test, CAD = Anisotropically consolidated drained test, CAU = Anisotropically consolidated undrained test.

<sup>b</sup> NA = Not available, MT = Moist tamping, AP = Air pluviation, WP = Water pluviated DV = Dry vibration.

<sup>c</sup> Average results based on two cases with the same $D_R$ and $\sigma'_{mf}$.

<sup>d</sup> Adjusted to correspond to $\sigma'_{mf}$ of intact specimen.

<sup>e</sup> Values reported are secant friction angles, which differ from the tangent friction angle values determined by Mimura (2003).

<sup>f</sup> Adjusted to correspond to $D_R$ of intact specimen as discussed in Chapter 3.
( > 8 m) with no indication of recent deposition. The upper bound age of the Natori River deposit was assumed to be 10,000 years, which marks the end of the Holocene.

The logarithmic average or geometric mean of the lower and upper ages for sites with a range of estimated ages was used for analysis of the data.

4.2.2 Sample preparation method

Sample preparation methods affect the monotonic stress-strain response of sands (Oda 1972; Mitchell and Soga 2005). As discussed in Chapter 3, monotonic triaxial compression results indicate that specimen preparation methods generating a preferred orientation towards the horizontal (e.g., water pluviation, or vibration) produce stiffer and more dilatant specimens than specimens prepared with less preferred grain orientation (e.g., moist tamping or air pluviation). The results also suggested that at higher strains, the peak strengths of specimens with less preferred grain orientation reach about 95% of the peak strength of specimens with more preferred orientation towards the horizontal plane. (Oda 1972, Wanatowski and Chu 2008).

For 8 of the 10 laboratory case studies in the dataset, the sample preparation method was reported as moist tamping (MT) or air pluviation (AP). For one case pluviation through water (WP) was used. The sample preparation method used by Daramola (1980) was not reported. Because the majority of remolded specimens were fabricated with the AP or MT methods, $\phi'_{T} - \phi'_{R}$ could be slightly overestimated compared to naturally deposited sands.
4.2.3 Peak friction angle evaluation

Table 4.2 reports the secant peak friction angles for each intact and remolded specimen (denoted $\phi'_I$ and $\phi'_R$, respectively). When not explicitly stated by the author, the $\phi'_I$ and $\phi'_R$ values were computed (Cases 1, 3, 4, 5, 6, 9). In the case of drained tests, the secant peak friction angle was computed from the initial minor effective stress, and the maximum principal stress difference using the following equation,

$$
\sin \phi' = \frac{(\sigma'_I/\sigma'_3 - 1)}{(\sigma'_I/\sigma'_3 + 1)}
$$

(4.2)

where $\sigma'_1$ and $\sigma'_3$ are the major and minor principal effective stresses at peak strength.

For undrained tests the secant peak friction angle was computed from the slope of $q-p'$ plots at the point of stress path tangency (i.e., peak $m$) using the following equation:

$$
\sin \phi' = \frac{3m}{6+m}
$$

(4.3)

where $m = q/p'$; and $q = (\sigma'_1 - \sigma'_3)/2$, and $p'_{mf} = (\sigma'_1 + 2\sigma'_3)/3$ correspond to the point of stress path tangency.

4.3 Peak friction angle comparison

Fig. 4.1 presents a comparison of peak effective friction angle for the 30 intact and remolded specimen pairs comprising Table 4.2. The mean $\phi'_I$ and $\phi'_R$ values are 39.7° and 37°, respectively. The mean and standard deviation of the compiled $\phi'_I - \phi'_R$ values are 2.7° and 2.2°, respectively, and the 95% confidence interval is 1.9-3.5°
Figure 4.1: Comparison of $\phi'_r$ and $\phi'_1$ of clean, predominantly quartzofeldspathic sands.
(treating $\phi'_I - \phi'_R$ as dependent paired samples and assuming equal variances). Twenty-five of the 30 $\phi'_I - \phi'_R$ values are greater than zero. Thus the differences in peak friction angle are statistically significant.

4.4 Simple linear models

From the information in Tables 4.1 and 4.2 the influences of $\sigma'_{mf}$ and $t$ on $\phi'_I - \phi'_R$ were investigated in Fig. 4.2. Fig. 4.2(a) presents the relationship between $\sigma'_{mf}$ and $\phi'_I - \phi'_R$, and Fig. 4.2(b) presents the relationship between $t$ and $\phi'_I - \phi'_R$. Both figures are presented as semi-log plots.

A trend of decreasing $\phi'_I - \phi'_R$ with $\sigma'_{mf}$ is observed in Fig. 4.2(a). A fit of the data with Eq. 4.1 suggests the following age-dilatancy relationship:

$$\phi'_I - \phi'_R = 1.8 \ln \left( \frac{740}{\sigma'_{mf}} \right)$$

Eq 4.2 is a good fit with the experimental data (coefficient of determination $r^2 = 0.50$, root mean square error RMSE = 1.6°) and implies a 4° decrease in $\phi'_I - \phi'_R$ for every ten-fold change in $\sigma'_{mf}$. The fitted value $C_D = 1.8^\circ$ is lower than the value obtained for CREC sand in Chapter 3 ($C_D = 1.95^\circ$) because the majority of the data in Table 4.2 are from younger deposits. The $\sigma'_R$ value of 740 kPa is also less than the value obtained in Chapter 3 ($\sigma'_R = 1,500$ kPa).

A trend of increasing $\phi'_I - \phi'_R$ with $t$ is observed in Fig. 4.2(b). A linear fit of the data suggests a second relationship for the term $\phi'_I - \phi'_R$. 

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\[ \phi'_I - \phi'_R = 0.36 \ln t + 0.23 \] (4.5)

Eq. 4.3 is a good fit with the experimental data (\( r^2 = 0.63 \), RMSE = 1.3\(^\circ\)) and strong evidence of an increase in \( \phi'_I - \phi'_R \) with age. The slope and intercept terms of Eq. 4.3 imply an increase in \( \phi'_I - \phi'_R \) of 0.8\(^\circ\) with a ten-fold increase in age and a reference age (age when \( \phi'_I - \phi'_R = 0 \)) of 0.5 years (6 months).

The relationships between \( \phi'_I - \phi'_R \) and (c) \( \sigma'_{m0} \), (d) \( \sigma'_{mc}/\sigma'_{m0} \), (e) \( D_{50} \), and (f) \( D_R \) are considered in Figs. 4.2(c-f), respectively. No strong trends between \( \phi'_I - \phi'_R \) and these variables are observed, except with \( \sigma'_{m0} \) which is related to \( \sigma'_{mf} \). The variables, listed in order of decreasing statistical significance (based on Fisher’s F-test statistic, \( F_{obs} \)), are \( \sigma'_{m0} \), \( D_{50} \), \( \sigma'_{mc}/\sigma'_{m0} \), and \( D_R \). The specimens were grouped by age into Pleistocene, Holocene, and Lab/Recent groups and plotted a second time (not shown). Each group was fitted separately to determine whether \( \sigma'_{m0} \), \( D_{50} \), \( \sigma'_{mc}/\sigma'_{m0} \), and \( D_R \) accounted for some of the variability within each age group, however no consistent trends were observed.
Figure 4.2: Relationships between $\phi'_I - \phi'_R$ and (a) $\sigma'_{mf}$, (b) $t$, (c) $\sigma'_{m0}$, (d) $\sigma'_m / \sigma'_{m0}$, (e) $D_{50}$, and (f) $D_R$. 
Figure 4.2: (continued) Relationships between $\phi' - \phi''_R$ and (a) $\sigma'_{mf}$, (b) $t$, (c) $\sigma'_{m0}$, (d) $\sigma'_{mc}/\sigma'_{m0}$, (e) $D_{50}$, and (f) $D_R$. 
4.5 General age-dilatancy model

The data points of Table 4.2 were grouped by age to further investigate the relationship between $\phi'_I - \phi'_R$, $\sigma'_mf$, and $t$. Presented in Fig. 4.3(a) are values of $\phi'_I - \phi'_R$ grouped by age as Lab/Recent, Holocene, and Pleistocene. The Lab/Recent grouping comprises nine data points from Cases 1, 3, 5, 6, and 9, with ages ranging from 0.02-200 years. The Holocene grouping comprises nine data points from Cases 2, 4, 7, and 8 with ages ranging from 1,200-2,450 years. The Pleistocene grouping comprises 11 data points from case 10 with an age of 100,000 years. The Lab/Recent, Holocene, and Pleistocene groups have mean $\phi'_I - \phi'_R$ values of 0.2, 3.0, and 4.8, respectively.

Each age group seen in Fig. 4.3(a) is regressed separately with Eq. 4.1 and fitted with values of $C_D$ and $\sigma'_R$. When the data are fitted in this manner, the following observations are made: (1) a decrease in $\phi'_I - \phi'_R$ with increasing $\sigma'_mf$ for each age group; (2) an increase in $C_D$ with increasing deposit age; (3) similar values of $\sigma'_R$ that range from 1,000-7,000 kPa; and (4) a considerable decrease in model error versus Eq. 4.4.

The $\sigma'_R$ values of the Lab/Recent, Holocene, and Pleistocene groups are 1,100, 6,100, and 5,800 kPa, respectively. A simpler relationship utilizing a common reference stress of 6,000 kPa is used in Fig. 4.3(b). It is seen that the relationship fits the three age groups well without any noticeable change in $R^2$.

The relationships presented in Fig. 4.3(b), suggest an age-dilatancy model in which $C_D$ varies with $t$. That is,
Figure 4.3: Comparison of two models for $\phi'_i - \phi'_R$ with (a) different $\sigma'_R$ values and (b) a common $\sigma'_R$ value.
\[
\phi'_t - \phi'_R = C_D \ln \left( \frac{\sigma'_R}{\sigma'_{mf}} \right) = c_D \ln \left( \frac{t}{t_R} \right) \ln \left( \frac{\sigma'_R}{\sigma'_{mf}} \right) \quad (4.6)
\]

where \( C_D = c_D \ln (t/t_R) \), \( c_D \) is an age dilatancy constant, \( t_R \) is a reference age in years, and \( \sigma'_R \) (kPa) is a reference confining stress in the same units as \( \sigma'_{mf} \).

Nonlinear regression of the dataset predicts mean values of \( c_D = 0.13 \), \( t_R = 0.88 \) years, and \( \sigma'_R = 2,500 \) kPa. Table 4.3 summarizes additional outputs of the regression including standard error (SE) 95% confidence intervals (CI) of \( c_D \), \( \log t_R \) and \( \log \sigma'_R \).

The model has a strong fit with the observed data with a \( R^2 \) value of 0.76 and RMSE of 1.1°.

4.5.1 Residuals

Figs. 4.4(a-f) plot the residuals (\( \varepsilon \)) of Eq. 4.6 with respect to (a) \( \sigma'_{mf} \), (b) \( t \), (c) \( \sigma'_{m0} \), (d) \( \sigma'_{mc}/\sigma'_{m0} \), (e) \( D_{50} \), and (f) \( D_R \). The best fit relationships between \( \varepsilon \) and each variable are also shown in these plots.

It is seen in Figs. 4.4(a) and 4.4(b) that the residuals are more or less randomly scattered about zero and depict little variation with \( t \) or \( \sigma'_{mf} \), suggesting that the logarithmic terms in Eq. 4.6 are suitable for the model. No obvious trends are seen in of the remaining plots (Figs. 4.4c-f), therefore it in unlikely that the variables \( \sigma'_{m0} \), \( \sigma'_{mc}/\sigma'_{m0} \), \( D_{50} \), \( D_R \) are predictors of \( \phi'_t - \phi'_R \). This finding is significant with respect to \( D_R \), because it is further evidence that dilatancy due to density is the same for intact and remolded specimens, as suggested in Chapter 3.
Table 4.3: Nonlinear regression of dataset based on Eq. 4.6.

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<td>$\phi'_I - \phi'_R = c_D \ln \left( \frac{t}{t_R} \right) \ln \left( \frac{\sigma'_R}{\sigma'_mf} \right)$</td>
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<td>1.1</td>
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<td>0.13</td>
<td>0.039</td>
<td>0.047</td>
<td>0.206</td>
</tr>
<tr>
<td>$\ln \left[ t_R \text{ (years)} \right]$</td>
<td>-0.13</td>
<td>1.5</td>
<td>-3.3</td>
<td>3.0</td>
</tr>
<tr>
<td>$\ln \left[ \sigma'_R \text{ (kPa)} \right]$</td>
<td>7.8</td>
<td>0.81</td>
<td>6.1</td>
<td>9.5</td>
</tr>
</tbody>
</table>
Figure 4.4: Residuals of age dilatancy model (Eq. 4.6) plotted against (a) $\sigma_{mf}'$, (b) $t$, (c) $\sigma_{m0}'$, (d) $\sigma_{mc}'/\sigma_{m0}'$, (e) $D_{50}$, and (f) $D_R$. 

\[ \varepsilon = -0.016 \ln t + 0.077 \\
\varepsilon = 0.14 \ln \sigma_{mf}' - 0.75 \\
\varepsilon = 0.23 \ln \sigma_{m0}' - 0.99 \\
\varepsilon = 0.053 \ln \sigma_{mc}'/\sigma_{m0}' - 0.13 \]
Figure 4.4: (continued) Residuals of age dilatancy model (Eq. 4.6) plotted against (a) $\sigma'_m$, (b) $t$, (c) $\sigma'_m$, (d) $\sigma'_m / \sigma'_0$, (e) $D_{50}$, and (f) $D_R$. 
4.5.2 Reanalysis with refined data set

Figs. 4.2-4.4 indicate that the variation in $\phi'_{I} - \phi'_{R}$ values is well explained by $t$ and $\sigma'_{mf}$, while other factors considered in the dataset have little influence on $\phi'_{I} - \phi'_{R}$. For this reason, the dataset was refined to reduce the influence of repetitious data. The refinement was done by averaging results from cases with the same $t$ and $\sigma'_{mf}$ values (i.e., cases with a percent difference in $\sigma'_{mf}$ of less than 10%). This reduces the number of aged-freshly deposited pairs from the Massey site, Yodo site, and CREC site (i.e., cases 3, 7, and 10) from 3, 3, and 11, to 2, 2, and 4, respectively, and the total number of points from 30 to 21. After refinements each point within the dataset represents a unique combination of $t$ and $\sigma'_{mf}$.

The $\phi'_{I} - \phi'_{R}$-$t$ and $\phi'_{I} - \phi'_{R}$-$\sigma'_{mf}$ relationships after refining the dataset are seen in Figs. 4.5(a) and 4.5(b), respectively. The estimated coefficients and model errors indicated by the regression lines are similar to the values in Eqs. 4.4 and 4.5 determined with all the data points, which demonstrates that the reduced dataset does not significantly alter the model.

Nonlinear regression of the refined dataset predicts mean values of $c_{D} = 0.13$, $t_{R} = 0.6$ years, and $\sigma'_{mf} = 2,000$ kPa. The predicted $t_{R}$ value of 0.6 years is similar to the $t_{R}$ value of 0.88 years predicted from regression of the full dataset. On the other hand, the $\phi'_{R}$ values correspond to specimens that were saturated for a period of time ranging from several hours to several days depending on the method used and consolidated for
less than 1 day. The higher $t_R$ values of 0.6 years and 0.88 years likely reflect uncertainty in the dataset as all of the predicted $\phi'_p - \phi'_R$ values are underpredicted for $t < t_R$ in Fig. 4.5(b). Therefore, a fixed reference age of $(t_R)$ of about 3-4 days (approximately 0.01 years) is more appropriate.

Nonlinear regression of the refined dataset with an assumed $t_R$ value of 0.01 years predicts mean $c_p$ and $\sigma'_R$ values of 0.095 and 1,600 kPa with an average estimating error of 1.0°. Table 4.4 summarizes additional outputs of the regression including the SE and the 95% CI of $c_p$ and $\log \sigma'_R$. This model based on the refined dataset is recommended versus the model based on the full dataset, because it assumes a realistic $t_R$ value and the case studies are more evenly represented in the fitting.

4.5.3 Discussion

Figs. 4.6(a) and 4.6(b) present $(\phi'_1 - \phi'_R) - t$ and $(\phi'_1 - \phi'_R) - \sigma'_{mf}$ relationships based on the recommended model given in Table 4.4. Predicted $\phi'_1 - \phi'_R$ values are plotted for $\sigma'_{mf}$ values of 100, 300, and 1000 kPa and the points comprising the dataset are sorted into $\sigma'_{mf} < 100$ kPa, $100 < \sigma'_{mf} < 300$ kPa, and $\sigma'_{mf} > 300$ kPa groups in Fig. 4.6(a). Similarly, predicted $\phi'_1 - \phi'_R$ values are plotted for ages of 1, 1,000, and 1,000,000 years, with the dataset sorted into lab/recent, Holocene, and Pleistocene groups.
Figure 4.5: Relationships between $\phi'_I - \phi'_R$ and (a) $\sigma'_{mf}$, and (b) $t$ after refining the dataset.
Table 4.4: Nonlinear regression of refined dataset based on Eq. 4.6 and assuming \( t_R = 10^2 \) years.

<table>
<thead>
<tr>
<th>Regression equation</th>
<th>( n )</th>
<th>( R^2 )</th>
<th>R.M.S.E.</th>
<th>F-observed</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi'_I - \phi'_R = c_D \ln \left( \frac{t}{t_R} \right) \ln \left( \frac{\sigma'_R}{\sigma'_mf} \right) )</td>
<td>21</td>
<td>0.76</td>
<td>1.0</td>
<td>59</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Regression terms</th>
<th>Mean</th>
<th>S.E.</th>
<th>95% C.I. Lower bound</th>
<th>95% C.I. Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c_D )</td>
<td>0.095</td>
<td>0.025</td>
<td>0.043</td>
<td>0.15</td>
</tr>
<tr>
<td>( \ln \left( t_R \right) ) years</td>
<td>-4.6</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>( \ln \left( \sigma'_R \right) ) kPa</td>
<td>7.4</td>
<td>0.67</td>
<td>5.9</td>
<td>8.8</td>
</tr>
</tbody>
</table>

in Fig. 4.6(b). In Figs. 4.6(a) and 4.6(b), \( \phi'_I - \phi'_R = 0 \) for \( t < t_R \) (i.e., 0.01 years) and \( \sigma'_mf > \sigma'_R \) (i.e., 1,600 kPa), respectively.

The relationships in Fig. 4.6(a) predict increases in \( \phi'_I - \phi'_R \) of 0.09°, 0.37°, and 0.60° for \( \sigma'_mf \) values of 100, 300, and 1,000 kPa with every ten fold change in age, respectively, implying a 0.5° decrease in the slope of Eq. 4.6 for each 10 fold change in \( \sigma'_mf \). This reduction does not imply that aging processes occur more slowly under higher confining pressures. Rather it implies that the dilatancy exhibited by aged specimens depends on the magnitude of confining pressure applied before shearing.

The slopes of the relationships in Fig. 4.6(b) for \( t \) values of 1, 1,000, and 1,000,000 years represent the term \( C_D \) (i.e. \( C_D = c_D \ln \left( t/t_R \right) = 0.095 \ln t/0.01 \)). Thus, Eq. 4.6 predicts \( C_D \) values of 0.44°, 1.09°, and 1.75° for ages of 1, 1,000, and 1,000,000
years respectively, implying a 0.22° increase in $C_D$ for each 10 fold change in $t$. The curves in Fig. 4.6(b) are similar in appearance to the curves representing the variation between $\phi_{\rho}$, $D_R$, and $\sigma_m$ proposed by Bolton (1986), except that the functions in Fig. 4.6(b) intersect the $\sigma_m$ axis at a common value of $\sigma_R$ whereas, the functions proposed by Bolton (1986) intersect the $\sigma_m$ axis at different values of $\sigma_m$.

The behavior of sands in isotropic compression serves as a partial validation of the value estimated for $\sigma_R$. Results of isotropic compression of predominantly quartzofeldspathic sands (Lee and Seed 1967, Ishihara 1993, Coop and Lee 1993) indicate that gradual yielding occurs at between pressures of 1,000-10,000 kPa due to particle breagage at highly stressed interparticle contacts. Thus, a mean value of $\sigma_R = 1,600$ kPa and 95% confidence interval in $\sigma_R$ of 400-7,000 kPa provide are reasonable limits to the region in which the influence of diagenesis is completely erased for natural quartzofeldspathic sands.

### 4.6 General MEVR-dilatancy model

A limitation of Eq 4.6 is that it is often difficult in practice to estimate $t$ of natural deposits, and aging processes or diagenesis can differ from location to location even in the same deposit. As a result, it is desirable to have an alternative parameter to use in place of age. As summarized in Chapters 1 and 2, Andrus et al. (2009) proposed MEVR to approximate the age of granular soils, and Hayati and Andrus (2009) successfully implemented MEVR as a proxy variable for age in a study to predict liquefaction.
Figure 4.6: Predicted relationships between $\phi'_K - \phi'_R$ and (a) $t$ and (b) $\sigma'_{mf}$ based on refined dataset and Eq 4.3.
resistance of aged sands. The MEVR-time relationship discussed in Chapter 2 is expressed with the equation:

\[ \text{MEVR} = 0.082 \log t + 0.935 \]  \hspace{1cm} (4.7)

Although criteria used to establish Eq 4.7 and the criteria used to compile the dataset in this study differ, one approach to relate \( \phi'_I - \phi'_R \) with MEVR is by substituting Eq. 4.7 into 4.6 with the recommended values of \( c_D = 0.095 \), \( t_R = 0.01 \) years, and \( \sigma'_R = 1,600 \) kPa. This produces the following equation for \( \phi'_I - \phi'_R \) in terms of MEVR,

\[ \phi'_I - \phi'_R = 2.7 \left( \text{MEVR} - 0.77 \right) \ln \left( 1,600 / \sigma'_{mf} \right) \]  \hspace{1cm} (4.8)

### 4.6.1 MEVR values established for dataset

Presented in Table 4.5 are the measured overburden stress and fines content corrected shear wave velocity (\( V_{S_{I,cs}} \)), estimated \( V_{S_{I,cs}} \) (based on the predictive equations of Andrus et al. 2004), and MEVR values for the cases used in this study. A single MEVR rather than multiple MEVRs is assigned for each case (excluding Case 1) to be consistent with the assignment of a single age for each case. Thus, the variation of MEVR with depth is not considered.

For Cases 2, 7, 8, 9, and 10, profiles of shear wave velocity, measured by either the downhole or crosshole test method, and cone tip resistance were reported. Average values of MEVR were established for these sites within a depth range that included the retrieval depths of all intact core samples used for laboratory testing, and at least three shear wave velocity measurements. For Cases 3 and 4 the average normalized shear wave velocity \( V_{S_{I}} \) and average cone tip resistance (\( q_c \)) reported by Robertson et al. (2000)
within the depth range of the deposit (8-13 m) were used to calculate measured and estimated $V_{S1,cs}$, respectively. Eq. 4.7 was used to estimate MEVR for the two case studies concerning lab-fabricated specimens aged in a triaxial chamber and for the recent sand fills deposited at the West Kowloon site (Cases 1, 5, and 6) because no shear wave velocity measurements were available.

The values of $t$ and MEVR determined for the cases in this study and the mean curve expressed by Eq. 4.7 are plotted in Fig. 4.7. A trend of increasing MEVR with $t$ and good general agreement with Eq. 4.7 is observed, however most of the MEVR values reported in this study are overestimated. This overestimation will be investigated in Chapter 6.

<table>
<thead>
<tr>
<th>Case</th>
<th>$t$ (years)</th>
<th>Measured $V_{S1,cs}$ (m/s)</th>
<th>Estimated $V_{S1,cs}$ (m/s)</th>
<th>MEVR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>0.027</td>
<td>n/a</td>
<td>n/a</td>
<td>0.81*</td>
</tr>
<tr>
<td>1B</td>
<td>0.083</td>
<td>n/a</td>
<td>n/a</td>
<td>0.85*</td>
</tr>
<tr>
<td>1C</td>
<td>0.42</td>
<td>n/a</td>
<td>n/a</td>
<td>0.90*</td>
</tr>
<tr>
<td>2</td>
<td>2,450</td>
<td>160</td>
<td>140</td>
<td>1.14</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>168</td>
<td>157</td>
<td>1.07</td>
</tr>
<tr>
<td>4</td>
<td>1,700</td>
<td>177</td>
<td>168</td>
<td>1.06</td>
</tr>
<tr>
<td>5</td>
<td>0.083</td>
<td>n/a</td>
<td>n/a</td>
<td>0.85*</td>
</tr>
<tr>
<td>6</td>
<td>0.019</td>
<td>n/a</td>
<td>n/a</td>
<td>0.79*</td>
</tr>
<tr>
<td>7</td>
<td>1,220</td>
<td>202</td>
<td>190</td>
<td>1.06</td>
</tr>
<tr>
<td>8</td>
<td>2,240</td>
<td>218</td>
<td>220</td>
<td>0.99</td>
</tr>
<tr>
<td>9</td>
<td>200</td>
<td>210</td>
<td>236</td>
<td>0.89</td>
</tr>
<tr>
<td>10</td>
<td>100,000</td>
<td>237</td>
<td>170</td>
<td>1.39</td>
</tr>
</tbody>
</table>

*MEVR estimated from Eq. 4.7
Figure 4.7: MEVR versus age for cases in dataset.

Andrus et al. (2009),

$$MEVR = 0.082 \log t + 0.935$$
Using Eq. 4.8 and the MEVR values in Table 4.5, a good fit with the refined dataset is obtained ($R^2 = 0.76$, RMSE = 1.0$^\circ$). The direct derivation of an MEVR dilatancy expression is described in the following subsection.

### 4.6.2 Results

The relationship between $\phi' - \phi'_R$ and MEVR based on the refined dataset described in section 4.5.2 is presented in Fig. 4.8. The values of $\phi' - \phi'_R$ exhibit a strong trend with MEVR and a best fit linear relationship given by:

$$\phi' - \phi'_R = 8.8\text{MEVR} - 7.3$$

Eq. 4.9 suggests an 0.88$^\circ$ increase in $\phi' - \phi'_R$ with each 10% increase in MEVR and a reference MEVR value (i.e., MEVR when $\phi' - \phi'_R = 0$) of 0.83.

An expression with the same functional form as Eq. 4.6 was used to estimate a MEVR-dilatancy relationship. That is,

$$\phi' - \phi'_R = c_{D,M} (\text{MEVR} - \text{MEVR}_R) \ln(\sigma'_R / \sigma'_{mf})$$

where $c_{D,M}$ is an MEVR dilatancy constant, and MEVR$_R$ is a reference value.

Nonlinear regression of the refined dataset predicts mean values of $c_{D,M} = 2.3$, MEVR$_R = 0.76$, and $\sigma'_R = 3,000$ kPa. Table 4.6 summarizes additional outputs of the regression including 95% confidence intervals of the estimated model parameters. The model is a good fit with the observed data ($R^2 = 0.77$, RMSE = 1.0$^\circ$), achieving the same degree of accuracy as Eqs. 4.6 and 4.8.
Figure 4.8: Relationship between $\phi'_I - \phi'_R$ and MEVR based on refined dataset.
Table 4.6: Nonlinear regression of refined dataset based on Eq. 4.7.

<table>
<thead>
<tr>
<th>Regression equation</th>
<th>$n$</th>
<th>$R^2$</th>
<th>RMSE</th>
<th>F-observed</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi'_I - \phi'<em>R = c</em>{D,M} (MEVR - MEVR_R) \ln \left( \sigma'<em>R / \sigma'</em>{mf} \right)$</td>
<td>21</td>
<td>0.77</td>
<td>1.1</td>
<td>30</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Regression terms</th>
<th>Mean</th>
<th>S.E.</th>
<th>95% C.I. Lower bound</th>
<th>95% C.I. Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{D,M}$</td>
<td>2.3</td>
<td>0.80</td>
<td>0.64</td>
<td>4.0</td>
</tr>
<tr>
<td>MEVR$_R$</td>
<td>0.76</td>
<td>0.084</td>
<td>0.59</td>
<td>0.94</td>
</tr>
<tr>
<td>$\ln[\sigma'_R \text{ (kPa)}]$</td>
<td>8.0</td>
<td>1.2</td>
<td>5.5</td>
<td>10.5</td>
</tr>
</tbody>
</table>

4.6.3 Discussion

The age-dilatancy model based on Eq. 4.6 and the MEVR-dilatancy model based on Eq. 4.10 predict similar $\sigma'_R$ values of 1,600 kPa and 3,000 kPa, respectively, supporting a range between 1,000-10,000 kPa for which age effects on $\phi'_I - \phi'_R$ are suppressed. The reference MEVR value of 0.76 corresponds roughly to a $t_R$ value 3 days ($\approx 0.01$ years) based on Eq. 4.7. Therefore the MEVR$_R$ value corresponds well with the reference age assumed in Eq. 4.6.

Predicted relationships for $\phi'_I - \phi'_R$ based on Eq. 4.10 and MEVR values of 0.9, 1.2, and 1.5 are compared with relationships for $\phi'_I - \phi'_R$ based on Eq. 4.7 and $t$ values of 1, 1,000, and 1,000,000 years in Fig. 4.9. MEVR values of 0.9, 1.2, and 1.5 roughly correspond to $t$ values of about 0.4, 1,700, and 8,000,000 years, respectively based on Eq. 4.8.
The relationships in Fig. 4.9 based on Eq. 4.10 predict $C_D$ values of 1.70°, 1.01°, and 0.32° for MEVR values of 0.9, 1.2, and 1.5, respectively, implying a 0.18° decrease in slope for each 0.08 change in MEVR. As observed previously, the relationships in Fig. 4.6 based on Eq. 4.6 imply a 0.22° decrease in slope for each 10 fold change in age. It is expected from Andrus et al. (2009) that a ten-fold change in age should correspond to a change of about 0.08 in MEVR, therefore the comparison indicates that $\phi'_i - \phi'_R$ changes slightly less with MEVR than Eq. 4.8 would imply.

4.7 Uses of proposed models

Eqs. 4.6 and 4.10 can be used to estimate $\phi'_i$ from known values of $\phi'_R$ under effective confining pressures of interest for design. Eqs. 4.6 and 4.10 may also be useful for predicting loss of peak strength during a disturbance or under large surcharges provided reliable in situ $\phi'_p$ values are available.

The equations also revise the model proposed in Chapter 3 (Eq. 3.4), by replacing the term $C_DZ$ with age dilatancy or MEVR dilatancy terms. That is,

\[
\phi'_p - \phi'_{crit} = \left[ \frac{a D_R}{100} + c_D \ln \left( \frac{t}{t_R} \right) \right] \ln \left( \frac{\sigma'_R}{\sigma'_mf} \right) + b \tag{4.11a}
\]

and

\[
\phi'_p - \phi'_{crit} = \left[ \frac{a D_R}{100} + c_{D,M} \left( MEVR - MEVR_R \right) \right] \ln \left( \frac{\sigma'_R}{\sigma'_mf} \right) + b \tag{4.11b}
\]

Thus, Eqs. 4.11(a) and 4.11(b) are two models that can be used to estimate the peak strength of remolded and natural, uncedmented predominantly quartzofeldspathic sands.
Figure 4.9: Comparison of age and MEVR dilatancy models.
The applicability of such a model for CREC sand has already been considered in Chapter 3, and shown to be a considerable improvement over a strength and dilatancy model with no age term. However a larger dataset including available triaxial test results of both remolded sands and intact sands is needed to verify the parameters of Eqs. 4.11(a) and 4.11(b), and verify several assumptions of the model. Issues that will need to be resolved include: (1) suitability of $a$, and $c_D$ or $c_{D,M}$ as constants or as intrinsic properties of a given soil influenced by particle shape, gradation, etc.; (2) suitability of $\sigma'_R$ as an intrinsic property of a given soil influenced by particle shape, gradation, etc., as this study proposes a single $\sigma'_R$ value that is suitable for different compositions; (3) the appropriateness of a common $\sigma'_R$ term for both $aD_R$ and $C_D$; (4) the appropriateness of a single $\sigma'_R$ value for which $\phi'_I - \phi'_R = 0$ for any value of $t$ (as opposed to the model proposed by Bolton (1986) for which $\phi'_p - \phi'_{crit} = 0$ for different values of $D_R$) and (5) further investigation on the relative influences of aging processes and soil fabric.

Eqs. 4.11(a) and 4.11(b) are intended for clean sands. Additional triaxial compression test results for intact and remolded sands are needed to extend these models to sands with significant fines content (FC > 5%).

4.8 Summary

In this chapter, a dataset comprising the results of triaxial compression tests on intact natural predominantly quartzofeldspathic sand specimens and corresponding remolded specimens was established from ten cases to estimate dilatancy caused by aging
processes, assumed to be quantified as the difference between intact and remolded peak friction angle. Strong dependencies of age dilatancy on confining stress and time since deposition were observed in the dataset.

The age dilatancy equation proposed in Chapter 3 was revised with a term that accounts for time since deposition (or time since a recent disturbance) in addition to confining pressure. An alternative MEVR dilatancy equation was also proposed because it is often difficult to estimate time since deposition or the last disturbance. Both equations are good fits with the compiled dataset.

Models for estimating the peak friction angles of natural and remolded quartzofeldspathic sands as functions of density, age, and confining pressure are implied by the results of this study. However, a larger dataset compiling results of triaxial compression test results on intact and remolded quartzofeldspathic sand is needed to validate the model parameters.
5.1 Introduction

The age and MEVR dilatancy models (Chapter 4) used to predict the peak secant friction angle ($\phi'_p$) of uncemented natural quartzofeldspathic sands imply corresponding relationships between peak secant shear strength ($\tau_p$) and age or MEVR. The implied relationship and a simplified expression to estimate the increase in $\tau_p$ of uncemented natural quartzofeldspathic sands due to diagenesis is presented in this chapter. Herein, a term called peak strength ratio (PSR) will be used to represent the ratio of intact ($\tau_p)_i$ to remolded ($\tau_p)_R$ peak shear (i.e. $\text{PSR} = (\tau_p)_i / (\tau_p)_R$).

Values of $\tau_p$ and normal stress ($\sigma_n$) associated with a conventional Mohr-Colomb diagram of a cohesionless soil (i.e., $c' = 0$) are determined for an individual triaxial test specimen from the major ($\sigma'_1$) and minor ($\sigma'_3$) principal stresses at failure (or peak strength), the secant peak friction angle ($\phi'_p$), and the geometry of a conventional Mohr's circle. That is,

$$\sigma_n = \cos^2 \phi'_p (\sigma'_1 + \sigma'_3) / 2$$

and

$$\tau_p = \sigma_n \tan \phi'_p$$
where \( \phi'_p = (\sigma'_1/\sigma'_3 - 1)/(\sigma'_1/\sigma'_3 + 1) \).

Plotted in Fig. 5.1(a) are the \( \tau_p \) and \( \sigma_n \) values evaluated with Eqs. 5.1 and 5.2 from triaxial compression tests conducted on medium dense intact and remolded CREC sand at effective confining stresses of 17, 35, and 69 kPa. Also plotted is a surface in \( \tau_p - \sigma_n \) corresponding to a critical state strength of 33.4\(^\circ\) and the predicted failure surfaces derived from the age dilatancy model (Eq. 4.8a) with the coefficient values of \( a = 2.41, \ c_D = 0.095, \ t_R = 0.01 \text{ years}, \ \sigma'_R = 1,600 \text{ kPa}, \ b = -1.76, \) and \( \phi'_\text{crit} = 33.4^\circ \) determined in Chapters 3 and 4 and ages of 100,000 and 0.01 years, for the intact and remolded specimens, respectively.

Both failure surfaces depicted in Fig. 5.1(a) are curved due to the suppression of dilatancy with increasing confining pressure. For intact specimens, both dilatancy due to age and due to density are suppressed, therefore the curvature of the failure surface is more pronounced. From the plotted data points in Fig. 5.1, PSR values of 1.38, 1.33, 1.28 are obtained for average \( \sigma'_n \) values of 28, 57, and 111 kPa, respectively.

Plotted in Fig. 5.1(b) are the residual shear strengths (\( \tau_{res} \)) of medium dense intact and remolded specimens measured at 16-20% axial strain. The \( \tau_{res} \) values of intact specimens are nearly the same as the \( \tau_{res} \) values of remolded specimens and degraded by about 20\% from their peak values. Therefore, intact and remolded specimens are well characterized by a single residual strength surface with \( \phi' = 35.3^\circ \), which is close to the critical state surface. For the CREC sand and the nine other cases summarized in Chapter 4, there is little evidence of improvement in residual shear strength due to diagenesis.
Figure 5.1: (a) Peak and (b) residual shear strength envelopes of medium dense CREC sand.
5.2 PSR-time and PSR-MEVG relationships

Peak shear strength values of intact \((\tau_p)_I\) and remolded \((\tau_p)_R\) specimens from the ten cases compiled in Chapter 4 are compared in Fig. 5.2. The \((\tau_p)_I\) values are about 17% greater than \((\tau_p)_R\) values on average. The predictive relationship for PSR as a function of \(\phi_p'\) is obtained:

\[
PSR = \frac{(\tau_p)_I}{(\tau_p)_R} = \frac{(\sigma'_n)_I \tan(\phi_p')_I}{(\sigma'_n)_R \tan(\phi_p')_R}
\]  

(5.3)

where \((\sigma'_n)_I\) and \((\sigma'_n)_R\) are the normal stress at failure and \((\phi_p')_I\) and \((\phi_p')_R\) are peak friction angles of intact and remolded specimens, respectively. To evaluate Eq. 5.3, an expression for \(\sigma'_n\) in terms of mean effective stress at peak strength \((\sigma'_{mf})\) is needed which can be given by,

\[
\sigma'_n = \sigma'_{mf} \frac{3\cos^2 \phi_p'}{3 - \sin \phi_p'}
\]  

(5.4)

where \(\phi_p'\) and \(\sigma'_n\) take values of \((\phi_p')_I\) and \((\sigma'_n)_I\) in the numerator and \((\phi_p')_R\) \((\sigma'_n)_R\) in the denominator of Eq. 5.3.

The resulting equations can be evaluated by substituting Eq. 4.8(a) or Eq. 4.8(b) with the values of \((\phi_p')_I\) and \((\phi_p')_R\), as functions of \(D_R\), \(\sigma'_{mf}\), \(\phi'_{crit}\), and \(t\) or MEVG, which can become fairly complicated. A simpler approximation of PSR is expressed as:

\[
PSR \approx 1 + 0.05(\phi'_I - \phi'_R)
\]  

(5.5)
where $\phi'_{I-R}$ is the intact minus remolded peak effective friction angle.

Fig. 5.3 plots the values of PSR against the values of $(\phi'_{I-R})$ for the ten cases in Chapter 4. It is seen that Eq. 5.5 is a very close approximation of PSR. Substituting Eqs. 4.8(a) with the values $c_D = 0.095$, $t_R = 0.01$ years, $\sigma'_{R} = 1,600$ kPa and substituting Eq. 4.8(b) with the values $c_{D,M} = 2.3$, $\text{MEVR}_R = 0.77$, and $\sigma'_{R} = 3,000$ kPa the following PSR-time and PSR-MEV R relationships are proposed:

$$\text{PSR} = 1 + 0.0048 \ln \left( \frac{t}{0.01} \right) \ln \left( \frac{1,600}{\sigma'_{mf}} \right)$$

(5.6a)

and

$$\text{PSR} = 1 + 0.12 (\text{MEVR} - 0.77) \ln \left( \frac{3,000}{\sigma'_{mf}} \right)$$

(5.6b)

Plotted in Fig. 5.4(a) and Fig. 5.4(b) are the relationships between $t$ and PSR and between $\sigma'_{mf}$ and PSR, respectively using the refined dataset of 21 intact/remolded datasets discussed in Chapter 4. Also plotted as dashed lines are the variations in PSR with $t$ and $\sigma'_{mf}$ predicted by Eq. 5.6(a) for $\sigma'_{mf}$ of 100, 300, and 1,000 kPa and $t$ of 1, 1,000, and 1,000,000 years, respectively. Lastly, solid lines are plotted in Fig. 5.4(a) and Fig. 5.4(b) which were fitted directly to the following equation for the 21 PSR, $t$, and $\sigma'_{mf}$ values:

$$\text{PSR} = 1 + a \ln \left( \frac{t}{0.01} \right) \ln \left( \frac{\sigma'_{R}}{\sigma'_{mf}} \right)$$

(5.7)
Figure 5.2: Comparison of $\tau_p$ and $\tau_p^R$ of uncemented quartz sands.
**Figure 5.3:** Relationship between $\phi'_I - \phi'_R$ and PSR.

The equation for PSR is given by:

$$PSR = 1 + 0.05(\phi_I' - \phi_R')$$

with $r^2 = 0.997$ and $n = 30$. The graph illustrates the linear relationship between the peak friction angle difference and the peak shear strength ratio.
where \( a \) is a constant and \( \sigma'_{h} \) is a reference confining stress in the same units as \( \sigma'_{mf} \).

The direct fitting provided \( a = 0.00479 \) and \( \sigma'_{h} = 1,450 \) kPa.

It is seen in both Fig. 5.4(a) and Fig. 5.4(b) that the approximate relationship using Eq. 5.6(a) (dashed lines) predicts PSR values that are nearly identical to a model fitted with the actual PSR values (solid lines). From the curves drawn in Fig. 5.4(a) it is observed that the change is PSR is about 3.0\%, 1.9\%, and 0.5\%, for \( \sigma'_{mf} \) values of 100, 300, and 1,000 kPa.

5.3 Comparison of PSR and \( K_{DR} \)

Fig. 5.5. compares the PSR-time relationship defined by Eq. 5.6(a) with the deposit resistance correction factor (\( K_{DR} \))-time relationship proposed by Hayati and Andrus (2009) based on cyclic triaxial test results of intact and remolded predominantly quartzofeldspathic sands. \( K_{DR} \) is the deposit resistance corrected cyclic resistance ratio (CRR)\(_K\) of an intact specimen divided by the CRR of a freshly deposited specimen defined as:

\[
K_{DR} = \frac{\text{CRR}_{K}}{\text{CRR}} = \frac{\left(\frac{\tau_{\text{cyc}}}{\sigma'_{v0}}\right)_{K}}{\left(\frac{\tau_{\text{cyc}}}{\sigma'_{v0}}\right)} \quad (5.8)
\]

where \( \frac{\tau_{\text{cyc}}}{\sigma'_{v0}} \) is the ratio of cyclic shear stress to effective overburden stress needed to cause 5\% double amplitude axial strain in 15 cycles. Thus the age-PSR and \( K_{DR} \)-time relationships quantify the increase in static shear strength and cyclic shear strength with age, respectively. The PSR-time model is plotted in Fig. 5.5 based on a \( \sigma'_{mf} \) value of 101.
Figure 5.4: Predicted relationships between PSR and (a) $t$ and (b) $\sigma_{nf}'$. 
kPa, because a majority of the specimens involved in determining $K_{DR}$ were consolidated at a confining pressure of about 101 kPa. It is seen that $K_{DR}$ increases at a rate of 12% per log cycle of time compared which four times greater than the increase in PSR per log cycle of time, suggesting that liquefaction resistance is more sensitive to diagenesis than peak static shear strength.

5.4 Summary

Age-dilatancy and MEVR-dilatancy models proposed in Chapter 4 were used in this chapter to approximate relationships for predicting the peak shear strength ratio (PSR). The PSR-time equation suggests that peak shear strength increases by about 3% for each tenfold change in silica sands age for a mean effective confining pressure at failure of 101 kPa. A comparison between the PSR-time model and the $K_{DR}$-time model proposed by Hayati and Andrus (2009) suggests that static shear strength increases much more gradually with time than the cyclic shear strength.
Figure 5.5: Comparison between the PSR-time relationship expressed by Eq. 5.7 and the $K_{DR}$-time relationships proposed by Hayati and Andrus (2009). Note: Data from Troncoso et al. (1988) are not shown in Fig. 5.5.
CHAPTER 6

INCREASE IN SMALL-STRAIN STIFFNESS WITH TIME IN SANDS FROM LABORATORY AND FIELD TESTS

6.1 Introduction

Shear wave velocity-time relationships determined in the laboratory from resonant column tests on remolded sands and determined from in situ field test measurements \((V_s, q_t, \text{ and } N_{60})\) and penetration resistance-\(V_s\) relationships were discussed in Chapter 2. From the discussion, the general form of the velocity ratio (VR) equation used to express the dependency of \(V_s\) on time can be expressed as:

\[
VR = \frac{V_{s,t}}{V_{s,R}} = 1 + N_{VS} \log{t/t_R}
\]

(6.1)

where \(V_{s,t}\) is shear wave velocity at time \(t > t_R\), \(V_{s,R}\) is shear wave velocity at the reference age of \(t_R\), \(N_{VS}\) is the rate increase of VR, and \(t\) is time since deposition or last critical disturbance.

Two problems discussed in Chapter 2 associated with the applicability of Eq. 6.1 are investigated in this chapter, which are: (1) establishing a reasonable \(N_{VS}\) value for natural sands based on the VR values interpreted from high quality intact specimens; and (2) resolving the difference in \(N_{VS}\) based on laboratory tests and based on field tests.

First a database of \(N_{VS}\) values is established from the results of laboratory tests performed on remolded clean sands. Then additional results of laboratory tests performed on high quality intact and corresponding remolded specimens are compiled to propose a
new VR-time relationship that is applicable for uncremented natural sands. Finally the
new VR-time relationship based on laboratory studies is compared with the VR-time
relationship based on field studies. It is hypothesized that a practical range of $N_{ys}$ values
can be recommended that is suitable for both laboratory and field tests.

6.2 VR-time relationship based on remolded sands

Several values of $N_G$ (and implied $N_{ys}$) for remolded sands have been reported
since the introduction of the term by Afifi and Richart (1973). Table 6.1 summarizes
results from eight independent laboratory studies which reported $N_G$ values of 15
different remolded clean sands.

Test method and pore fluid used during testing for each case is indicated in Table
6.1. First mode resonant frequency or shear wave travel time measurements needed to
compute $V_s$, $G_{max}$, and $N_G$ were obtained using either a resonant column (RC) device, a
triaxial device equipped with piezoceramic bender elements (BE), or a fixed ring
consolidometer equipped with BEs. RC tests were generally conducted on air dry
specimens, whereas $V_s$ travel time measurements in a triaxial device or consolidometer
were conducted on dry or saturated specimens.

Mineralogy, fines content (FC), mean grain size ($D_{50}$), relative density ($D_r$),
mean confining stress ($\sigma'_{mc}$) applied during testing, and $t$ are presented for each sand in
Table 6.1 based on available information. The majority of sands represented are
predominantly quartzofeldspathic with FC less than 1%. Jamiolkowski and Manassero
(1995) reported values of $N_G$ for sand with varying mineralogy.
<table>
<thead>
<tr>
<th>Case</th>
<th>Test Method&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Pore Fluid</th>
<th>Soil Type</th>
<th>Mineralogical Description</th>
<th>FC (%)</th>
<th>$D_{50}$ (mm)</th>
<th>$D_{R}$ (%)</th>
<th>$\sigma'_{mc}$ (kPa)</th>
<th>$t$ (days)</th>
<th>$N_{VS}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Afifi and Woods (1971)</td>
<td>RC</td>
<td>Air</td>
<td>Ottawa sand (30-50)</td>
<td>Quartz</td>
<td>0.0</td>
<td>0.45</td>
<td>92</td>
<td>206</td>
<td>100</td>
<td>0.5</td>
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<td>137</td>
<td>3</td>
<td>0.6</td>
</tr>
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<td>2. Wang and Tsui (2009)</td>
<td>RC</td>
<td>Air</td>
<td>Ottawa sand (20-30)</td>
<td>Quartz</td>
<td>0.0</td>
<td>0.8</td>
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<td>78</td>
<td>100</td>
<td>7</td>
<td>0.5</td>
</tr>
<tr>
<td>3. Afifi and Richart (1973)</td>
<td>RC</td>
<td>Air</td>
<td>Agsco No. 1 sand</td>
<td>Quartz</td>
<td>0.0</td>
<td>0.27</td>
<td>NA&lt;sup&gt;b&lt;/sup&gt;</td>
<td>137-275</td>
<td>4-70</td>
<td>0.5</td>
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<td>4. Ni (1987); Laird (1994)</td>
<td>RC</td>
<td>Air</td>
<td>Wash Mortar sand</td>
<td>40% Quartz, 30% Feldspar, 20% other minerals, 10% shell fragments</td>
<td>&lt; 1.0</td>
<td>0.45</td>
<td>65</td>
<td>21</td>
<td>0.7-1.4</td>
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<td>330</td>
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<td>0.6</td>
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<td>5. Human (1992)</td>
<td>BE</td>
<td>Air</td>
<td>Crystal Silica sand</td>
<td>92% Quartz (by weight)</td>
<td>1.4</td>
<td>0.38</td>
<td>21</td>
<td>50</td>
<td>3</td>
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<td>—</td>
<td>63</td>
<td>150</td>
<td>4</td>
<td>0.7</td>
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<td>6. Baxter and Mitchell (2004)</td>
<td>BE</td>
<td>Water</td>
<td>Evanston Beach sand</td>
<td>80% Quartz by weight</td>
<td>&lt; 1.0</td>
<td>0.3</td>
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<td>100</td>
<td>42</td>
<td>0.8</td>
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<td>100</td>
<td>28-122</td>
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<td>Water</td>
<td>Air</td>
<td>Density sand</td>
<td>99% Quartz by weight</td>
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<td>100</td>
<td>42</td>
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<td>100</td>
<td>28-120</td>
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<td>80</td>
<td>100</td>
<td>NA</td>
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Table 6.1 $N_{VS}$ values of remolded clean sands from laboratory aging studies.
Table 6.1 (continued) $N_{VS}$ values of remolded clean sands from laboratory aging studies.

<table>
<thead>
<tr>
<th>Case</th>
<th>Test Method$^a$</th>
<th>Pore Fluid</th>
<th>Soil Type</th>
<th>Mineralogical Description</th>
<th>FC (%)</th>
<th>$D_{50}$ (mm)</th>
<th>$D_{R}$ (%)</th>
<th>$\sigma'_{mc}$ (kPa)</th>
<th>$t$ (days)</th>
<th>$N_{VS}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7. Jamiolkowski and Manassero (1995);</td>
<td>NA</td>
<td>NA</td>
<td>A. Ticino sand</td>
<td>30% Quartz, 65% Feldspar, 5% Mica</td>
<td>NA</td>
<td>0.54</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>0.6</td>
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<tr>
<td>Jamiołkowski and Lo Presti (2003)</td>
<td></td>
<td></td>
<td>B. Hokksund sand</td>
<td>35% Quartz, 55% Feldspar, 10% Mica</td>
<td>NA</td>
<td>0.45</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>0.5</td>
</tr>
<tr>
<td>Jamiołkowski et al. (2003);</td>
<td></td>
<td></td>
<td>C. Messina gravelly sand</td>
<td>25% Quartz, 45% Feldspar, 30% Rock fragments</td>
<td>NA</td>
<td>2.1</td>
<td>NA</td>
<td>NA</td>
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<td>1.0-1.6</td>
</tr>
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<td>8. Wang and Tsui (2009);</td>
<td>RC</td>
<td>Air</td>
<td>D. Glauconite sand</td>
<td>50% Quartz</td>
<td>NA</td>
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<td>NA</td>
<td>NA</td>
<td>NA</td>
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<td>50% Mica</td>
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<td>E. Quiou sand</td>
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<td>Carbonate</td>
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<td>F. Kenya sand</td>
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<td></td>
<td>Carbonate</td>
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<td></td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>4.8</td>
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<tr>
<td>9. Gao et al. (2013)</td>
<td>BE</td>
<td>Air</td>
<td>Leighton Buzzard sand</td>
<td>Quartz</td>
<td>&lt; 1.0</td>
<td>0.23</td>
<td>23</td>
<td>35</td>
<td>7</td>
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<td>78</td>
<td>100</td>
<td>7</td>
<td>2.0</td>
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</table>

Notes:

$^a$RC = Resonant column; BE = Triaxial chamber or consolidometer with bender elements

$^b$NA = Not available
Values of $N_G$ were either reported by the authors or interpreted from plots of $G_{\text{max}}$ or $V_S$ versus time, $t_R = 1000$ min, and Eq. 6.1. $N_G$ values were converted to $N_{VS}$ using equation Eq. 2.4, if necessary. It is found in Table 6.1 that the average $N_{VS}$ (weighing each sand type equally) is 1.3%, and that $\approx 85\%$ of $N_{VS}$ values are between 0.4-2%.

Fig. 6.1 plots the $N_{VS}$ values versus mineralogy for four main mineralogical groups: quartz, quartz/feldspar, quartz/mica, and carbonate. An average $N_{VS}$ value (weighing cases within each group equally) is also plotted. It is seen that the $N_{VS}$ values presented for quartz and quartz/feldspar sands have similar ranges and the same mean of 0.8%. The $N_{VS}$ values presented for sand composed of quartz/mica (case 6D) and for carbonate sands (cases 6E and 6F) are higher than these average values, supporting the argument by Mesri et al. (1990) that $N_{VS}$ values of compressible soils is greater than $N_{VS}$ values of predominantly silica sands.

Presented in Figs. 6.2 are relationships between $N_{VS}$ and (a) $D_{50}$, (b) $D_R$, and (c) $\sigma'_{mc}$. Results of 15 types of clean sands are plotted in Fig. 6.2(a), whereas results of 8 and 9 types of clean sands are plotted in Figs. 6.2(b) and 6.2(c), respectively.

A slight logarithmic decrease in $N_{VS}$ with $D_{50}$ and slight linear increase in $N_{VS}$ with $D_R$ is observed in Fig. 6.2(a) and 6.2(b), respectively. The observation of a decrease in $N_{VS}$ with $D_{50}$ for particle sizes ranging from $10^{-1}$-2x$10^{0}$ agrees well with the decrease in $N_G$ with $D_{50}$ observed by Afifi and Richart (1973) for sands, silts, and clays with $D_{50}$.
of $10^{-3}$-$10^0$ and Ishihara (1996) for clay soils with $D_{50}$ of $10^{-4}$-$10^{-1}$. The increase of $N_{VS}$ with increasing $D_R$ and decreasing $D_{50}$ also suggests that $N_{VS}$ is mildly influenced by the number of particles in contact.

A small decrease in $N_{VS}$ with $\sigma'_mc$ is observed in Fig. 5.2(c). The data were found to exhibit a slightly better fit with the linear relationship depicted, rather than a logarithmic one, however there is considerable uncertainty for either.

Due to the degree of uncertainty in the relationships depicted in Figs. 5.2(a-c), a typical $N_{VS}$ value of about 1% and range of 0.5-2%, characterizes clean, predominantly quartzofeldspathic sands.

6.3 VR-time relationship based on intact and remolded sands

Compiled in Table 6.2 are results of laboratory studies conducted on high quality intact uncemented natural clean sands and remolded clean sands established from 9 independent studies for which values of VR and $t$ could be determined.

Cases 1-3 compare $G_{max}$ of six intact natural sands retrieved from sites in Japan by in situ ground freezing and frozen core sampling with $G_{max}$ of corresponding remolded sand specimens prepared by either dry vibration or air pluviation with matching densities ($\rho$). VR was calculated based on Eq. 6.1 and the relationship $G_{max} = \rho V_S^2$ in the following manner for these cases:

$$VR = \frac{V_{S,I}}{V_{S,R}} = \frac{\sqrt{G_{S,I}/\rho}}{\sqrt{G_{S,R}/\rho}} = \frac{\sqrt{G_{S,I}}}{\sqrt{G_{S,R}}} = \sqrt{\frac{G_{S,I}}{G_{S,R}}}$$

(6.2)
**Figure 6.1:** Influence of mineralogy on $N_{ys}$ based on laboratory tests.
Figure 6.2  Relationships between $N_{V_{55}}$ and (a) $D_{50}$, (b) $D_R$, and (c) $\sigma_{mc}$. 
Figure 6.2 (continued) Relationships between $N_{VS}$ and (a) $D_{50}$, (b) $D_R$, and (c) $\sigma'_{me}$.

$$N_{VS} = -0.0023\sigma'_{me} + 1.1$$

$r^2 = 0.085$

$n = 32$
Thus, for these cases $V_{S,t}$ is $V_S$ of an intact specimen retrieved from a deposit with known age (i.e., $t$) and $V_{S,R}$ is $V_S$ of a corresponding remolded specimen. Remolded specimens were first saturated and then consolidated prior to measuring $V_S$. The time required for saturation ranges from several hours to several days depending on the method used and the consolidation phase for cohesionless soils is relatively short ($< 1$ day). Therefore VR is based on a reference age ($t_R$) of about 3 days or $\approx 0.01$ years (i.e. $VR = VR_{0.01}$).

Intact Niigata, Tone, and Edo specimens described in the studies by Tokimatsu et al. (1986) and Kiyota et al. (2009b) (cases 1, 3A, 3B, 3C) were isotropically consolidated under $\sigma_{mc}$ equal to the estimated in situ effective overburden stress ($\sigma'_{v0}$) at the depths of frozen ground sampling.

For intact Higashi Ogishima, Yodo, Natori, and Edo specimens described in the study by Yamashita et al. (2003) (cases 2 and 4D), $G_{\text{max}}$ values of isotropically and anisotropically (at rest lateral earth pressure coefficient, $K_0 = 0.5$) consolidated intact and remolded specimens were determined under vertical consolidation stresses ($\sigma'_{v0}$) equal to $\sigma'_{v0}$. The values of $G_{\text{max}}$ recorded under each stress state were used to determine two VR values and then averaged to be consistent with the evaluations of the other cases (i.e., a single VR value per specimen).

$G_{\text{max}}$ values of intact and remolded Niigata sands (case 1) were evaluated from axial specimen deformation measured with highly sensitive deformation sensors housed
Table 6.2 Test details, physical properties, and results of laboratory studies on intact and remolded clean sands.

| Case/Study | Site/sand description | FC (%) | $D_{50}$ (mm) | $D_R$ (%) | $\sigma_{v0}$ (kPa) | $\sigma_{mc}$ (kPa) | Intact $G_{max}$ (MPa) | Remolded $G_{max}$ (MPa) | $t$ (years) | Velocity Ratio, $VR_{0.01}$
|------------|-----------------------|--------|--------------|-----------|------------------|-----------------|-------------------|--------------------------|-------------|------------------------
| 1. Tokimatsu et al. (1986); Tokimatsu and Uchida (1990) | Niigata | 0.2 | 0.30 | 87 | 98 | 78 | 54 | 0.5-10k | 1.20 |
| 2. Yamashita et al. (2003); Yamashita et al. (1997); Mimura and Suzuki (2002) | A. Higashi Ogishima | 1.9 | 0.24 | NA | 97 | 67-100 | 57-89 | 69-85 | 30 | 0.97 |
| | B. Yodo River | 0.2 | 0.3 | NA | 96 | 67-100 | 69-84 | 69-80 | 0.5-3k | 1.012 |
| | C. Natori River | 0.1 | 0.3 | NA | 77 | 53-100 | 89-114 | 72-103 | 0.5-10k | 1.082 |
| 3. Yamashita et al. (2003); Kiyota et al. (2009b) | A. Tone River | 1.2 | 0.19 | ≈70-100 | 100 | 100 | 93 | 71 | 8k | 1.14 |
| | B. Edo River 1 | 3.0 | 0.56 | ≈60-70 | 100 | 100 | 134 | 95 | 130k | 1.19 |
| | C. Edo River 2 | 2.9 | 0.19 | ≈70-100 | 160 | 160 | 283 | 142 | 130k-300k | 1.41 |
| | D. Edo River 3 | 0.0 | 0.85 | NA | 150 | 100-150 | 152-186 | 217-274 | 130k-300k | 1.20 |
| 4. Afifi and Woods (1971); Human (1992); Baxter and Michell (2004); Wang and Tsui (2009); Gao et al. (2013) | A. Ottawa (30-50) | 0.0 | 0.45 | 92 | — | 206 | — | — | — | 0.27 |
| | B. Ottawa (20-30) | 0.0 | 0.8 | 23-78 | — | 35-100 | — | — | — | 0.019 |
| | C. Agsco No 1 | 0.0 | 0.27 | NA | — | 69-275 | — | — | — | 0.011-0.19 |
| | D. Crystal silica | 1.4 | 0.35 | 21 | — | 50-300 | — | — | — | 0.0082-0.022 |
| | E. Evanston beach | < 1.0 | 0.3 | 40-80 | — | 100 | — | — | — | 0.12-0.16 |
| | F. Density | < 1.0 | 0.5 | 40-80 | — | 100 | — | — | — | 0.12-0.16 |
| | G. Toyoura | 0.0 | 0.23 | 23-78 | — | 35-100 | — | — | — | 0.019 |
| | H. Leighton Buzzard | < 1.0 | 0.15 | 78 | — | 50-200 | — | — | — | 0.0082 |

Notes:

$^aVR_{0.01} = VR$ based on a 3 day or $\approx 0.01$ year old remolded specimen.  
$^b$NA = Not available
inside a conventional triaxial chamber. $G_{\text{max}}$ determinations in the remaining cases were evaluated from $V_s$ travel time measurements using either accelerometers or bender elements attached to the side of the intact or remolded specimens at different heights.

Mineralogical compositions of sands retrieved from each site were not documented by the primary investigators. The Yodo and Natori specimens (cases 2B and 2C) are assumed to contain mainly quartz and feldspar because Mimura (2003) found that Yodo and Natori river samples retrieved from the same approximate depths consisted of $\approx 80\%$ quartz/feldspar. The Tone and Edo specimens sands (case 3) are assumed to contain mainly quartz, feldspar, and fragments derived from igneous rocks because Mimura (2003) found that Tone and Edo samples from shallower depths consisted of $\approx 40\%$ quartz/feldspar and $\approx 50\%$ rock fragments. The Niigata and Higashi Ogishima sands (cases 1 and 2A) likely contain significant proportions of quartz/feldspar particles based on their reported specific gravities of 2.69 and 2.73, respectively.

The estimated time since initial deposition ($t$) was available for cases 2A, 3A, 3B, 3C, and 3D and established for cases 1, 2B, 2C. Lower bound ages of 500 years were assumed for the Holocene-age Niigata, Yodo, and Natori sites (cases 2, 3B, 3C) because the samples were obtained at considerable depth ($>7$ m) with no indication of recent deposition. Upper bound ages of the Niigata and Natori deposits were assumed to be 10,000 years, which marks the end of the Holocene.

Case 4 consists of VR values which were interpreted from Table 6.1 case studies with known values of $t$ (cases 1-3, 5, 7-10), permitting calculation of VR based on $N_{VS}, t$, and Eq. 6.1. The VR values were normalized with respect to a $t_R$ value of 0.01 years (3
days), as opposed to 1000 minutes (0.7 days). One average VR value was assigned for each distinct sand type or value of t. Thus, case 4 consists of nine VR values determined from eight types of clean, quartzofeldspathic sands.

Figure 6.3 presents the relationship between VR and t based on 14 different clean sands. The logarithmic average t or geometric mean of lower and upper t values are used for plotting the data for sites with an estimated age range.

The best fit relationship between VR and t using Eq. 6.1 is also plotted in Fig. 6.3, which is a strong fit with the data (coefficient of determination $r^2 = 0.63$). The mean predicted $N_{vs}$ and $t_r$ values of 3.2% and 25 days, respectively and the 95% confidence interval in $N_{vs}$ corresponding to Eq. 6.1 is (1.9%, 4.6%). Thus, $N_{vs}$ predicted based on intact and remolded clean sands is higher than the typical $N_{vs}$ value of 1% predicted based on remolded clean sands only.

The $t_r$ value of 25 days based on Eq. 6.1 is high for VR values which are normalized with respect to 1-5 day old saturated remolded specimens. A second fitting of the dataset was performed using Eq. 6.1 and a fixed $t_r$ value of 0.01 years (3 days), resulting in a predicted $N_{vs}$ value of 2.8%, 95% confidence interval in $N_{vs}$ of (1.8%, 3.7%), and $r^2$ of 0.61. Thus, a $N_{vs}$ value of about 3% and range of about 2-4% characterizes the results compiled in Table 6.2.
Figure 6.3: VR-time relationship based on laboratory results of intact and remolded clean sands.
6.4 Comparison of laboratory and field based $V_S$-time relationships

Figure 6.4 compares the VR values referenced to 0.01 years ($VR_{0.01}$) determined in this study (solid circles) with VR values interpreted from MEVR ($VR_{6.2}$) based on in situ field test measurements and penetration resistance-$V_S$ relationships (open circles).

While the data points seen in Fig. 6.4 based on laboratory and field cases exhibit fair agreement, it is also seen that the predicted VR-time relationships of laboratory cases (solid line, $N_{VS} = 2.8\%$, $t_R = 0.01$ years) and field cases (dashed line, $N_{VS} = 8.2\%$, $t_R = 6.2$ years) are quite different. The difference in reference ages reflects the different normalization approaches used in computing VR (i.e., laboratory VR is $V_S$ normalized by its value at $t \approx 0.01$ years and field VR is $V_S$ normalized by its value at $t \approx 6.2$ years). The difference in $N_{VS}$ values however, indicates that the laboratory cases are characterized by a much shallower increase in VR with time than the field cases. If the $N_{VS}$ values were similar, the VR-lab values would plot at a constant amount above the VR-field values at corresponding ages.

The difference in $N_{VS}$ values was investigated further by considering the types of cases that were compiled in each study. The field cases included penetration resistance and $V_S$ pairs of predominantly sandy soils with fines content (FC) $< 20\%$, and classifying as SP, SP-SM, SP-SC, SM or SC by the United Soil Classification system. Cases from the intact/remolded laboratory test specimens used in this study involve clean sands (SP or SW) with FC $\leq 3\%$. Thus, the field cases include sands containing significant proportions of fines.
**Figure 6.4:** Comparison of VR-time relationships based on laboratory and field test results for sands.
The influence of fines content on the VR-time relationships for field cases is considered in Fig. 6.5. The VR values are separated by fines content based on the FC values provided in Andrus et al. (2009) into clean sands (SP) with FC ≤ 5%, sands with silt or clay (SP-SM or SP-SC) with 5 < FC ≤ 12%, and silty or clayey sands (SM or SC) with 12 < FC ≤ 20% and fitted separately using Eq. 6.1. The CPT soil behavior type index ($I_c$) corresponding to these groups are 1.38-2.11, 1.50-2.15, and 2.00-2.22, respectively. It should be noted that FC was estimated from soil behavior type index $I_c$ for 34 of 91 $V_s$-penetration resistance pairs (Andrus et al. 2009).

The relationships seen in Fig. 6.5 exhibit near constant values of $t_r$, ranging from 3.5-7.9 years, which are similar to the reference age of 6.2 years predicted with entire dataset. The predicted $N_{V_S}$ values are 5.2, 7.8 and 11.6% for the $F_C < 5\%$, $5 < F_C < 12\%$, and $12 < F_C < 20\%$ groups, respectively, exhibiting a general trend of increasing $N_{V_S}$ with $F_C$ and suggesting that $N_{V_S}$ is generally lower for soils with fewer fines for a given age. Thus, the mean VR-time relationship depicted in Fig. 6.4 generally overpredicts the VR values for clean sands, especially for sites older than 1000 years.

The VR values and VR-time relationships based on laboratory and field cases for clean sands only are replotted in Figure 6.6. The laboratory and field based relationships predict mean $N_{V_S}$ values of 2.8% and 5.2%, 95% confidence intervals of (1.8%, 3.7%) and (3.6%, 6.9%), and $r^2$ of 0.60 and 0.65, respectively. Despite a remaining mean $N_{V_S}$ difference of 2.4%, the two relationships are similar after removing non-clean sands from the set of field VR-age pairs.
Figure 6.5: Influence of fines content on the VR-time relationship based on field test results.
Figure 6.6: VR-time relationship based on laboratory and field result for clean sands.
One reason why $N_{Vs}$ values based on laboratory tests conducted on intact and remolded specimens may underestimate $N_{Vs}$ values based on field tests is because of disturbance during sampling or during preparation and thawing. Although it is likely impossible to assess disturbance due to sampling during frozen ground coring, a small change in void ratio during thawing is commonly reported during laboratory testing of frozen specimens (Hofmann 1997, Ghionna and Porcino 2003, Kiyota et al. 2009a, Chapter 3 of this study), likely caused by differing laboratory and field imposed stresses. Issues associated with disturbance could be avoided by directly comparing field $V_s$ with remolded $V_s$ values, provided the remolded specimens reflect the density and state of stress existing in situ.

Another possible reason is that dynamic laboratory measurements of $G_{max}$ are made at a greater shear strain levels ($\gamma = 10^{-5}$) than field measurements ($\gamma = 10^{-6}$), so laboratory $V_s$ values may be slightly degraded from field $V_s$ values.

A third possible reason is the influence of the remaining difference in the fines content between the laboratory and clean sand field cases. As seen in Table 5.2, the laboratory cases are characterized by an FC range of about 0-3% and a mean value of about 1%. The clean sand field cases are characterized by an FC range of 0-5% and a mean value of about 3%. Further attempts were made to investigate the field based VR-time relationship with FC ≤ 4% cases only, FC ≤ 3% cases only, etc., to compare even more closely with the laboratory based VR cases, however too many VR-time data points were removed in the process.
Finally, there may be additional differences between the properties of the specimens used in the laboratory cases and the sites investigated in the field cases such as, mineralogical compositon, range and average $D_{s0}$, $D_R$, and $\sigma'_m$, and isotropic versus anisotropic state of stress. Further comparison of the field and laboratory cases is needed to understand the relative importance of these properties on the VR-time relationships proposed in this study.

A dependency of MEVR on FC was not originally considered in the study by Andrus et al. (2009) because $V_s$, $N_{60}$, and $q_t$ measurements were adjusted to clean sand equivalent values. However, the finding is consistent with previous studies indicating that laboratory $N_G$ values vary significantly with fines content and plasticity (Mitchell and Soga, 2005). In Pleistocene and older sediments the presence of fines and light cementation may be correlated, as suggested by Kokusho et al. (2012). Further work is needed to understand and quantify the relationship between FC, $V_s$, and age.

### 6.5 Summary

A new VR-time relationship based on the results of intact and remolded clean sands was proposed in this chapter. The relationship extends the applicability of the VR-time equation proposed by Afifi and Richart (1973) to be used in natural uncemented sands.

VR-time relationships interpreted from field cases and from laboratory cases were compared. The results provide strong evidence that sands with fines aged at a faster rate that clean sands. VR-time relationships interpreted from field cases in clean sands only
compare more favorably with the relationship based on laboratory cases, however the field based relationship predicts a greater rate increase in VR with time. Further work is needed to explain the remaining difference in the rate increase of VR.

The relationships proposed in this chapter can be used as indices for degree of diagenesis. The relationships can also be used to predict stiffness gain with time occurring over the lifetime of an engineered fill, or to predict loss of stiffness due to disturbance. Finally, VR based on field cases may lead to improved penetration-\(V_S\) relationships.
CHAPTER 7

PREDICTED DYNAMIC RESPONSE OF A PLEISTOCENE SAND DURING AN IN SITU LIQUEFACTION TEST

7.1 Introduction

This chapter presents the results of a numerical study performed to predict the response of a Pleistocene age uncemented sand deposit during an in situ liquefaction test that was performed in April 2011 at the Coastal Research and Education Center (CREC) site. The study was conducted to assess the applicability of current numerical simulation software and constitutive soil models to predict cyclic strain accumulation and excess pore water generation in an aged soil deposit during dynamic loading. An advanced constitutive model intended for earthquake engineering applications is calibrated based on field and laboratory data available for the Pleistocene sand layer as a part of the study.

Numerical models are often calibrated with one set of experimental results before attempting to reproduce the results of a different experiment. In this study, however, the simulations are conducted without any results of the in situ liquefaction test. Therefore the purpose of the present study is to (1) collect available inputs needed to model the experiment, (2) explore the suitability of a sand plasticity model for applications involving aged sands, (3) develop a better understanding of the complex loading applied to the subsurface during the in situ liquefaction test, and (4) assess whether the predicted response based on the current modeling approach is reasonable.
7.2 In situ liquefaction test using a mobile field shaker

Procedures and equipment to instrument shallow liquefiable deposits (i.e., with accelerometers or pore pressure transducers) with little or no disturbance and induce cyclic shear strain from the ground surface using mobile field shaker trucks have been developed in an effort to characterize the in situ liquefaction resistance and dynamic stress-strain behavior of soils (Chang 2002, Rathje et al. 2005, Cox 2006).

A mobile field shaker is a hydraulically powered oscillator used as a dynamic source to produce shear stress waves from the ground surface that propagate downward through an instrumented area. A fleet of shakers is maintained and operated at the University of Texas at Austin Network for Earthquake Engineering Simulation (NEES) equipment site (https://nees.org/sites/?view=site&id=280, July 20, 2015).

Previous studies of in situ dynamic liquefaction tests utilizing field shaker (or vibroseis) trucks have been conducted where cyclic shear strain and excess pore water pressures were successfully generated. Experiments summarized by Chang (2002) and Rathje et al. (2005) involved operating the shaker trucks adjacent to an instrumented test pit filled with reconstituted sands in Austin, TX. The shaker was oscillated vertically, producing surface (or Rayleigh) waves that propagated laterally through the pit. Cox (2006) used newly developed push-in liquefaction sensors to instrument a native deposit at the Wildlife Liquefaction Array (WLA) in Imperial Valley, CA. At WLA, the shaker was positioned directly over an instrumented area and oscillated laterally with a specific force amplitude, producing downward propagating shear waves that passed through the sensor array.
A schematic of the test setup and methodology used at WLA is presented in Fig. 7.1. Five sensors were installed below the shaker truck, four specially designed liquefaction sensors consisting of a miniature accelerometers and a built-in pore pressure transducers (sensors 1-4) and one dedicated pore pressure transducer (PPT) (sensor 5). The sensors are pushed into a trapezoidal array, as they cannot be pushed into the ground directly on top of one another.

The rigid plate of the shaker was pressed down with a static force of 200 kN to provide solid coupling onto the ground surface. Then sinusoidal dynamic loading was applied in the horizontal direction at a specified frequency (typically 10 or 20 Hz), force amplitude (as high as 135 kN), and duration (up to 200 cycles). Several dynamic loading sequences are applied, starting with very small shaking levels in the linear elastic strain range, before increasing to a shaking level strong enough to induce plastic strain and pore pressure generation.

Presented in Fig. 7.2 are results of a dynamic loading sequence applied by “T-Rex”, the mobile field shaker truck used at WLA, which includes time histories of the force applied by the mobile field shaker, the shear strain ($\gamma$) at the center of the instrumented array and the excess pore water pressure ratio ($r_u$) recorded by each pore pressure transducer. The mobile field shaker was operated at its highest output, with a horizontal force amplitude of 135 kN and frequency of 10 Hz for 200 cycles. The mobile field shaker produces a relatively uniform force history which induces cyclic shear strains that increase in magnitude throughout the test as plastic strain accumulates, leading to excess pore pressure build up. In Fig. 7.2 the average value of $\gamma$ during 200
Figure 7.1: Mobile field shaker “T-Rex” and schematic of instrumented sensor array in a liquefiable soil layer (Cox 2006).
Figure 7.2: Time histories of force applied at the ground surface by T-Rex, shear strain induced at the center of the sensor array, and excess pore pressure ratio at each sensor location. The mobile field shaker was operated at its highest output (force amplitude 30 kips or 135 kN, frequency of 10 Hz, and duration of 200 cycles) (Cox 2006).
cycles was about 0.05% and the value of \( r_u \) after 200 cycles ranged from 10-30%.

Shear strain is evaluated from the acceleration time histories of the four accelerometers (sensors 1-4). The acceleration time histories are integrated twice to obtain displacement time histories. Each sensor is considered to be a node of a quadrilateral finite element and the displacement at the center of the array is interpolated from the displacements of each corner point based on a 4-node isoparametric element formulation (Chang 2002).

### 7.3 In situ liquefaction test at the CREC site

In situ liquefaction testing was conducted at the Coastal Research and Education Center (CREC) site by personnel from the University of Texas at Austin and the University of Arkansas with the T-Rex mobile field shaker in April 2011. As discussed in Chapter 3, a shallow sand layer exists at the CREC site which is believed to be a part of the 70,000 to 130,000 year old Wando Formation. The static strength of intact sand specimens retrieved from the CREC site was characterized in Chapter 3. The objective of the in situ liquefaction tests was to measure and characterize the in situ dynamic stress-strain behavior and liquefaction resistance of the sand deposit at CREC.

Plan and profile views depicting the liquefaction test setup are presented in Fig. 7.3. The distances from the edge of the 2.3 m by 2.3 m base plate of T-Rex to nearby cone penetration tests (i.e., RB4, RB5, and SC1) are also shown in Fig. 3 for reference (see Chapter 3, Fig 3.1 for a site map with locations of all investigations at the CREC site).
Fig. 7.4 presents a photograph of liquefaction sensors being installed at the CREC site. The sensors were installed near the centerline of the base plate in two trenches separated by about 0.6 m. In one trench, a trapezoidal array of accelerometers with embedded pore pressure transducers, and a centrally located dedicated pore pressure transducer were installed. In the other trench, two additional accelerometers with embedded pore pressure sensors and a second dedicated pore pressure transducer were installed. The sensors were spaced 0.3 m on center in the north-south direction.

The sensor array was installed in the middle of the sand deposit, which generally extends from depths of 0.6 m to 4.7 m below the ground surface. Boller (2008) subdivided the sand deposit into a denser region at depths of about 0.6 to 2.9 m and a looser region at depths of about 2.9 to 4.7 m, designated B1 and B2 respectively. The sensors were installed such that the top row of sensors was located in sublayer B1 and the bottom row of sensors was located in sublayer B2. The sensors were spaced 0.3 m on center with depth, with a top row, a middle row, and a bottom row of accelerometers with build in PPTs sandwiching two dedicated PPTs (3 and 6). The depth of the sensors is approximate. Just the configuration and spacing of the sensors was provided by request from the University of Texas at Austin (Personal communication, June 2015). The groundwater table was at a depth of 1.3 m at the time of the experiment.

A photograph of T-Rex situated above the installed liquefaction sensor array is presented in Fig. 7.5. The procedures described by Cox (2006) were used at the CREC site, with staged dynamic loading starting at small shaking amplitudes before attempting to liquefy the soil at large shaking levels. Results of the in situ liquefaction tests were not
Figure 7.3: Profile and plan views of the in situ liquefaction test setup at CREC. Depths of the sensors are approximate.
Figure 7.4: Photograph depicting the installation of an in situ liquefaction sensor at the CREC site. The two parallel instrumentation trenches in which the sensors were installed are visible in the photograph.
Figure 7.5: Photograph of the mobile field shaker “T-Rex” with the base plate centered over the liquefaction sensor array at the CREC site.
available to the author at the writing of this dissertation, however pore pressure transducer computer screen recordings observed at the time of testing indicated that the T-Rex shaker induced positive excess pore water pressures during a dynamic loading sequence at its highest force output of 135 kN. The pore pressure build up was limited and did not indicate that liquefaction occurred.

7.4 Numerical modeling inputs

Results presented in Chapter 3 and in previous investigations at the CREC site were used to create a generalized soil profile and to estimate inputs for numerical modeling. Illustrated in Fig. 7.6 are general ground conditions at the site. As seen in Fig. 7.6(a), a surficial sand layer (A) overlies layers B1 and B2. For the purposes of the simulations conducted in this study, it was assumed that Layer A extends to the groundwater table depth of 1.3 m. Layer C classifies as sand with clay and shells. Layer D is a stiff, cemented Tertiary deposit known as the Cooper Marl which extends to a considerable depth below the surficial deposits at CREC.

Compression wave velocities measured at CREC indicate that an unsaturated zone (degree of saturation, $S < 100\%$) exists to depths of about 2 m (Hossain et al. 2014). A reduced fluid bulk modulus ($K_w$) was assigned in this zone to account for its influence on pore pressure generation between the depths of 1.3 m and 2 m.

A profile of shear wave velocities at CREC from Hossain (2014) and a thick line representing the profile of $V_S$ assumed in this study is presented in Fig. 7.6(b). The
Figure 7.6: Profiles of (a) the generalized soil layering used in the numerical model, (b) shear wave velocity from Hossain (2014), (c) cone tip resistance, (d) friction ratio, (e) pore pressure, and (f) lateral earth pressure from Boller (2008).
abrupt increase in $V_S$ observed in Fig. 7.6 at a depth of 6 m marks the top of the stiff Cooper Marl. The profiles of cone tip resistance, friction ratio, and pore pressure measured during cone sounding RB4, which was pushed to a depth of about 6 m in the immediate vicinity of the base plate of the mobile field shaker, are presented in Figs 7.6(c-e). The low values of $FR$ and the value of $u_2$ equal to the hydrostatic pressure indicate little or no fines content at the location of the in situ liquefaction test. The profile of $q_t$ assumed in this study is also depicted in Fig. 7.6(c). The profile of assumed $K_0$ and the profile of $K_0$ estimated from DMT D1 by Boller (2008) is presented in Fig. 7.6(f). Within layers B1, B2, and C, $K_0$ of about 0.6 is found to be representative of the $K_0$ values estimated by Boller (2008), and agrees with the $K_0$ range of 0.4-0.6 suggested by Hossain (2014). Linear increases in $K_0$ to maximum values of 1.0 and 3.0 were assumed above layer B1 and below layer C, respectively based on the pattern of increasing $K_0$ within these layers.

Table 7.1 summarizes the main set of inputs that are not plotted in Fig. 7.6. The degree of saturation, wet density, and dry densities of the soils in layers A, C, and D were determined based on split-spoon samples collected by Boller (2008). In layer B, the wet and dry densities of intact frozen specimens (Chapter 3) were used. The initial shear modulus ($G_{max}$) was estimated from the densities and shear wave velocities of each layer according to the relationship,

$$G_{max} = \rho V_S^2$$

and the bulk modulus ($K_{max}$) was determined according to the relationship,
<table>
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<tr>
<th>Layer</th>
<th>Depths (m)</th>
<th>Soil Class</th>
<th>Degree of Saturation</th>
<th>Wet Density $\rho$ (kg/m$^3$)</th>
<th>Dry Density $\rho_d$ (kg/m$^3$)</th>
<th>Shear Modulus $G_{max}$ (MPa)</th>
<th>Bulk Modulus $K_{max}$ (MPa)</th>
<th>Peak Friction Angle (deg)</th>
<th>Dilation Angle (deg)</th>
<th>Cohesion Intercept (kPa)</th>
<th>Hydraulic Conductivity $k_H$ (cm/sec)</th>
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<td>1410</td>
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<td>1920</td>
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<td>0.002</td>
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<td>SP</td>
<td>100</td>
<td>1900</td>
<td>1440</td>
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<td>811</td>
<td>43</td>
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<td>0.00002</td>
</tr>
</tbody>
</table>

**Table 7.1:** Table of input properties used in numerical simulations.
where poisson’s ratio ($\nu$) was assumed to be 0.3 in all the layers.

Average peak friction angles ($\phi'_{p}$) in B1 and B2 were estimated from Bolton’s (1986) dilatancy index based empirical relationship, with the diagenesis-dilatancy correction and fitted parameter values for intact frozen specimens discussed in Chapter 3. The average $\phi'_{p}$ values in layers A and C were estimated from the empirical $q_{r} \phi'_{p}$ relationship of Uzielli et al. (2013) for predominantly quartz sands,

$$
\phi'_{p} = 25q_{1N}^{0.10} = 25 \left( \frac{q_{t}}{\sigma'_{v0}} \right) \left( \frac{\sigma'_{v0}}{P_{a}} \right)^{0.5}
$$

(7.3)

where $q_{1N}$ is the effective overburden stress normalized cone tip resistance, and $P_{a}$ is 101 kPa. Because layer C contains a significant percentage of fines and an average soil behavior type index ($I_{c}$) of 2.35 (based on RB4), the $q_{1N}$ value determined for layer C was corrected for the influence of fines content prior to evaluating Eq. 7.3 using the soil $I_{c}$-based correction proposed by Robertson and Wride (1998). This resulted in a clean sand correction factor ($K_{c}$) of 2.1. A small cohesion intercept ($c'$) value of 1 kPa was used for layers A, B, and C to account for the slight curvature of the peak shear strength envelopes.

Peak angles of dilation ($\psi$) were approximated based on the $\phi'_{p} - \psi$ relationship presented in Chapter 3 for CREC sand, i.e.,

$$
\phi'_{p} - \phi'_{c_{\text{crit}}} = 0.7\psi
$$

(7.4)
where the critical state friction angle \(\phi'_{\text{crit}}\) determined for intact CREC sand was assumed to be 33\(^\circ\) in layers A, B, and C.

The \(\phi'_p\) and \(c'\) values of the Cooper Marl (layer D) were reported in Camp (2004) based on the results of undrained triaxial compression tests. A higher \(\phi'_{\text{crit}}\) value of 37\(^\circ\) was used to approximate \(\psi\) with Eq. 7.4 as the Cooper Marl consists of rough irregularly shaped particles (Camp 2004).

Hydraulic conductivity \((k_H)\) in layer B was estimated based on the empirical correlations for clean granular soils between \(k_H\) and percentage of soil particles passing the No. 5 sieve \(D_5\) recommended in Terzaghi, Peck, and Mesri (2003), from the particle gradation curves presented in Chapter 3. Particle gradation curves of layer A were not available, and the \(k_H\) estimated for layer B was assumed. In layers C and D, a \(k_H\) value that is two orders of magnitudes lower than the \(k_H\) value estimated for clean sand layers were assumed.

Density of the ground water was assumed to 1000 kg/m\(^3\). The fluid bulk modulus was assumed to be zero above the groundwater table, 2.0x10\(^8\) Pa in the unsaturated zone, and 2.0x10\(^9\) Pa in the fully saturated zone.

### 7.5 Calibration of a sand plasticity model for CREC sand

Constitutive soil models intended for earthquake engineering applications have been developed to realistically predict the progressive accumulation of plastic strain during cyclic loading which lead to pore pressure generation and liquefaction (Manzari and Dafalias 1997, Yang et al. 2003, Byrne et al. 2004, Dafalias and Manzari 2004,
Generally, the models are calibrated against laboratory test results (i.e. direct simple shear, cyclic triaxial, shake table, centrifuge) of remolded sands.

A sand plasticity model called PM4Sand, introduced by Boulanger and Ziotopoulou (2013) is used in this study to model the in situ response of CREC sand. The PM4Sand model was developed by modifying the bounding surface plasticity model for sand presented by Dafalias and Manzari (2004) to better approximate engineering design correlations commonly used in liquefaction analysis (Boulanger and Ziotopoulou, 2013). The PM4Sand model was intended to be practice oriented as summarized in Ziotopoulou and Boulanger (2013):

“The goal of the generalized calibration of the model was to produce drained and undrained monotonic and cyclic responses under a broad range of stress conditions that are reasonably consistent with the behaviors expected based on engineering correlations to commonly available in-situ test data (i.e., SPT, CPT, and $V_s$ data).”

To that end, Boulanger and Ziotopoulou (2015) presented a calibration procedure using three primary inputs and 18 secondary inputs. The primary inputs are the relative density ($D_R$), the mean effective stress normalized shear modulus ($G_o$), and the contraction rate parameter ($h_{po}$). $G_o$ is the primary input variable controlling elastic strains, $D_R$ is the primary input variable controlling plastic volumetric strain during dilation, and $h_{po}$ is the primary input controlling plastic volumetric strain during contraction within the PM4Sand model architecture.
The value of $D_R$ is estimated from the in situ void ratio ($e$), or through empirical relationships with penetration resistances. The parameter $G_o$ is the value of $G_{\text{max}}$ under 101 kPa of mean effective confining stress ($p'$) calculated from,

$$G_o = \left( \frac{G_{\text{max}}}{p_a} \right) \left( \frac{p_a}{p'} \right)^{0.5}$$

(7.5)

where Pa is a reference stress of 1 atmosphere or 101 kPa.

After $D_R$ and $G_o$ are established, the value of $h_{po}$ is obtained through calibration by performing single element simulations to achieve a desired cyclic resistance ratio (CRR) (e.g., CRR values for an effective overburden stress of 1 atm. and an earthquake magnitude of 7.5 based on liquefaction triggering correlations), and simultaneously meeting a desired failure criteria (e.g., 3% single amplitude shear strain in 15 uniform stress cycles). Therefore, a target CRR value is an additional model input. A code for performing the calibrations in FLAC is available at the code developers website ([http://faculty.engineering.ucdavis.edu/boulanger/pm4sand/](http://faculty.engineering.ucdavis.edu/boulanger/pm4sand/), June 15, 2015).

### 7.5.1 Calibration inputs

Adopted values of $D_R$, $G_o$, CRR and other parameters used to calibrate the PM4Sand model for CREC sand are summarized in Table 7.2. The average $D_R$ values of 70% and 62% were established from the $D_R$ of intact frozen specimens reported in Chapter 3 within depths of 2-4 m and from moisture content measurements obtained from split spoon samples by Boller (2008) above 2 m in B1 and below 4 m in B2, respectively assuming a specific gravity of solids ($G_S$) of 2.68 and degree of saturation ($S$) of 100%.
The $G_o$ values were determined by evaluating Eq. 7.5 using the $G_{max}$ values reported in Table 7.1.

The CRR values are based on the liquefaction assessment of the CREC site by Hossain (2014). The assessment accomplished using seismic cone SC1 data and the general CPT-based procedure recommended by Youd et al. (2001), which indicated an average CRR value of about 0.35 within layer B. The seismic cone records of SC1 were preferred over the closer cone records of RB4 because shear wave velocity measurements were also available. Therefore, the measured to estimated velocity ratio (MEVR) and the deposit resistance correction factor ($K_{DR}$) proposed by Hayati and Andrus (2009) could be evaluated to correct for the influence of diagenesis on liquefaction resistance based on the equation,

\[
CRR_K = CRR \times K_{DR}
\]

(7.6)

where

\[
K_{DR} = 1.08 \text{MEVR} - 0.08
\]

(7.7)

Average MEVR values of 1.4 and 1.2 based on SC1 predict $K_{DR}$ values of 1.43 and 1.14 and corrected $CRR_K$ values of 0.5 and 0.4 in B1 and B2, respectively.

A second correction was applied based on the diageneis dilatancy model developed for intact CREC sand specimens (Chapter 3). The stress ratio at peak strength, stress ratio at the onset of dilatancy, and the plastic volumetric strain that occurs after the

<table>
<thead>
<tr>
<th>Layer</th>
<th>$e_{min}$</th>
<th>$e_{max}$</th>
<th>$D_R$</th>
<th>$G_o$</th>
<th>CRR</th>
<th>MEVR</th>
<th>$K_{DR}$</th>
<th>$CRR_K$</th>
<th>$n_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0.68</td>
<td>1.15</td>
<td>70</td>
<td>1510</td>
<td>0.35</td>
<td>1.4</td>
<td>1.43</td>
<td>0.5</td>
<td>0.55</td>
</tr>
<tr>
<td>B2</td>
<td>0.68</td>
<td>1.15</td>
<td>62</td>
<td>960</td>
<td>0.35</td>
<td>1.2</td>
<td>1.14</td>
<td>0.4</td>
<td>0.62</td>
</tr>
</tbody>
</table>

Table 7.2: Inputs used to calibrate the PM4Sand model.
onset of dilation depend on three terms which are predicted from empirical formulae in
the PM4Sand model by default. These equations are:

\[ M_b = M e^{-n_b \xi_R} = 2 \sin \phi'_p \]  \hspace{1cm} (7.8)

\[ M_d = M e^{-n_d \xi_R} = 2 \sin \phi'_d \]  \hspace{1cm} (7.9)

\[ A_{do} = \frac{1}{0.4} \frac{\sin^{-1}(M_b/2) - \sin^{-1}(M/2)}{M_b - M_d} \]  \hspace{1cm} (7.10)

where \( M_b \) is the deviatoric to mean effective stress ratio \( (q/p') \) at peak strength, \( \phi'_p \) is
the peak friction angle, \( M_d \) is the value of \( q/p' \) at the onset of dilation, \( \phi'_d \) is the
friction angle at the onset of dilatancy, \( M \) is the value of \( q/p' \) at critical state strength
\( (M = 2 \sin \phi'_{c_v}) \), \( \xi_R \) is the relative state parameter index, \( n_b \) and \( n_d \) are secondary inputs
of the PM4Sand model with default values of 0.5 and 0.1 respectively, and \( A_{do} \) is a factor
which controls the incremental volumetric strain for a given increment of deviatoric
stress when \( q/p' > M_d \). The relative state parameter index, \( \xi_R \) accounts for the influence
of \( D_R \) and \( p' \) on \( M_b \) and \( M_d \) through the formula:

\[ \xi_R = \frac{1.5}{10 - \ln(100 p'/p'_a)} - D_R \]  \hspace{1cm} (7.11)

For the low confining pressure range \( (p' < 50 \text{ kPa}) \) of interest in this study, the
default value of \( n_b = 0.5 \) predicts \( \phi'_p \) values that are comparable to the values obtained
for dense CREC sand in Chapter 3 \( (D_R \approx 75-85\%) \) but less than the \( \phi'_p \) values for loose
intact CREC sand specimens \( (D_R < 75\%) \). Adjusted \( n_b \) values of 0.55 and 0.62 were
obtained by solving Eqs. 7.8 and 7.11 for \( n_b \) with \( \phi'_p \) values of 46 and 44, \( D_R \) of 70% and 62%, and \( p' \) of 32 and 38 kPa for B1 and B2, respectively (considers the increase in \( p' \) caused by application of the mobile shaker plate load).

### 7.5.2 Calibration results

Fig. 7.7 presents the simulated response of a single PM4Sand element in undrained direct simple shear under a vertical effective confining stress of 101 kPa, and cyclic stress ratio (CSR) of 0.4 for layer B2. The responses recorded during the simulation in Fig. 7.7 are (a) the cyclic shear strain (\( \gamma \)) with number of uniform loading cycles, (b) the excess pore pressure ratio (\( r_u \)) with number of uniform loading cycles, (c) the cyclic stress ratio (CSR)-strain relationship, and (d) the relationship between CSR and normalized vertical effective stress (\( \sigma'_v/\sigma'_{v0} \)). In the simulations, stress controlled cyclic loading was applied until a single amplitude strain of 3% was reached.

The PM4Sand element was assigned the properties in Table 7.2 and the value of \( h_{po} \) was varied until 3% strain occurred in 15 cycles. It was found during this process that the predicted response of the element became more dilatant, and therefore more resistant to excess pore pressure buildup and shear strain accumulation, as \( h_{po} \) was increased. This behavior was observed because the amount of plastic volumetric strains during contraction is calculated from a term that is inversely proportional to \( h_{po} \) within the PM4Sand model architecture (Boulanger and Ziotopoulou 2015).

As seen in Fig. 7.7(a) 3% single amplitude strain is achieved in 15 cycles with a \( h_{po} \) value of 22. A similar calibration process for B1 yielded an \( h_{po} \) value of 5.5. These values of \( h_{po} \) are large compared to the calibrated hpo values of 0.40 and 0.63.
recommended by Boulanger and Ziotopoulou (2015) for medium to dense sand, but do not seem to inhibit the generation of excess pore pressure or degradation of stiffness during loading as observed in the plots of excess pore pressure versus number of loading cycles (Fig. 7.7b), and the CSR-strain relationships (Fig. 7.7c). Similar behaviors were observed for a single element simulation using the inputs of B1.

High values of $h_{po}$ are needed to match the target CSR values because the inputs for B1 and B2 pair relatively low $D_R$ values with large CSR values. A low $D_R$ value results in a smaller value of the relative state parameter index (Eq. 7.10) which decreases the factor by which plastic volumetric strains occur during dilation and increases the factor by which plastic volumetric strain occurs during contraction. Aged soils tend to be more dilative and less contractive than remolded soils for the same initial state, therefore increasing $h_{po}$ to model a more dilatant response is reasonable. This approach may be limited for certain pairings of low $D_R$ and high CSR needed to model diagenesis for different sites, however, because assigning too high a value of $h_{po}$ was found to restrict excess pore pressure buildup and stiffness degradation in single element simulations.

Presented in Fig. 7.8 are relationships between CSR and number of cycles to cause 3% single amplitude strain using the calibrated inputs for B1 and B2. Each curve is produced from the results of five simulations performed at different CSR values. The curves obtained for B1 with $D_R$ of 70% are initially steeper than the curves obtained for B2 with $D_R$ of 62% but level off to similar values of CSR beyond about 50 loading cycles. Therefore, the responses of each layer during 200 cycles of dynamic loading are expected to be fairly similar after about 50 loading cycles.
Figure 7.7: Response of a single PM4Sand element with the calibrated inputs for layer B2 in stress controlled undrained direct simple shear. Depicted in the figure are relationships of (a) shear strain and number of stress cycles, (b) excess pore pressure ratio and number of cycles, (c) cyclic stress ratio and shear strain, and (d) cyclic stress ratio and vertical effective stress.
Figure 7.8: Cyclic strength curves produced by conducting a series of PM4Sand element DSS simulations based on the calibrated inputs for B1 and B2.
7.6 Numerical modeling procedure

Fully coupled plane strain simulations of the in situ liquefaction test were performed with the commercial finite difference program FLAC 7.0 (Itasca 2011). FLAC was used to perform the simulations because the finite difference solution procedure is well suited to nonlinear geotechnical engineering applications (Itasca 2011), inputs for the simulation are available, and the PM4Sand model is already implemented as a user defined material model (Ziotopoulou and Boulanger, 2013). A code used for performing the simulations with FLAC in this study is included as Appendix D.

The problem domain modeled in this study is presented in Fig. 7.9. A 10 m high profile including the five soil layers A, B1, B2, C, and D, a water table at a depth of 1.3 m, and a rigid plate located at the center of a symmetric grid composed of uniformly sized elements were modeled in FLAC. All soil layers were first assigned a Mohr-Coulomb material model with \( k_H \), \( G_{\text{max}} \), \( K_{\text{max}} \), \( \phi' \), \( \psi \) and \( c' \) values given in Table 7.1 as inputs for calculating initial stresses and for simulating the static vertical plate loading.

The base plate of T-Rex shown in Fig. 7.9 was modeled with rigid beam elements. The nodes of the plate and ground surface were connected assuming no slip at the plate/ground interface.

The left and right boundaries of the grid were initially fixed in the horizontal direction and the bottom boundary was fixed in the vertical and horizontal direction. The vertical and lateral stresses under gravity loading were initialized, then a
Figure 7.9: Illustration of the problem domain modeled in FLAC.
uniform vertical stress of 38 kPa (equal to 200 kN divided by base plate area of 5.29 m$^2$) was applied to the base plate under drained conditions.

The material model for the zones located within 20 m of the base plate centerline in layer B was changed to PM4Sand for dynamic analysis with the $D_R$, $G_o$, and $h_{po}$ values obtained from calibration, as illustrated in Fig. 7.9. In addition, the top row of elements within 2.3 m of the base plate were replaced with a stiff zone with tension and cohesion values of 100 kPa to prevent the soil immediately surrounding the base plate from failing and prematurely halting the simulation.

The left, right, and bottom boundaries of the model were changed to quiet, or absorbent, boundaries (Fig. 7.9a), which are used when a dynamic source is located within the model, as opposed to seismic ground shaking scenarios in which the entire base is displaced and free field boundary conditions are used. Quiet boundaries are based on the viscous boundary developed by Lysmer and Kuhlemeyer (1969) which involves dashpots attached independently to the boundary in the normal and/or shear directions. The dashpots provide viscous normal and shear tractions that are calculated and applied at every timestep in the same way that boundary loads are applied (Itasca 2011).

The grid was configured for large strain and groundwater flow, initial x and y displacements were zeroed and a harmonic shear stress time history with amplitude of 25 kPa (equal to 135 kN divided by base plate area of 5.29 m$^2$) and frequency of 10 Hz was applied to the base plate to simulate the dynamic loading provided by T-Rex at its highest shaking output. Full Rayleigh damping of 1% at a central frequency of 10 Hz was applied during shaking to account for mechanical dissipation of energy within the model. The
value of 1% is an approximate average of the minimum damping ratio values for the soils within the profile based on the $D$-$\gamma$ relationships proposed by Zhang et al. (2005) for Quaternary and Tertiary soils.

The simulations were run for 20 seconds or 200 cycles of applied loading and 5 seconds without loading.

### 7.6.1 Domain sensitivity

Initial simulations indicated that the response of the model was influenced by the location of the quiet boundaries. The ratio of domain width to height ($a_R$) was studied by varying the width of the model while keeping the height of the model and element size constant. Fig. 7.10 presents profiles of the maximum shear strain at (a) a distance of 0.5 m from the model centerline and (b) at the model centerline. It is seen in Figs. 7.10(a) and (b) that the domain width influences the maximum shear strain values between depths of about 0-6 m but has little influence on the values below 6 m, indicating a model depth of 10 m is sufficient. The maximum shear strain values in the top 6 m appear to converge with an $a_R$ ratio of 7:1. Maximum values of vertical displacement along the ground surface (not shown) were also found to converge at an $a_R$ of 7:1, therefore a model width of 70 m was selected.

### 7.6.2 Mesh sensitivity

The sensitivity of the model to the resolution of the mesh was studied by varying the element size globally while maintaining a width of 70 m and a height of 10 m. Fig. 7.10 presents profiles of the maximum shear strain at (a) a distance of 0.5 m from the model centerline and (b) the model centerline for element sizes of 0.5, 0.33, and 0.25 m.
Figure 7.10: Influence of domain width on the profile of maximum shear strain at (a) a distance of 0.5 m from the model centerline, and (b) at the model centerline.
Figure 7.11: Influence of element size on the profile of maximum shear strain at (a) a distance of 0.5 m from the model centerline, and (b) at the model centerline.
(total number of elements are 2,800, 6,300, and 11,200 elements respectively). It is seen in Figs. 7.11(a) and 7.11(b) that the maximum shear strain profiles with element sizes of 0.33 m and 0.25 m are reasonably similar below a depth of about 2 m, however the smaller element size of 0.25 m was selected for better accuracy.

7.7 Results

Fig. 7.12 presents predicted time histories of (a) horizontal acceleration, (b) horizontal displacement, and (c) vertical displacement of the rigid base plate during the simulation. It is seen in Fig 7.12(a) and Fig. 7.12(b) that the dynamic loading produced a constant horizontal acceleration of about 1.6 m/s² and horizontal displacement of about 0.35 mm. The base plate settled throughout the dynamic portion of the simulation by about 1.2 mm and rebounded to a final displacement of about 0.9 mm during unloading (Fig. 7.12c).

7.7.1 Response in the vicinity of the mobile field shaker

Fig. 7.13(a) presents contours of maximum shear strain (γ), expressed as percentages, that were induced by ground shaking. It is seen that the main effects of dynamic loading were observed within a horizontal distance of about 5 m from the base plate centerline and vertical distance of 5 m. The strain concentration is symmetric about the centerline but non uniform, as large concentrations of shear strain with maximum values of about 0.2% are observed just outside the edges of the rigid base plate. This observation is consistent with the operating principles of the mobile field shakers discussed by Menq et al. (2010) who explain that large alternating vertical stresses are
Figure 7.12: Time histories of predicted (a) acceleration (b) horizontal displacement and (c) vertical displacement at the center of the rigid base plate.
produced at the edges of the base plate during shaking. Directly beneath the base plate, the distribution of shear strain is more uniform and characterized by lower amplitudes of maximum shear strain.

Fig. 7.13(b) and 7.13(c) present contours of the maximum excess pore pressure ratio \( (r_u) \) and maximum cyclic stress ratio (CSR) that were induced by ground shaking. Like maximum shear strain, \( r_u \) and CSR values are highest within zones that are outside of the edges of the base plate. Underneath the base plate and within the area of the instrumented sensor array, \( r_u \) and CSR values are fairly low \( (r_u < 23\% \text{ and } \text{CSR} < 0.15) \). Thus, the model predicts the development of realistically low excess pore water pressures, which generally agrees with the limited field observations (i.e. excess pore water pressure increased by a small amount) that were available to the author at the time of testing.

7.7.2 Response within the sensor array

It is clear from Fig 7.13 that the mobile field shaker produces a non-uniform distribution of stresses and strains within its zone of influence. This behavior is also observed within the liquefaction sensor array as illustrated in Fig. 7.14(a-c), which plots the predicted stress-strain relationships of (a) an element 0.25 m to the left of the plate centerline, (b) at the plate centerline, and (c) 0.25 m to the right of the plate centerline in layer B1; and in Fig. 7.14 (d-f), which plots the predicted stress-strain relationships of elements with the respective horizontal positions in the sensor array, but in layer B2. As observed in Fig. 7.14 the elements on the left side of the sensor array (7.14a and 7.14d)
Figure 7.13: Predicted contours of (a) the maximum shear strain ($\gamma$), (b) the maximum excess pore pressure ratio ($r_u$), and (c) the maximum cyclic stress ratio (CSR) in the vicinity of ground shaking induced during dynamic loading.
Figure 7.14: Relationships between cyclic stress ratio and shear strain during ground shaking at (a) 0.25 m left of the plate centerline, (b) the plate centerline, and (c) 0.25 m right of the plate centerline in layer B1; and at (d) 0.25 m left of the plate centerline, (e) the plate centerline, and (f) 0.25 m right of the plate centerline in layer B2.
and the elements on the right side of the sensor array (7.14c and 7.14f) gradually accumulate small incremental strains away from the plate centerline. It is seen in all of the plots that the stress level applied during shaking caused very little plastic shear strain accumulation or stiffness degradation. The highest CSR value is about 0.1, which plots well below the calibrated cyclic strength curves in Fig. 7.8 if they are extrapolated to 200 cycles.

The general behavior of the CREC sand in layers B1 and B2 was studied by averaging shear stress, shear strain, and excess pore water pressures recorded in all elements within the instrumented zones (i.e. within 0.5 m of the plate centerline and depths of about 2.5-3.5 m).

Time histories of the (a) average cyclic stress ratio, (b) average shear strain, and (c) average excess pore water pressure induced by the mobile shaker in B1 and B2 are presented in 7.15(a-c) and 7.16(a-c), respectively. Maximum cyclic shear stresses observed in layers B1 and B2 are about 0.05, however the soil in B2 has a lower initial shear modulus and cyclic shear strength, which caused the elements in B2 to strain more during the dynamic loading sequence.

The shear strain time histories in B1 and B2 indicate characteristic shear strain values of about 0.004% and 0.005%, respectively. This magnitude of shear strain is approximately at the threshold for pore pressure generation (Dobry et al. 1982, Chang 2002, Cox 2006). As seen in Figs. 7.15(c) and 7.16(c), the excess pore pressures exhibit small increases within the first 2.5 seconds (or 25 cycles) of loading, reaching maximum \( r_u \) values of about 13% and 18% in B1 and B2, respectively, before gradually dissipating.
Figure 7.15: Time histories of the (a) average cyclic stress ratio, (b) average shear strain, and (c) average excess pore pressure ratio within layer B1.
Figure 7.16: Time histories of the (a) average cyclic stress ratio, (b) average shear strain, and (c) average excess pore pressure ratio within layer B2.
Time histories of volumetric strain during loading (not shown) in B1 and B2 indicated that elements contracted during the initial period of 25 cycles when $r_u$ increased and dilated during the remaining 175 cycles of loading. Thus it is believed that the sands within the sensor array at the CREC site were too dilatant to liquefy during field testing under the given loading.

The results shown in Figs. 7.15 and 7.16 are similar to dynamic loading sequences performed at WLA that did not generate significant pore pressures. Referring to Fig. 7.2, a clear increase in cyclic strain is observed when significant pore pressures are generated.

Shear strains as high as 0.1% were observed in the instrumented zones at WLA when the mobile shaker was operated at its highest output. At present, it is speculated that the difference in soil properties and the influence of diagenesis explain why a lower shear strain of 0.005% is predicted at CREC. The average shear wave velocity in the top 5 m is $\approx 180$ m/s at CREC and $\approx 105$ m/s at WLA (Cox 2005). Simulations of in situ liquefaction testing at the WLA site could be performed in the future and compared with published experimental results to confirm the modeling approach used in this study. In addition, experimental results of in situ liquefaction testing at the CREC site can be used to validate results of this study when they become available.

### 7.8 Summary

Results of numerical simulations performed to predict the response of a Pleistocene age uncremented sand deposit at the CREC site during an in situ liquefaction test were presented in this chapter. A constitutive sand plasticity model intended for earthquake engineering applications was calibrated to represent the nonlinear dynamic
stress-strain behavior of the Pleistocene sand deposit. Calibration of the model required considerably adjusting the contraction rate parameter using the procedure recommended by Boulanger and Ziotopoulou (2015) due to the relatively low density and high predicted cyclic strength of the CREC sand.

Results of numerical simulations of the in situ liquefaction test agree with field observations that the soil at CREC did not liquefy or generate significant excess pore water pressure. Further work is needed however, to verify that cyclic shear strains predicted by the simulations are realistic for the CREC site and to compare the predictions with the measured excess pore pressure recordings.
CHAPTER 8
CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

The dissertation investigated the influence of diagenesis on the static peak shear strength and dilatancy behavior, and the small strain dynamic stiffness of natural sands.

The static peak strength and dilatancy behavior of intact sand specimens retrieved from a Pleistocene deposit at the CREC site and remolded specimens prepared with equal densities were compared based on drained triaxial compression test results in Chapter 3. Intact specimens of all densities were characterized by a dilatant response and higher peak strengths compared to remolded specimens. With increasing confining pressure, the dilatancy was reduced. It was concluded that intact specimens exhibit dilatancy due to density and aging processes, which are both suppressed under high confining pressures. For this reason, an existing model for estimating the peak strength and dilatancy of sands was found to be poorly suited to characterizing the strength and dilatancy of both intact and remolded sands unless a diageneis-dilatancy term was added. The resulting general model implied that density effects on peak strength of intact and remolded specimens are the same. The diagnosis-dilatancy term and the measured to estimated velocity ratio (MEVR) exhibited similar variations when plotted with depth, indicating a relationship between the two factors.

A profile of in situ peak friction angle values was compared with peak friction angles estimated from cone tip resistance and shear wave velocity. The predictions based
on high strain cone tip resistance were close to the in situ values and depicted a similar variation with depth for a majority of the profile. The predictions based on small strain shear wave velocity overpredicted the in situ peak friction angle values. Thus, a correction to small strain shear wave velocity based predictions may be needed in older sediments.

In Chapter 4, the diagenesis-dilatancy term determined in Chapter 3 was generalized as a function of age or MEVR by considering the triaxial compression test results determined by various investigators for ten different sands. Strong evidence was shown that diagenesis-dilatancy increases with time, and that a model including age and confining pressure terms significantly improved predictions over a model with no age term. Therefore an age-dilatancy model was proposed. It was also shown that other properties, such as density, do not improve the age-dilatancy model, supporting the conclusion in Chapter 3 that density has little influence on dilatancy due to age.

MEVR of the sites compiled in the study were determined or predicted from MEVR-time relationships to investigate its variation with diagenesis-dilatancy. It was shown that MEVR correlates strongly with age. Therefore, an MEVR-dilatancy model was also proposed. The MEVR- and age-dilatancy models suggested similar variations with confining pressure, indicating the dilatancy due to diagenesis is suppressed under high confining pressures. The MEVR- and age-dilatancy equations were recommended to estimate intact peak friction angle from remolded peak friction angle or for predicting loss of strength during a disturbance or under large surcharges provided reliable in situ peak friction angle estimates are available. General models for estimating peak strength
are implied by the MEVR- and age-dilatancy equations and can be used once the model is validated with the data presented in this study and the data compiled by Bolton (1986).

The relationships proposed in Chapter 4 were used to determine a corresponding peak shear strength-time relationship. A plot of the resulting peak shear strength-time relationship is compared with the $K_{DR}$-time relationship proposed by Andrus et al. (2009). From the comparison it is found that liquefaction resistance is far more sensitive to age than static shear strength. It is also possible that liquefaction resistance due to diagenesis is suppressed by high confining pressures.

In contrast to the evidence that peak strength increases due to diagenesis, there is little evidence among the cases studied in Chapters 3, 4, and 5 of an improvement in residual shear strength due to diagenesis. As a result, large deformations can result from post peak strain softening if the peak shear strength of an older deposit is exceeded during an extreme loading scenario (e.g. combined dynamic and static loading).

In Chapters 2 and 6, it was shown that the $G_{max}$-time relationship proposed by Afifi and Richart (1973) and the MEVR-time relationship proposed by Andrus et al. (2009) could be related using a term called velocity ratio VR (the ratio of shear wave velocity at a given time, relative to a reference value). VR datasets were established from laboratory tests conducted on remolded sands and from laboratory tests conducted on intact sands and remolded sands. The VR datasets were combined to propose a VR-time relationship intended for natural sands. The proposed VR-time relationship was found suggest a change in shear wave velocity of about 3% for each ten fold change in age, which is higher than the rate of about 1% observed from laboratory tests conducted on
remolded sands only. However, the VR-time relationship from field tests in natural sands predicts a much higher rate change of 8% with each ten fold change in age.

A major difference in the laboratory VR datasets and field test VR datasets is fines content. It was shown that the slope of the VR-time relationship from field test varied significantly for clean sand, silty or clayey sand, and sand with silt or clay groups, providing strong evidence of an influence of fines on diagenesis. Much closer agreement between the VR-time relationships of field and laboratory cases for clean sands were observed.

The relationships proposed in Chapter 6 were recommended to be used as indices for degree of diagenesis, however the influence of fines content must be considered in such an assessment. The relationships were also recommended to predict stiffness gain with time occurring over the lifetime of an engineered fill, or to predict loss of stiffness due to disturbance. Finally, VR based on field cases is a factor that can be used to correct empirical correlations between penetration resistance and $V_s$.

Chapter 7 summarized the preliminary results of a numerical study to predict the response of a Pleistocene age natural sand deposit during an in situ liquefaction experiment involving the NEES@UTexas mobile field shakers at the CREC site. Fully coupled plane strain simulations were performed using a model that consisted of a 10 m deep and 70 m wide soil profile composed of 11,200 elements. A centrally located 2.3 m wide rigid base plate, was statically loaded in the vertical direction, and then dynamically loaded in the horizontal direction for 200 cycles.
The PM4Sand model (Boulanger and Ziotopoulou 2013, 2015), a plasticity model intended for earthquake engineering applications, was used for the Pleistocene sand layer, which was divided into a denser region (B1) and a looser region (B2). Calibration of the model required considerably adjusting one of three main model inputs called the contraction rate parameter using the procedure recommended by Boulanger and Ziotopoulou (2015) due to the relatively low density and high predicted cyclic strength of the CREC sand. This approach may be limited for certain pairings of density and cyclic strength because increasing the contraction rate parameter too much was found to restrict excess pore pressure buildup and stiffness degradation in simulations involving PM4Sand elements.

The simulation of the in situ liquefaction test predicted concentrations of cyclic shear strain, cyclic stress ratio, and excess pore pressure that were located near the corners of the base plate during shaking, and tended to produce a biased accumulation of shear strain toward either side of the sensor area. Within the sensor array, directly below the rigid base plate the cyclic shear strain was just at the threshold for excess pore pressure generation and the excess pores pressure ratio was predicted to reach a maximum value of 12% in layer B1 of the 18% in layer B2. The prediction of low excess pore pressure buildup agrees with the limited field observations that were available to the author at the writing of this dissertation.
8.2 Recommendations

The following are recommendations for future works:

1. Only physical properties and drained monotonic response of the intact Pleistocene specimens retrieved from the CREC site were summarized in this study. An investigation of the undrained monotonic or cyclic response would provide a more complete characterization of the influence of diagenesis on the deformation behavior of natural sands. A microscopy study could also be conducted to characterize the microfabric of CREC sand and to better understand the possible mechanisms of diagenesis-dilatancy.

2. Results presented in this study indicate that in situ peak friction angles are generally well predicted by empirical relationships based on high strain cone tip resistance and overpredicted by empirical relationships based on small strain shear wave velocity. The influence of diagenesis on empirical predictions of peak friction angle should be further investigated.

3. The general form and parameters of the age dilatancy and MEVR dilatancy models presented in this study should be further validated using the data presented in this study and the data compiled by Bolton (1986). The models could also be extended to sands with fines, non silica sands, and/or locked sands in a future study.

4. Based on a conclusion that confining pressure significantly influences peak strength due to diagenesis, it is possible that liquefaction resistance due to diagenesis is similarly affected. A future investigation involving the influence of confining pressure on diagenesis correction factors for liquefaction resistance is needed.
5. Based on a conclusion of this study that fines content significantly influences MEVR, a comprehensive investigation on the relationship between fines content and diagenesis is warranted.

6. The difference between VR-time relationships based on laboratory cases and based on field cases was only partially explained by fines content. Possible reasons for the remaining difference were summarized in Chapter 6 and could be further investigated in the future.

7. The in situ liquefaction test conducted with a mobile field shaker at the CREC site was modeled as a two-dimensional problem with full Rayleigh damping applied to the entire domain. These modeling choices are reasonable if the majority of strains occurred in plane during the experiment, and the applied damping is characteristic for the level of anticipated strain during the simulation. Maximum excess pore pressures and shear strain amplitudes predicted in this study will be compared to the experimental in situ test results when they become available. Refinements to the model will be considered if needed to better predict the results of in situ testing.
APPENDICES
APPENDIX A

PROCEDURES AND RESULTS OF ADDITIONAL TESTS PERFORMED ON CREC SAND
A.1 Introduction

Several quantitative and qualitative tests were conducted on samples of intact and remolded CREC sands to better characterize its density, mineralogy, and peak strength. These tests include specific gravity, initial void ratio, maximum and minimum index void ratio, x-ray diffraction, and angle of repose. Procedures and results of the tests are summarized in the following sections.

A.2 Specific gravity

The specific gravity of soil solid particles ($G_s$) was estimated from three 100 g samples of CREC sand trimmings (trimmings from all frozen core samples throughout deposit) according to ASTM D854 using a calibrated volumetric flask. The procedure for performing the test consists of: carefully measuring the volume of one or more volumetric flasks; adding known quantities of dry soil and distilled water to the flask; boiling the mixture to remove entrapped air from solution; allowing the mixture to cool; and measuring the weight and temperature of the cooled mixture to calculate $G_s$. The value of $G_s$ determined for the three samples were 2.67, 2.68, and 2.69, for an average value of 2.68. The average $G_s$ value is close to a value of 2.65, which is commonly assumed for quartz sand.

A.3 Initial void ratio

The frozen weights ($W$), and volumes ($V$) of the cylindrical intact specimens (A1, A2…D2) were recorded before they were placed in the triaxial test chamber. The specimens were carefully retrieved from the triaxial cell after shearing, dried in an oven.
for at least 16 hours, and weighed to determine their dry weights ($W_s$). From $W$, $W_s$, $V$ and a $G_s$ value of 2.68 the initial void ratios of the frozen specimens were obtained with following equation assuming they were fully saturated,

$$e_0 = w \frac{G_{ice}}{G_s}$$

(A.1)

where $w$ is moisture content [$(W - W_s)/W_s$] and $G_{ice}$ is the specific gravity of ice. A value of 0.917 was used for $G_{ice}$. Measurements of $W$, $W_s$, $V$ and calculated $w$ and $e$ values are provided in table A.1.

**Table A.1:** Weight, volume, moisture content, and initial void ratios of frozen specimens.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Depth (m)</th>
<th>Frozen Weight, $W_s$ (g)</th>
<th>Dry Weight, $W$ (g)</th>
<th>Volume, $V$ (cm$^3$)</th>
<th>Moisture Content, $w$ (%)</th>
<th>Initial void ratio, $e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>2.21</td>
<td>1025.8</td>
<td>798.9</td>
<td>537.1</td>
<td>0.28</td>
<td>0.83</td>
</tr>
<tr>
<td>A2</td>
<td>3.00</td>
<td>1053.8</td>
<td>838.6</td>
<td>555.9</td>
<td>0.26</td>
<td>0.75</td>
</tr>
<tr>
<td>B1</td>
<td>2.04</td>
<td>971.4</td>
<td>773</td>
<td>507.4</td>
<td>0.26</td>
<td>0.75</td>
</tr>
<tr>
<td>B2</td>
<td>2.53</td>
<td>1096.7</td>
<td>849.6</td>
<td>600.4</td>
<td>0.29</td>
<td>0.85</td>
</tr>
<tr>
<td>C1</td>
<td>2.44</td>
<td>973.2</td>
<td>730.7</td>
<td>540.4</td>
<td>0.33</td>
<td>0.97</td>
</tr>
<tr>
<td>C2</td>
<td>2.64</td>
<td>1055.3</td>
<td>826.3</td>
<td>541.3</td>
<td>0.28</td>
<td>0.81</td>
</tr>
<tr>
<td>C3</td>
<td>2.85</td>
<td>922.7</td>
<td>677.1</td>
<td>512.8</td>
<td>0.36</td>
<td>1.06</td>
</tr>
<tr>
<td>C4</td>
<td>3.25</td>
<td>1026.4</td>
<td>758.9</td>
<td>569.5</td>
<td>0.35</td>
<td>1.03</td>
</tr>
<tr>
<td>C5</td>
<td>3.57</td>
<td>989.2</td>
<td>768.4</td>
<td>536.2</td>
<td>0.29</td>
<td>0.84</td>
</tr>
<tr>
<td>D1</td>
<td>2.22</td>
<td>1068.0</td>
<td>845.3</td>
<td>563.7</td>
<td>0.26</td>
<td>0.77</td>
</tr>
<tr>
<td>D2</td>
<td>2.76</td>
<td>923.8</td>
<td>681.3</td>
<td>524.7</td>
<td>0.355</td>
<td>1.04</td>
</tr>
</tbody>
</table>
A.4 Maximum and minimum index void ratio

Values of the maximum void ratio \(e_{\text{max}}\) were estimated from the minimum index density of three oven-dried 500 g samples of CREC sand from specimens B1, C4, and C5 according to ASTM D4254 by inverting a graduated cylinder. The procedure for performing the test consists of filling a graduated cylinder with the sample, quickly inverting the cylinder, and measuring the volume occupied by the loosely compacted soil. For each sample, three trials were performed to ensure that consistent values had been measured. The resulting \(e_{\text{max}}\) values were 1.14, 1.15, and 1.16, for an average value of 1.15.

Values of the minimum void ratio \(e_{\text{min}}\) were estimated from the maximum index density in accordance with ASTM D4253. Four 500 g samples of CREC sand (trimmings and leftover soil from all frozen core samples throughout deposit) were vibrated on a vertically oscillating shaking table in a 15.2-cm (6-in) diameter compaction mold under an applied surcharge of 13.8 kPa (2.0 psi) for a duration of ten minutes before measuring the final volume occupied by the sample. Trials were conducted on wet and dry samples and it was found that dry samples were denser after the vibration period. The values of \(e_{\text{min}}\) estimated from four dry samples were 0.67, 0.68, 0.69, and 0.69, for an average value of 0.68.

A.5 X-Ray diffraction

A qualitative investigation of the mineral composition of the CREC sand was conducted by performing x-ray diffraction (XRD), as discussed in Chapter 3. The XRD
test is performed by radiating x-rays at a soil sample at different incident angles and measuring the intensity of diffracted waves. Intensities measured at different angles form a unique pattern that is compared with the patterns of common mineral types to determine the predominant mineral composition of the soil.

Fig. A.1 presents the XRD patterns of 300 g samples taken from specimens C1 and C4. The signatures of crystalline quartz and plagioclase feldspar are shown below the records for C1 and C4. It is seen that the pattern is identical to the signature pattern for quartz and possesses some similarities to the pattern for Plagioclase feldspar.

**Figure A.1:** X-Ray diffraction patterns for CREC sand samples retrieved from depths of 2.4 and 3.4 m. X-Ray diffraction signatures for crystalline quartz and plagioclase feldspar are shown below the CREC pattern.
A.6 Angle of repose

The static angle of repose of the CREC sand was measured by depositing dry sand through a funnel onto a glass plate, forming a conical heap. The funnel was placed immediately above the surface of the cone without touching its apex, such that the fall height was not great enough to affect the sloped surface. A photograph of the conical heap of CREC sand is shown in Fig. A.2. When the cone reached a height of about 5 cm (2 in), any further deposited soil slid down the sides of the cone, and the angle of the slope was about 32-24°. The test was repeated several times and an average value of about 33° was confirmed.

Figure A.2: Photograph depicting conical heap of CREC sand formed by slow deposition through a funnel. The approximate slope angle is 32-34°.
APPENDIX B

RESULTS OF DRAINED TRIAXIAL COMPRESSION TESTS ON OTTAWA SAND
B.1 Introduction

This appendix presents the results of 17 consolidated drained (CD) triaxial compression tests performed with Ottawa sand conducted to obtain familiarity with the procedures for the CD test and to ensure that good results were obtained with the recently acquired Clemson University soil mechanics laboratory triaxial testing equipment. Proficiency with the triaxial testing equipment and procedures was especially important because intact specimens used in this research are an expensive and limited resource.

B.2 Methodology

Tests were conducted on dry Ottawa sand specimens in a standard triaxial test chamber (triaxial cell volume of 1042 cm$^3$). Specimens with lengths and diameters of 71 mm and 36 mm, respectively, were prepared by tamping in five layers to obtain a “dense” state and poured slowly through a funnel to obtain a “loose” state. The difference in “dense” and “loose” states is not great between specimens, however typically the specimens prepared by pouring had a void ratio of around 0.55 and the specimens prepared by tamping had a void ratio of around 0.50. Eight dry specimens were sheared at a rate of 1%/min to 20% axial strain at confining pressures of 34.5, 68.9, 135, and 275 kPa.

Another series of tests was performed on saturated Ottawa sand specimens, which were fabricated by pouring and by tamping in the same manner as the dry Ottawa sand specimens. After filling the triaxial chamber and applying an isotropic confining pressure of 34.5 kPa, deaired water was percolated through the specimens under very low
pressures to saturate the specimen and back pressure was applied with a net effective stress of 34.5 kPa. The back pressure and cell pressure were increased in stages until the specimens were fully saturated (B-value > 0.95) while maintaining a net effective stress of 34.5 kPa. Seven saturated specimens were consolidated for 60 minutes at confining pressures of 68.9, 135, and 275 kPa and sheared at a rate of 1%/min to 20 percent axial strain.

Three additional tests were performed on saturated Ottawa sand specimens with lengths and diameters of around 142 mm and 71 mm, respectively, in a larger triaxial cell (triaxial cell volume of 2725 cm$^3$), which would later be used to test intact frozen specimens. The three specimens were prepared in loose states by slowly pouring dry sand through a funnel. The specimens were saturated under a net effective stress of 34.5 kPa, consolidated for 60 minutes at an effective confining pressure of 68.9 kPa, and sheared at a rate of 1%/min to 20 percent axial strain.

B.3 Results

Presented in Table B.1 are test data and results for the Ottawa sand specimens. Initial void ratios ($e_0$) were computed assuming a typical value of the specific gravity of solids ($G_s$) for quartz sand of 2.65. The peak secant friction angle is calculated from the principal stresses at peak stress difference ($\sigma_1 - \sigma_3$) assuming no cohesion intercept (i.e., $c' = 0$).

Often, seating errors were observed when preparing the stress-strain curves for the Ottawa sand specimens. During shearing, seating was automated by the triaxial testing
software, which slowly brought the load piston down until registering a load of 3.5 lbs. In some cases, the automated seating did not bring the piston into full contact with the top cap and contact was made during the shearing stage after a certain amount of piston travel. In these cases, the axial strain values calculated from the change in height from the beginning of the test did not correspond to the true start of shearing. Furthermore, initial contact between the piston and top cap produced a series of stress-strain points that had to be omitted from the true stress-strain response. Figure B.1 presents an example illustrating the procedure for correcting the axial strain and initial stress-strain points to develop a new origin corresponding to the beginning of shearing. Noting the difficulty in seating during the series of tests performed on Ottawa sand, subsequent tests performed on CREC sand (Chapter 3) were seated manually.
Table B.1: Test details and results of consolidated drained triaxial tests conducted on Ottawa sand specimens.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Test type</th>
<th>Preparation method</th>
<th>Triaxial chamber size (cm³)</th>
<th>Initial void ratio, ( \varepsilon_0 )</th>
<th>Effective confining pressure, ( \sigma'_c ) (kPa)</th>
<th>Peak secant friction angle, ( \phi' ) (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O1</td>
<td>Dry</td>
<td>Pouring</td>
<td>1042</td>
<td>0.532</td>
<td>34.5</td>
<td>39.6</td>
</tr>
<tr>
<td>O2</td>
<td>Dry</td>
<td>Pouring</td>
<td>1042</td>
<td>0.522</td>
<td>68.9</td>
<td>38.9</td>
</tr>
<tr>
<td>O3</td>
<td>Dry</td>
<td>Pouring</td>
<td>1042</td>
<td>0.550</td>
<td>138</td>
<td>34.2</td>
</tr>
<tr>
<td>O4</td>
<td>Dry</td>
<td>Pouring</td>
<td>1042</td>
<td>0.615</td>
<td>275</td>
<td>34.0</td>
</tr>
<tr>
<td>O5</td>
<td>Dry</td>
<td>Tamping</td>
<td>1042</td>
<td>0.501</td>
<td>34.5</td>
<td>37.2</td>
</tr>
<tr>
<td>O6</td>
<td>Dry</td>
<td>Tamping</td>
<td>1042</td>
<td>0.496</td>
<td>68.9</td>
<td>38.9</td>
</tr>
<tr>
<td>O7</td>
<td>Dry</td>
<td>Tamping</td>
<td>1042</td>
<td>0.507</td>
<td>138</td>
<td>37.8</td>
</tr>
<tr>
<td>O8</td>
<td>Dry</td>
<td>Tamping</td>
<td>1042</td>
<td>0.504</td>
<td>275</td>
<td>36.8</td>
</tr>
<tr>
<td>O9</td>
<td>Sat</td>
<td>Pouring</td>
<td>1042</td>
<td>0.552</td>
<td>68.9</td>
<td>37.2</td>
</tr>
<tr>
<td>O10</td>
<td>Sat</td>
<td>Pouring</td>
<td>1042</td>
<td>0.514</td>
<td>138</td>
<td>37.8</td>
</tr>
<tr>
<td>O11</td>
<td>Sat</td>
<td>Pouring</td>
<td>1042</td>
<td>0.541</td>
<td>275</td>
<td>36.3</td>
</tr>
<tr>
<td>O12</td>
<td>Sat</td>
<td>Tamping</td>
<td>1042</td>
<td>0.499</td>
<td>68.9</td>
<td>40.2</td>
</tr>
<tr>
<td>O13</td>
<td>Sat</td>
<td>Tamping</td>
<td>1042</td>
<td>0.516</td>
<td>138</td>
<td>37.5</td>
</tr>
<tr>
<td>O14</td>
<td>Sat</td>
<td>Tamping</td>
<td>1042</td>
<td>0.516</td>
<td>275</td>
<td>36.9</td>
</tr>
<tr>
<td>O15</td>
<td>Sat</td>
<td>Pouring</td>
<td>2725</td>
<td>0.660</td>
<td>68.9</td>
<td>32.1</td>
</tr>
<tr>
<td>O16</td>
<td>Sat</td>
<td>Pouring</td>
<td>2725</td>
<td>0.690</td>
<td>68.9</td>
<td>30.0</td>
</tr>
<tr>
<td>O17</td>
<td>Sat</td>
<td>Pouring</td>
<td>2725</td>
<td>0.630</td>
<td>68.9</td>
<td>33.2</td>
</tr>
</tbody>
</table>
Figure B.1: Typical uncorrected and corrected stress-strain relationships for Ottawa sand specimens after adjusting for seating errors (test specimen O14).
The stress-strain relationships for both poured and tamped dry Ottawa sand are presented in Figure B.2. The stress-strain relationships for both poured and tamped saturated Ottawa sand are presented in Figure B.3. Typical behavior of sands is observed in Figures B.2 and B.3, with peak strengths reached at 4-12% axial strain and post peak softening at large strain. For specimens loaded under the same initial confining pressure, the residual strengths of specimens with different initial void ratios tend to converge as the specimens’ strength approach the critical state shearing resistance. Generally, there is a trend of increasing stiffness with increasing effective confining pressure. There is also a trend of increased stiffness and peak strength and decreased axial strain at peak strength with initial void ratio for specimens loaded under the same effective confining pressure. One instance in Figure B.2 depicts greater stiffness for a specimen consolidated under 68.9 kPa (O2) than a specimen consolidated under 138 kPa (O3). Typically, sands prepared with similar void ratios exhibit increasing stiffness with confining pressure. The reason for the low stiffness of specimen O3 is likely related to the higher initial void ratio compared to specimen O2, and difficulties resolving the true stress and strain values because of seating errors. At large strain values, more consistency was achieved in the test results.

The stress-strain relationships of Ottawa sand specimens tested in the large triaxial cell and loaded under an isotropic confining pressure of 68.9 kPa are presented in Figure B.4. There is no noticeable difference between the results presented in Figure B.4 and the results presented in Figures B.2 and B.3. The influence of initial void ratio on the peak strength and axial strain at peak strength is also apparent in Figure B.4.
Besides occasional differences between the anticipated and observed stiffnesses, the stress-strain characteristics of Ottawa sand are typical of remolded sands. Confirmation of these characteristics indicates that the tests were performed with sufficient care and consistency to test intact samples.

Values of the drained peak friction angle are plotted versus void ratio in Figure B.5 along with previously published values obtained by Salgado et al. (2000) for saturated Ottawa sand. The peak friction angles of Ottawa sand determined in this study follow the trend determined by Salgado et al. (2000) reasonably well. The values of friction angle vary between 41 and 30 degrees. Using both sets of data, a linear relationship provides a strong fit with the experimental values. The consistency between the values obtained at Clemson University and the values observed by Salgado et al. (2000) provide support that the triaxial tests were performed properly.
Figure B.2: Stress-strain relationships of dry Ottawa sand under effective confining stresses of 34.5, 68.9, 138, and 275 kPa.
Figure B.3: Stress-strain relationships of saturated Ottawa sand under effective confining stresses of 68.9, 138, and 275 kPa.
Figure B.4: Stress-strain relationships of saturated Ottawa sand in a large triaxial chamber at an effective confining stress of 68.9.
Figure B.5: Variation in peak friction angle with initial void ratio for Ottawa sand specimens determined in this study and reported by Salgado et al. (2000).
B.4 Conclusions

The stress-strain behavior and peak strengths of Ottawa sand were found to be typical of medium to dense sand, characterized by increasing stiffness and peak strength with decreasing initial void ratio and increasing stiffness and peak strength with effective confining pressure. The variation in peak friction angle with initial void ratio of Ottawa sand determined in this study and determined by Salgado et al. (2000) were found to be in good agreement.

Two observations regarding the triaxial testing system were made. Firstly, it was difficult to correctly measure stresses and strains at small to medium strain levels ($\varepsilon_a < 1\%$). Secondly, an optional, automatic seating procedure was found to complicate initial seating of the loading piston in the top specimen cap. A manual procedure was used for subsequent tests reported in Chapter 3 of this study.
APPENDIX C

RESULTS OF DRAINED TRIAXIAL COMPRESSION TESTS ON CREC SAND
Figure C.1: Relationships of (a) stress ratio-axial strain and (b) volumetric strain-axial strain for intact and remolded specimens under an effective confining pressure of 17 kPa.
Figure C.2: Relationships of (a) stress ratio-axial strain and (b) volumetric strain-axial strain for intact and remolded specimens under an effective confining pressure of 35 kPa.
Figure C.3: Relationships of (a) stress ratio-axial strain and (b) volumetric strain-axial strain for intact and remolded specimens under an effective confining pressure of 69 kPa.
Figure C.4: Relationships of (a) stress ratio-axial strain and (b) volumetric strain-axial strain for intact and remolded specimens under an effective confining pressure of 138 kPa.
Code used in FLAC 7.0 (Itasca 2011) for mobile field shaker simulations

;Mobile field shaker test simulation
config dyn gw cppud ex 9
set flow off

;Important grid pts
def $gridpts
    $jzhalf = 40       ;j of z = 0.5
    $jh1 = 9          ;middle of layer D
    $jh2 = 17         ;bottom of layer C
    $jh3 = 23         ;bottom of layer B2
    $jh4 = 29         ;bottom of layer B1
    $jh5 = 33         ;bottom of unsaturated zone, z = 2
    $jh6 = 34         ;j of $jh7 minus 1
    $jh7 = 35         ;bottom of layer A
    $jh8 = 36         ;j of $jh7 plus 1
    $ipl = 137        ;left plate boundary
    $ipr = 146        ;right plate boundary
    $ic = 141         ;plate centerline
    $jc = 41          ;plate centerline
    $ist1 = 138       ;plate element gridpoints
    $ist2 = 139
    $ist3 = 140
    $ist4 = 142
    $ist5 = 143
    $ist6 = 144
    $ist7 = 145
    $isl = 129        ;left stiff soil boundary
    $isr = 153        ;right stiff soil boundary
    $isnsl1 = 139     ;sensor locations
    $isnsr1 = 142
    $isnsr2 = 143
    $isnsr3 = 144
    $jsnsb1 = 26
    $jsnsb2 = 27
    $jsnsb3 = 28
    $jsnsm = 29
    $jsnst1 = 30
    $jsnst2 = 31
    $jsnst3 = 32
    $jl2 = 28         ;Boundaries
    $jl3 = 16
    $it1 = 92
    $it2 = 62
    $ir = 282
end
$gridpts
; Grid generation
  g 281 40
  gen 0 0 0 10 70.25 10 70.25 0 i 1 $ir j 1 $jc

; Material models
  model mohr

; Properties
  ; Cooper marl
  prop dens 1320 poros 0.51 perm 2e-11
  prop cohesion 1e4 friction 43.0 dilation 9.0 tension 0.0 j 1 $jh2
  prop shear 375e6 bulk 811e6 j 1 $jh2

; Layer C
  prop dens 1340 poros 0.51 perm 2e-11 shear 47e6 bulk 100e6 j $jh2 $jh3
  prop cohesion 1e3 friction 40.0 dilation 10.0 tension 0.0 j $jh2 $jh3

; Layer B2
  prop dens 1440 poros 0.48 perm 2e-9 shear 53e6 bulk 115e6 j $jh3 $jh4
  prop cohesion 1e3 friction 44.0 dilation 15.0 tension 0.0 j $jh3 $jh4

; Layer B1
  prop dens 1470 poros 0.47 perm 2e-9 shear 76e6 bulk 165e6 j $jh4 $jh7
  prop cohesion 1e3 friction 46.0 dilation 18.0 tension 0.0 j $jh4 $jh7

; Layer A
  prop dens 1410 poros 0.47 perm 2e-9 shear 49e6 bulk 106e6 j $jh7 $jc
  prop cohesion 1e3 friction 43.0 dilation 14.0 tension 0.0 j $jh7 $jc
  prop cohesion 1e5 friction 43.0 dilation 14.0 tension 1e5 i $isl $isr j $jzhalf $jc

; Water
  water bulk 0.0 dens 1000 tens 1.0e10

; Boundary and Initial conditions
  fix x i 1
  fix x i $ir
  fix x y j 1

  ini sat 0.625 j $jh8 $jc
  ini sat 1 j 1 $jh7

  ini syy -1.7e4 var 0 0.9e4 j $jh8 $jc
  ini syy -2.6e4 var 0 2.8e4 j $jh7 $jh8
  ini syy -5.4e4 var 0 2.7e4 j $jh4 $jh7
  ini syy -8.1e4 var 0 2.7e4 j $jh3 $jh4
  ini syy -10.8e4 var 0 2.7e4 j $jh2 $jh3
  ini syy -18.0e4 var 0 7.2e4 j 1 $jh2

  ini sxx -3.4e4 var 0 3.4e4 j $jh8 $jc
ini sxx -2.6e4 var 0 -0.8e4 j $jh7 $jh8
ini sxx -3.8e4 var 0 1.2e4 j $jh4 $jh7
ini sxx -6.0e4 var 0 2.2e4 j $jh3 $jh4
ini sxx -8.2e4 var 0 2.2e4 j $jh2 $jh3
ini sxx -37.4e4 var 0 29.2e4 j 1 $jh2

ini szz -3.4e4 var 0 3.4e4 j $jh8 $jc
ini szz -2.6e4 var 0 -0.8e4 j $jh7 $jh8
ini szz -3.8e4 var 0 1.2e4 j $jh4 $jh7
ini szz -6.0e4 var 0 2.2e4 j $jh3 $jh4
ini szz -8.2e4 var 0 2.2e4 j $jh2 $jh3
ini szz -37.4e4 var 0 29.2e4 j 1 $jh2

ini pp 8.3e4 var 0 -8.3e4 j 1 $jh7
fix sat j $jc
fix pp j $jc

; Gravity loading
set dyn off
set gravity 9.81
solve

; Add structural beam element
struct prop 1 e 1 i 1 a 1 den 1e-3
struct beam beg grid $ipl $jc end grid $ist1 $jc
struct beam beg grid $ist1 $jc end grid $ist2 $jc
struct beam beg grid $ist2 $jc end grid $ist3 $jc
struct beam beg grid $ist3 $jc end grid $ic $jc
struct beam beg grid $ic $jc end grid $ist4 $jc
struct beam beg grid $ist4 $jc end grid $ist5 $jc
struct beam beg grid $ist5 $jc end grid $ist6 $jc
struct beam beg grid $ist6 $jc end grid $ist7 $jc
struct beam beg grid $ist7 $jc end grid $ipr $jc

def ggg
loop nn (1,10)
command
struct node nn fix r
end_command
nn1 = nn - 1
if nn > 1 then
command
struct node nn slave x nn1
struct node nn slave y nn1
end_command
end_if
end_loop
end
ggg

;Plate loading

def ramp
ramp = min(1.0,float(step)/2000.0)
end
apply syy -3.8e4 hist ramp from $ipl $jc to $ipr $jc

solve

model dll pm4sand i 61 221 j 23 34
prop dens 1375 poros 0.48 perm 2e-9 i 61 221 j 23 29
prop D_r 0.62 G_o 960 h_po 22 n_b 0.62 i 61 221 j 23 29
prop dens 1430 poros 0.47 perm 2e-9 i 61 221 j 29 34
prop D_r 0.70 G_o 1510 h_po 5.5 n_b 0.56 i 61 221 j 29 34

step 1000
step 2000

;Effective vertical stresses and initial pore pressures
print esyy i $isnsl1 $isnsr1 j $jsnsb1 $jsnst1
print pp i $isnsl1 $isnsr2 j $jsnsb1 $jsnst2

;Dynamic loading
;Water properties

ini fmodulus 2e8 j $jh5 $jh7
ini fmodulus 0.0 j $jh7 $jc
ini fmodulus 2e9 j 1 $jh5
water dens 1000 tens 1.0e10

set flow on

solve

;Fish functions
;ex_2 = max gamma
;ex_7 = max ru
;ex_9 = max CSR

call mon_ex.fis
mon_ex
call getExcesspp.fis
call getcsr.fis
set nsample=50 nstep=1
$savemp
$getExcesspp
$getcsr

;Histories
def dummy ; ... count number of history points
count = count + 1
end

hist dytime

;Plate
hist xdisp i $ipl j $jc
hist ydisp i $ipl j $jc
hist xdisp i $ic j $jc
hist ydisp i $ic j $jc
hist xdisp i $ipr j $jc
hist ydisp i $ipr j $jc
hist xaccel i $ic j $jc

;Sensor array

;strain histories
def strain_hist
array arr1(4)
array arr2(4)
array arr3(4)
array arr4(4)
array arr5(4)
array arr6(4)
array arr7(4)
array arr8(4)
array arr9(4)
array arr10(4)
array arr11(4)
array arr12(4)
array arr13(4)
array arr14(4)
array arr15(4)

while_stepping
  dum1 = fsr($isnsl2,$jsnsb1,arr1)
dum2 = fsr($ic,$jsnsb1,arr2)
dum3 = fsr($isnsr1,$jsnsb1,arr3)
dum4 = fsr($isnsl2,$jsnsb2,arr4)
dum5 = fsr($ic,$jsnsb2,arr5)
dum6 = fsr($isnsr1,$jsnsb2,arr6)
dum7 = fsr($isnsl2,$jsnsb3,arr7)
dum8 = fsr($ic,$jsnsb3,arr8)
dum9 = fsr($isnsr1,$jsnsb3,arr9)
dum10 = fsr($isnsl2,$jsnsm,arr7)
dum11 = fsr($ic,$jsnsm,arr8)
dum12 = fsr($isnsr1,$jsnsm,arr9)
dum13 = fsr($isnsl2,$jsnst1,arr10)
dum14 = fsr($ic,$jsnst1,arr11)
dum15 = fsr($isnsr1,$jsnst1,arr12)
dum16 = fsr($isnsl2,$jsnst2,arr13)
dum17 = fsr($ic,$jsnst2,arr14)
dum18 = fsr($isnsr1,$jsnst2,arr15)

str_1=str_1+2.0*arr1(4)
str_2=str_2+2.0*arr2(4)
str_3=str_3+2.0*arr3(4)
str_4=str_4+2.0*arr4(4)
str_5=str_5+2.0*arr5(4)
str_6=str_6+2.0*arr6(4)
str_7=str_7+2.0*arr7(4)
str_8=str_8+2.0*arr8(4)
str_9=str_9+2.0*arr9(4)
str_10=str_10+2.0*arr10(4)
str_11=str_11+2.0*arr11(4)
str_12=str_12+2.0*arr12(4)
str_13=str_13+2.0*arr13(4)
str_14=str_14+2.0*arr14(4)
str_15=str_15+2.0*arr15(4)
str_16=str_16+2.0*arr16(4)
str_17=str_17+2.0*arr17(4)
str_18=str_18+2.0*arr18(4)

end
strain_hist

hist str_1
hist str_2
hist str_3
hist str_4
hist str_5
hist str_6
hist str_7
hist str_8
hist str_9
hist str_10
hist str_11
hist str_12
hist str_13
hist str_14
hist str_15
hist str_16
hist str_17
hist str_18

;pore pressures
hist pp i $isns12 j $jsnsb1
hist pp i $ic j $jsnsb1
hist pp i $isnsr1 j $jsnsb1
hist pp i $isns12 j $jsnsb2
hist pp i $ic j $jsnsb2
hist pp i $isnsr1 j $jsnsb2
shear stresses

; shear stresses
hist vsxy i $isnsr1 j $jsnsb1
hist vsxy i $ic j $jsnsb1
hist vsxy i $isnsr1 j $jsnsb1
hist vsxy i $isnsr2 j $jsnsb1
hist vsxy i $ic j $jsnsb1
hist vsxy i $isnsr1 j $jsnsb1
hist vsxy i $isnsr2 j $jsnsb1
hist vsxy i $ic j $jsnsb1
hist vsxy i $isnsr1 j $jsnsb2
hist vsxy i $isnsr2 j $jsnsb2
hist vsxy i $ic j $jsnsb2
hist vsxy i $isnsr1 j $jsnsb2
hist vsxy i $isnsr2 j $jsnsb2
hist vsxy i $ic j $jsnsb2
hist vsxy i $isnsr1 j $jsnsb3
hist vsxy i $ic j $jsnsb3
hist vsxy i $isnsr1 j $jsnsb3
hist vsxy i $isnsr2 j $jsnsm
hist vsxy i $ic j $jsnsm
hist vsxy i $isnsr1 j $jsnsm
hist vsxy i $isnsr2 j $jsnsm
hist vsxy i $ic j $jsnsm

; displacement

; displacement
hist ydisp i $isnsr1 j $jsnsb1
hist ydisp i $ic j $jsnsb1
hist ydisp i $isnsr1 j $jsnsb1
hist ydisp i $isnsr2 j $jsnsb1
hist ydisp i $ic j $jsnsb1
hist ydisp i $isnsr1 j $jsnsb1
hist ydisp i $isnsr2 j $jsnsb1
hist ydisp i $ic j $jsnsb1
hist ydisp i $isnsr1 j $jsnst1
hist ydisp i $ic j $jsnst1
hist ydisp i $isnsr1 j $jsnst1
hist ydisp i $isnsr2 j $jsnst1
hist ydisp i $ic j $jsnst1
hist ydisp i $isnsr1 j $jsnst2
hist ydisp i $ic j $jsnst2
hist ydisp i $isnsr1 j $jsnst2
hist ydisp i $isnsr2 j $jsnst2
hist ydisp i $ic j $jsnst2

hist xdisp i $isnsl2 j $jsnsb1
hist xdisp i $ic j $jsnsb1
hist xdisp i $isnsl2 j $jsnsb1
hist xdisp i $ic j $jsnsb1
hist xdisp i $isnsl2 j $jsnsb1
hist xdisp i $ic j $jsnsb1

hist xdisp i $isnsr1 j $jsnsb1
hist xdisp i $isnsr2 j $jsnsb1
hist xdisp i $isnsr2 j $jsnsb1
hist xdisp i $ic j $jsnsm
hist xdisp i $isnsr1 j $jsnsm
hist xdisp i $isnsr2 j $jsnsm
hist xdisp i $isnsl2 j $jsnst3
hist xdisp i $ic j $jsnst3
hist xdisp i $isnsr1 j $jsnst3
hist xdisp i $isnsr2 j $jsnst3

; Left boundary

hist ydis i 1 j 1
hist ydis i 1 j $jl2
hist ydis i 1 j $jl3
hist ydis i 1 j $jc
hist ydis i 1 j $jsnsb2
hist ydis i 1 j $jsnsm
hist ydis i 1 j $jsnst1

hist xdis i 1 j 1
hist xdis i 1 j $jl2
hist xdis i 1 j $jl3
hist xdis i 1 j $jc
hist xdis i 1 j $jsnsb2
hist xdis i 1 j $jsnsm
hist xdis i 1 j $jsnst1

; Bottom

hist ydis i $it1 j 1
hist ydis i $it2 j 1
hist xdis i $it1 j 1
hist xdis i $it2 j 1
hist xaccel i $it1 j 1
hist xaccel i $it2 j 1

; Top

hist ydis i $it1 j $jc
hist ydis i $it2 j $jc
hist xdis i $it1 j $jc
hist xdis i $it2 j $jc
hist xaccel i $it1 j $jc
hist xaccel i $it2 j $jc

; Right
hist ydis i $ir j 1
hist ydis i $ir j $jl2
hist ydis i $ir j $jl3
hist ydis i $ir j $jc
hist ydis i $ir j $jsnsb2
hist ydis i $ir j $jsnsm
hist ydis i $ir j $jsnst1

hist xdis i $ir j 1
hist xdis i $ir j $jl2
hist xdis i $ir j $jl3
hist xdis i $ir j $jc
hist xdis i $ir j $jsnsb2
hist xdis i $ir j $jsnsm
hist xdis i $ir j $jsnst1

hist dummy

;Dynamic setup
set dyn on
set large
set dytime = 0.0
initial xdisp = 0.0 ydisp = 0.0
ini dy_damp rayleigh 0.01 10

;Boundary conditions
app xquiet yquiet i=1
app xquiet yquiet i=$ir
app xquiet yquiet j=1

;Dynamic Loading
def s_wave
if dytime > 20.0
s_wave = 0.0
else
s_wave = sin(2.0*pi*10*dytime)
endif
end

apply sxy 2.5e4 hist=s_wave i $ipl $ipr j $jc ;

set step 1000000000
solve dytime 25.0

;Fish functions
;mon_ex.fis stores the maximum shear strain as a grid variable
def _ini_ex
loop i (1,izones)
  loop j (1,jzones)
    if model(i,j) # 1
      ex_1(i,j)=0.
def

ex_2(i,j) = 0.
endif
endloop
endloop
end

def mon_ex
array arr(4)
while_stepping
loop i (1,izones)
  loop j (1,jzones)
    if model(i,j) # 1
      dum = fsr(i,j,arr)
      ex_1(i,j) = ex_1(i,j) + 2.0 * arr(4)
      ex_2(i,j) = max(ex_2(i,j),abs(ex_1(i,j)))
    endif
  endloop
endloop
end

;getpp.fis stores the maximum pore pressure ratio as a grid variable
def $savepp
loop i(1,izones)
  loop j(1,jzones)
    ex_3(i,j) = pp(i,j)
    ex_4(i,j) = -(syy(i,j) + pp(i,j))
    ex_7(i,j) = 0.0
  end_loop
end_loop
end

def $getExcesspp
whilestepping
  if nstep = nsample then
    loop i(1,izones)
      loop j(1,jzones)
        if pp(i,j) > ex_3(i,j) then
          ex_6(i,j) = pp(i,j) - ex_3(i,j)
        else
          ex_6(i,j) = 0.0
        endif
        ex_5(i,j) = abs(ex_6(i,j)/ex_4(i,j))
        ex_7(i,j) = max(ex_7(i,j),(ex_5(i,j)))
      end_loop
    end_loop
    nstep = 1
  endif
  nstep = nstep + 1
end
;getcsr.fis stores the maximum cyclic shear stress ratio as a
gridpoint variable

def $getcsr
while stepping
  if nstep=nsample then
    loop i(1,izones)
      loop j(1,jzones)
        ex_8(i,j) = abs(sxy(i,j)/ex_4(i,j))
        ex_9(i,j) = max(ex_8(i,j),ex_9(i,j))
      end_loop
    end_loop
    nstep = 1
  endif
  nstep = nstep + 1
end


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