

Performance of Columnar Reinforced Ground during Seismic Excitation

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ACADEMIC ABSTRACT

Deep soil mixing to construct stiff columns is one of the methods used today to improve performance of loose ground and remediate liquefaction problems. This research adopts a numerical approach to study seismic performance of soil-cement columnar reinforcements in loose sandy profiles. Different constitutive models were investigated in order to find a model that can properly predict soil behavior during seismic excitations. These models included NorSand, Dafalias-Manzari, Plasticity Model for Sands (PM4Sand) and Pressure-Dependent-Multi-Yield-02 (PDMY02) model. They were employed to predict behavior of soils with different relative densities and under different confining pressures during monotonic and cyclic loading. PDMY02 was identified as the most suitable model to represent soil seismic behavior for the system studied herein.

The numerical aspects of the finite element approach were investigated to minimize the unintended numerical miscalculations. The focus was put on convergence tolerance, solver time-step, constraint definition, and, integration, material and Rayleigh damping. This resulted in forming a robust numerical configuration for 3-D nonlinear models that were later used for studying behavior of the reinforced grounds.

Nonlinear finite element models were developed to capture the seismic response of columnar reinforced ground during dynamic centrifuge testing. The models were calibrated with results from tests with unreinforced profiles. Thereafter, they were implemented to predict the response of two reinforced profiles during seismic excitations with different intensities and liquefaction triggering. Model predictions were compared with recordings and the possible effects from the reinforcements were discussed. Finally, parametric studies were performed to further evaluate the efficiency of the reinforcements with different extension depths and area replacement ratios.

The results collectively showed that the stiff elements, if constructed appropriately, can withstand seismic excitations with different intensities, and provide a firm base for overlying structures. However, the presence of the stiff elements within the loose ground resulted in stronger seismic intensities on the soil surface. The columns were not able to considerably reduce pore water pressure generation, nor prevent liquefaction triggering. The reinforced profiles, comparing to the free-field profiles, had larger settlements on the soil surface but smaller settlements on the columns. The results concluded that utilization of the columnar reinforcements requires great attention as these reinforcements may result in larger seismic intensities at the ground surface, while not considerably reducing the ground deformations.

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GENERAL ABSTRACT

The mitigation of seismic damage potential of soft soil sites remains one of the leading challenges in geotechnical earthquake engineering. It is well-established that structures located on these sites generally experience more damage due to excessive ground deformation during earthquakes. Ground reinforcements are often required to improve these sites for support of overlying structures. A remediating approach is to construct stiff columns within these sites by mechanically mixing soil with cementitious materials. Cemented soil has higher strength, and thereby, undergoes less deformations. Moreover, stiff columns can provide resistance against movement of their surrounding soil providing a firm base for possible above foundations.

The primary focus of this research is to evaluate the effect of stiff column reinforcements on seismic behavior of loose ground. For this purpose, a numerical model was developed for the reinforced ground, and it was validated with results from experiments. The model was then used to study the performance of the reinforced ground during earthquake excitations with different intensities. The observed behavior was discussed and compared with findings from previous studies in literature. Finally, the numerical model was employed to evaluate efficiency of the reinforcements with different extension depths and occupied area.

The results collectively showed that stiff columns, if constructed appropriately, can withstand different shaking levels, and provide a firm support for overlying structures. However, they were not efficient in reducing deformation of the surrounding soils. The presence of the stiff elements within the loose ground resulted in stronger seismic intensities on the soil surface. The study concluded that utilization of stiff columns requires great attention and understanding of the reinforcing mechanism. These columns might increase seismic intensity below foundations, while not considerably reducing the ground deformations.

Dedication

To Mom and Dad.
The ocean and the mountain of my life.

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Chapter 1

Introduction

1.1 Background

The mitigation of seismic damage potential of soft soil sites remains one of the leading challenges in geotechnical earthquake engineering. These sites have limited bearing capacity and undergo extensive deformations during seismic excitations. Ground reinforcements are often required to improve these profiles for support of overlying structures. Deep Soil Mixing (DSM) is one of the approaches used today to remediate the performance of soft ground during earthquakes.

Deep Soil Mixing is a ground improvement technique whereby soil is mechanically mixed with cementitious materials to make columns of uniform diameter (Mitchell 2008). If necessary, these columns can be overlapped to make grids of rigid walls. Cemented soil material generally has a higher strength, lower permeability and lower compressibility than the native soil, although total unit weight may be less (Bruce 2000). Constructing soil-cement columns is relatively fast, with minimal ground vibration and noise pollution. Material strength can be conveniently calibrated by varying the ratio of solidifying material. The method gained reliability after new advancements in mixing equipment, alignment control and integrated systems for real-time monitoring during installation. Moreover, it does not require any particular material such as gravels

to be imported to the site especially in areas with remote access (Porbaha 1998). All these and many other advantageous have made DSM an attractive method particularly in urban constructions on soft soil profiles.

DSM reinforcements can improve the performance of a site by (1) reducing the shear strain imposed on the enclosed soil, thereby slowing the generation of excess Pore Water Pressure (PWP); (2) providing resistance against lateral deformations or vertical settlements even if liquefaction is triggered in the enclosed soils; (3) providing a barrier against high excess PWP migrating from surrounding unimproved soils to the area enclosed by the reinforcements (Nguyen et al. 2013).

Case histories from Kobe (1995) and Tohoku (2011) earthquakes have provided evidence on effectiveness of DSM reinforcements for seismic damage mitigation (Tokimatsu et al. 1996, and Tokunaga et al. 2015). For example, during the Kobe (1995) earthquake, the 14-story Oriental Hotel, founded on piles protected by DSM grid, was able to survive the earthquake without damage to its pile foundation or evidence of liquefaction within the grid zone, despite large liquefaction-induced deformation of surrounding untreated quay walls. In similar investigations, Tokunaga et al. (2015) evaluated the performance of 789 structures supported by different DSM configurations (column, block, grid and wall types), and found that the soil-cement reinforced facilities experienced superficial damage during and after the Tohoku (2011) earthquake.

Over the last five decades, an extensive amount of research has been contributed to different aspects of DSM approach (e.g. Okumura and Terashi 1975, Adalier et al. 1998, and Kitazume and Terashi 2013). Physical model testing has provided a better understanding of the effect of reinforcement properties on ground settlements and different modes of failure. (e.g. Almeida et al. 1985, Babasaki et al. 1991 and Takahashi et al. 2006). Numerical modeling has further extended these studies to more severe ground conditions and stronger earthquake intensities (e.g. O'Rourke and Goh 1997, and Namikawa et al. 2007). These have resulted in better understanding of reinforcing mechanism and effectiveness of DSM reinforcement for remediating against liquefaction.

It is often assumed that reinforcements and surrounding soils have shear strain compatible deformations (Baez and Martin 1992, O'Rourke and Goh 1997, and Durgunoglu 2006). If pure shear beam behavior was occurring, the shear stresses in stiff reinforcement relative to the soil

mass would be in proportion to the reinforcement-to-soil stiffness ratio. In other words, if the reinforcement is 100 times stiffer than the surrounding soil, it will absorb 100 times more shear stress than surrounding soil during pure shear behavior. Therefore, based on this assumption, reinforcements can play an important role in reducing seismic intensity and preventing liquefaction triggering.

Yet, theoretical analyses (e.g. Goughner and Pestana 1998) and numerical simulations (e.g. Olgun and Martin 2008, Green et al. 2008, Rayamajhi et al. 2012 and 2014) suggest that reinforcements deform in combination of both shear and flexure during seismic loading, such that, they carry substantially smaller amounts of shear stress. Therefore, they might not be beneficial as expected from previous studies in reducing seismic intensity. Adding to this, Bradley et al. (2013) reported increase in seismic intensity at surface of a grid reinforced ground, which would result in higher inertial loads for overlying structures.

Accordingly, there is a significant need to investigate the performance of DSM reinforcements during seismic excitations. Rayamajhi et al. (2014) performed centrifuge testing with free-field and reinforced profiles to observe the effect of reinforcements on seismic behavior of loose grounds. This research builds on these experiments with a numerical approach. The goal was to develop and validate well-structured finite element models to capture and expand the results obtained from the experiments.

1.2 Experimental Studies

Rayamajhi et al. (2014 and 2015) performed four dynamic centrifuge tests to experimentally observe the effect of soil-cement column reinforcements on seismic performance of loose sandy profiles. These tests were used in this research for calibration and validation of the numerical models. Two of these tests had free-field unreinforced profiles, and the other two were reinforced with soil-cement columns. The tests were performed under 50g centrifugal acceleration and model prototype dimensions were 20 m $\times$ 10 m $\times$ 10 m (length, width, height).

A summary of test configurations is shown in Table 1.1. The models consisted of soil profile with 2 m dense Monterey sand ($D_r \approx 60\%$) underlain by 6 m loose Nevada sand ($D_r \approx 35\%$). Underneath the Nevada sand, Model 1 (free-field) and Model 2 (reinforced) had 2 m very stiff

cemented base, whereas Model 3 (free-field) and Model 4 (reinforced) had 2 m very dense Monterey sand ($D_r \approx 90\%$). Soil was saturated 1.5 m below ground surface with methyl-cellulose and de-ionized water. The fluid was 30 times more viscous than water to more properly represent pore fluid diffusion and dynamic response. The model was built in a hinge plate container developed by Fiegel et al. (1994) with negligible friction for lateral movement to represent an infinite soil profile. Chi-Chi (1999) earthquake motion recording at station TCU078E was scaled to different amplitudes from 0.02g to 0.74g and applied to the base of the model in the longitudinal direction. In between each earthquake excitation, a small amplitude sine-sweep was applied to the model to test the elastic response of site profile and evaluate effects of earthquake shakings. Instrumentations included accelerometers and PWP transducers throughout the profile with settlement transducers on surface.

Reinforcements were in form of 3×6 columns extended from surface to bottom of Nevada sand layer. Columns had 1.75 m diameter with 2.8 m spacing in rectangular grid setting, giving an area replacement ratio of approximately 30%. Soil-cement columns were constructed to achieve 1 MPa unconfined compressive strength in seven days. Average shear modulus of the columns were estimated as 174 MPa.

1.3 Numerical Studies

In this research, nonlinear numerical models were developed and calibrated with results from the free-field centrifuge tests. Thereafter, the calibrated models were used to predict response of the reinforced centrifuge tests during seismic excitations with varying intensities and liquefaction triggering. The accuracy of simulations was evaluated and effect of the reinforcements was discussed. Parametric studies were then performed to further evaluate effect of reinforcements with different geometries and area replacement ratios. The objectives and tasks to reach these goals included:

Objective 1: Evaluate capability of advanced constitutive models in predicting soil behavior during static and dynamic loading.

In geotechnical problems, the constitutive model that represents soil behavior plays a key role in the accuracy of numerical simulations. Different constitutive models with complex

frameworks and multiple sets of input parameters have been developed to more realistically predict soil behavior. Therefore, there was a need to evaluate these constitutive models and find the most suitable one for the targeted study herein. For this purpose, we investigated the framework and capability of four advanced constitutive models in predicting soil behavior. The models included: NorSand by Jefferies and Been (2006), DM04 by Dafalias and Manzari (2004), Plasticity Model for Sands (PM4Sand) by Boulanger and Ziotopoulou (2012), and Pressure-Dependent-Multi-Yield-02 (PDMY02) by Yang and Elgamal (2008). These models were implemented to predict behavior of soils with different relative densities and under different confining pressures during monotonic and cyclic loading. Model simulations were compared, and PDMY02 was identified as the most suitable model to be implemented in the numerical simulations of the reinforced grounds.

Objective 2: Develop and calibrate a numerical model with results from the experiments.

The core proposal of this research recommends OpenSees finite element program for all the required numerical modeling. OpenSees is an open-source program with limited tools for pre- and post-processing stages. Moreover, more than 20 parameters were needed to be calibrated before performing, long run-time, 3-D nonlinear simulations for the reinforced profiles. Also, it was recognized that numerical aspects of finite element modeling can considerably affect the targeted study herein. Therefore, there was a need to comprehensively study these aspects and ensure least numerical errors in the planned simulations.

In this study, multiple tools were developed for pre- and post-processing stages of the simulations. We investigated the effect from numerical aspects of finite element approach, and an optimized solution system with least numerical errors was developed for nonlinear simulations. Model parameters were also calibrated with results from the free-field centrifuge tests. The parameters that resulted the closest predictions to the experiments were selected to be used in the numerical model for the reinforced profile.

Objective 3: Evaluate effect of stiff column reinforcements on seismic performance of loose sandy profiles.

Nonlinear 3-D finite element models were developed for the columnar reinforced centrifuge tests. The numerical models were employed to capture the experiments during earthquake excitations with different intensities. Simulations were compared with recordings

for PWP generation and liquefaction triggering, surface settlements and seismic wave propagation. Both numerical and experimental results were discussed thoroughly and compared with findings in literature from previous studies. Moreover, the behaviors of unreinforced and reinforced profiles were compared to evaluate the efficiency of columnar reinforcements in mitigating seismic hazards.

Objective 4: Perform parametric study on columnar reinforcements.

Due to laboratory inherent difficulties, only two centrifuge tests were performed with reinforced profiles. These tests had area replacement ratios of $\sim 30\%$ with reinforcement extending all the way through the loose soil layer. To expand these studies, we employed the validated numerical models and evaluated the effect of columnar reinforcements in a 20 m loose sandy profile. The reinforcements were extended to 5 m, 10 m and 20 m depths with approximate area replacement ratios of 6%, 9%, 13%, 20% and 35%. The simulations were performed under base excitations from 113 different earthquakes with magnitudes ranging from $M_w = 5.5$ to 9.0 and maximum accelerations of 0.05g to 0.5g. In total, 1808 simulations were performed.

1.4 Organization

This dissertation is organized as a collection of five manuscripts with an introductory chapter and a closing chapter. Chapter 2 presents our study of two advanced constitutive models: NorSand and DM04. The framework of these models and the features they include to predict soil behavior are described. Thereafter, the models are calibrated with results from literature, and employed in prediction of soil behavior with different relative densities and under different confining pressures. For this purpose, 26 different triaxial tests on Erksak and Toyoura sands are simulated during drained and undrained conditions. The predictions are compared with experiments for stress-strain and volumetric behaviors at 2%, 5% and 10% axial strains, and at peak and critical states. Finally, dispersion analyses are performed to identify which model provides less scattered results from the experiments.

Chapter 3 presents our study of PM4Sand constitutive model during cyclic loading. The framework of the model and the hypotheses it builds on are explained. Thereafter, capability of

the model in predicting soil cyclic behavior is evaluated during experiments with different relative densities, under different confining pressures and with different pre-existing shear stresses. For this purpose, simulations have been performed for shake table tests by De Alba et al. (1976), direct simple shear tests by Wijewickreme et al. (2005) and Norwegian Geotechnical Institute (1977), cyclic triaxial tests by Seed and Lee (1965) and cyclic torsional shear tests by Ishihara (1996). The simulations are compared with the experiments for uniform number of cycles to liquefaction with different triggering criteria; and the accuracy of the model is discussed.

Chapter 4 presents our study of numerical aspects of finite element modeling. The focus was put on some of the most influential aspects from a geotechnical perspective. These aspects included convergence tolerance, solver time-step, constraint definition, and, integration, material and Rayleigh damping. Finite element simulations with different numerical settings are compared with analytical solutions, and the effect from these settings are discussed. The chapter describes substantial, often adverse, effect from improper numerical settings, and provides guidance on proper assignment of these settings. Thereafter, the capability of a well-structured finite element model is evaluated in predicting the seismic behavior of a loose ground. For this purpose, following the suggested numerical guidelines, a systematic approach is taken to calibrate a nonlinear numerical model. The model is then implemented to predict the response of a free-field dynamic centrifuge test. The simulations are compared with the recordings for generated PWP, surface settlements and seismic wave propagation throughout soil profile, and accuracy of the predictions is discussed.

Chapter 5 presents our numerical simulations for the centrifuge model tests 1 and 2, shown in Table 1.1, with the cemented base. PDMY02 is used as the core constitutive model to represent nonlinear behavior of the loose sand. The model parameters are calibrated with results from the free-field centrifuge test, and they are implemented in a 3-D finite element model to capture soil response in the reinforced profile. The predicted and recorded behaviors are compared, and the differences are discussed.

Chapter 6 presents our numerical simulations for the centrifuge model tests 3 and 4, shown in Table 1.1, with the sandy base. This chapter, in more details comparing to Chapter 5, explains the calibration procedure, accuracy of the model in predicting seismic performance of the unreinforced profile, model development for the reinforced profile, and accuracy of the model in predicting

seismic performance of the reinforced profile. The chapter also compares both simulations and experiments with previous findings in literature. Moreover, the observed behaviors in the reinforced and free-field profiles are compared, and possible effect from reinforcements on PWP generation, surface settlement and seismic intensity are explained.

Chapter 7 provides a summary of this research and presents the key conclusions reached in the study.

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1.6 Tables

Table 1.1: Summary of centrifuge tests performed by Rayamajhi et al. (2014).

Test Model	Soil Profile	Description	Base Configuration
1	2 m dense Monterey sand, underlain by 6 m loose Nevada sand, underlain by 2 m cemented base	Free-Field	Fixed end with cemented base
2		Reinforced	
3	2 m dense Monterey sand, underlain by 6 m loose Nevada sand, underlain by 2 m very dense Monterey sand	Free-Field	Free end with sandy base
4		Reinforced	

Chapter 2

Evaluation of Constitutive Models in Predicting Soil Behavior during Monotonic Loading: NorSand vs. Dafalias-Manzari

The authors of the following manuscript intend to submit it to the International Journal of Numerical and Analytical Methods in Geomechanics.

Soheil Kamalzare made the following contributions to this manuscript:

- Developed two programs for NorSand and DM04 models.
- Calibrated the numerical models with results from literature.
- Performed the comparative studies between the numerical models and the experiments.

Dr. Olgun made the following contributions to this manuscript:

- Supervised the study, and provided valuable feedback.
- Suggested the underlying ideas in the comparative studies.
- Provided guidelines on the calibration procedure of the DM04 model.

Evaluation of Constitutive Models in Predicting Soil Behavior during Monotonic Loading: NorSand vs. Dafalias-Manzari

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2.1 Abstract

Predicting soil strength and deformability continues to be one of the significant challenges in geotechnical engineering. Over the years, constitutive models with different features have been developed to more realistically represent soil behavior in numerical platforms. In this paper, we have evaluated capabilities of NorSand and Dafalias-Manzari models in predicting soil behavior during monotonic loading. These models build on critical state soil mechanics and are developed to predict behavior of sands under different loading conditions. NorSand utilizes a framework similar to Cam Clay model with further modifications to more realistically predict sand volumetric behavior. Dafalias-Manzari model incorporates a two-surface plasticity framework, and more importantly, considers the effects of fabric change and void ratio evolutions during loading. We implemented these models to simulate Toyoura and Erksak sands stress-strain and volumetric behaviors in a comprehensive set of triaxial tests in drained and undrained conditions. The soils in these tests had different relative densities and were under a range of confining pressures. Our results showed that both these models are able to predict the overall soil behavior fairly well. Dafalias-Manzari model predicted better values for deviatoric stress at peak, and 2%, 5% and 10% axial strains, whereas, NorSand predicted better values at critical state. Dafalias-Manzari model also predicted closer values for volumetric behavior. NorSand, missing the response in small strains, provided better volumetric simulations in larger strains. Moreover, in detailed regression analyses for both stress-strain and volumetric behaviors showed that both model predictions closely fit values measured in experiments.

2.2 Introduction

Predicting soil strength and deformability depends substantially on accurate representation of the relation between loads (or stresses) and deformations (or strains). In numerical computations, constitutive models describe the relation between stresses and strains. These models must be able to simulate soil behavior realistically by addressing main aspects of soil behavior. These aspects can be identified as: bulk and shear stiffness of soil and their dependency on confining pressure and void ratio; rapid reduction in stiffness of overconsolidated soils when they reach their pre-consolidation stress; change in stiffness of the soil when loading direction changes; volumetric behavior of soil and its dependency on confining pressure and void ratio; reduction in shear stiffness as soil is sheared due to an increase in deviatoric stress and strain; failure criteria that soil state reaches eventually if sheared enough; dependency on particle size, fabric and soil structure; and, tendency to behave in an anisotropic manner in most soils (Potts and Zdravkovic 1999).

Over the years, much experimental research has been contributed to understanding soil complex behavior. Rendulic (1933) performed a comprehensive set of triaxial tests to investigate the relationship between void ratio and the three principal effective stresses for saturated clays. He obtained a unique failure line later identified as the Critical State Line (CSL). Terzaghi (1936) introduced principal of effective stress to explain all measurable effects of a change of stress, such as compression, distortion and change of shearing resistance. Casagrande (1936) identified a critical density at which continuous deformation takes place at constant shear stress. Hvorslev (1937) studied different types of consolidation histories in a shear box and tried to isolate the influence of water content (or void ratio) on failure. He defined a state boundary surface for reconstituted OC clays. Skempton (1985) studied soil behavior after large shearing displacements and introduced residual strength as the minimum drained strength attained at slow rates of shearing. Burland (1990) noted that the strength of intact stiff clays at low to intermediate confining pressures fall very rapidly from peak to a well-defined plateau which he called the post-rupture strength. In lights of these researches constitutive models have been developed and continuously modified to better predict soil behavior.

Naturally initial constitutive models were relatively simple and built on elastic theories (e.g. elastic hyperbolic model by Duncan and Chang 1970, and small strain stiffness model by Jardine et al. 1986). These models are based on the assumption that principal incremental stress and strain

directions coincide, and may predict soil behavior with reasonable accuracy for stress states not reaching failure. However, phenomena such as irreversibility of a portion of the strains, stress-path dependency, coupling effects such as volume changes due to shear stresses, referred to as shear-dilatancy, rotation of principal stress axes, and most behavior patterns near and beyond failure cannot be handled by elasticity theory (Lade 2005).

Later, elasto-plastic models were introduced. These models are based on the assumption that the principal directions of accumulated stress and incremental plastic strain coincide. Elasto-plastic models incorporate a yield surface to distinct purely elastic behavior from elasto-plastic behavior, and a plastic potential surface which defines the direction of plastic straining. Further, three different plastic behaviors were identified: perfectly plastic and strain (work) hardening and softening plasticities. For perfectly plastic behavior, yield surface remains the same during shearing while for strain hardening and softening, yield surface expands gaining strength (hardening) or contracts losing strength (softening) during shearing. Conventional Mohr-Coulomb, Tresca, Von Mises and Drucker and Prager (1952) models incorporate elastic perfectly plastic yield surfaces with different shapes in the deviatoric plane. These models fail to adequately model many basic aspects of soil behavior mentioned above, and in general over-predict soil volumetric behavior especially for overconsolidated clays and dense sands.

In 1950's further developments lead to more comprehensive numerical frameworks. Hill (1950) and Hodge (1955) proposed the so-called isotropic hardening rule, according to which the yield surface expands uniformly. Ishlinskii (1954) and Prager (1956) introduced kinematic hardening with the idea that the yield surface translates without rotation in the direction of the strain increment. Drucker et al. (1957) suggested the existence of a cap yield surface controlled by volume change. Roscoe et al. (1958) proposed the critical state framework and the existence of a state boundary surface. Calladine (1963) suggested the theory of hardening plasticity as a basis for a consistent formulation of the models.

The first critical state models were the series of Cam Clay models by Roscoe and Schofield (1963) and Schofield and Wroth (1968). The models were later modified by Roscoe and Burland (1968). Critical State Models (CSMs) are based on isotropic elastic strain hardening/softening plasticity theory. Conventionally, the consolidation and shearing behavior of soils were considered

separately. CSMs were able to simulate both aspects of soil behavior providing significant advancement in soil modelling over the simpler Tresca and Mohr-Coulomb models.

Many modifications to the basic frameworks have been made in the literature. These include improved shapes for the yield and plastic potential surfaces (e.g. Di Maggio and Sandler 1971, Zienkiewicz and Naylor 1973, and Ohata and Wroth 1976), modification in elastic components of the model to obtain proper patterns of displacement distribution (e.g. Houlsby 1985, and Jardine and Potts 1988), representing plastic behavior inside the main yield surface by introducing multiple nested yield surfaces (Mroz 1967, Prevost 1985, and Elgamal et al. 2003), incorporation of bounding surface (Dafalias and Popov 1975, and Dafalias and Herrmann 1982) and phase transformation surface (Manzari and Dafalias 1997), representing elastic behavior during unloading by incorporating kinematic yield surfaces, which move within the outer bounding surface (e.g. Al-Tabbaa 1987, and Al-Tabbaa and Wood 1989), and representing soil fabric change within the framework (Dafalias and Manzari 2004, and Boulanger and Ziotopoulou 2013). Over the years researches have utilized different frameworks to develop soil constitutive models. A comprehensive set of these models including some critical state based models is evaluated and compared by Lade (2005).

NorSand and Dafalias-Manzari models are two critical state constitutive models for sandy soils. Multiple features, explained subsequently, have been incorporated in these models to distinguish volumetric behavior of soils with different relative densities. The objective of this paper is to investigate the capability of these models in predicting soil behavior with different relative densities and under a range of confining pressures in drained and undrained conditions. For this purpose, we implemented these models to simulate soil behavior in comprehensive set of triaxial tests on Erksak and Toyoura sands. The focus was put on soil stress-strain and volumetric behaviors. Our investigations showed that both models are capable of predicting general soil behavior fairly accurate, and with proper calibration they can be implemented in numerical domains to represent soil behavior. It should be noted that in this paper, we have only investigated monotonic loading. In other attempts, we have investigated capability of constitutive models during cyclic loading (Kamalzare and Olgun 2014a, 2014b and 2015).

2.3 NorSand Model

NorSand was developed by Jefferies (1993), and was later modified by Jefferies and Been (2006). It is a plasticity model applicable to uncemented sands, and the first model to adjust the predicted response with the distance soil state accommodates from critical state line. This was attained by incorporating the state parameter of Been and Jefferies (1985) in formulation of dilatancy and hardening aspects of the model.

NorSand utilizes a framework similar to Cam Clay model, but with controversy of assuming infinite possible yield surface in p' - e space such that any yield surface does not necessarily intersect the critical state line (Shuttle and Jefferies 2010). In Cam Clay, it is assumed that there is only one normal compression line (NCL), which is located above the critical state line and moves parallel to it. However, a large number of triaxial tests carried out by Ishihara, Tatsuoka, and Yasuda (1976), field data from hydraulic filling of offshore sandfill islands by Stewart et al. (1983) and detailed experiments by Jefferies and Been (2000) provided evidence on infinite number of NCL lines for sands, although only one locus exists for an element of sand at any instant. The infinite number of NCLs means that there are an infinite number of yield surfaces because any NCL can be viewed as the trace of a set of yield surfaces as they harden (Jefferies and Been 2006).

2.3.1 NorSand Yield Surface

NorSand utilizes a single stationary yield surface, shown in Figure 2.1, similar to the classical Cam Clay model. Soil shows elastic behavior inside the yield surface and elasto-plastic behavior on it. The model imposes a limit on yield surface formation to prevent unrealistic amount of dilation in dense soils.

2.3.2 NorSand Elasto-Plastic Law

NorSand adopts an isotropic elasticity with constant shear modulus and Poisson's ratio. The model utilizes an associated flow rule with normality law that defines plastic strains perpendicular to the stresses on the yield surface. Moreover, it defines an image state, shown in Figure 2.1, where no plastic volumetric strain occurs. Soil at stress ratios (q/p') less than image state shows contractive behavior (compression in drained tests and positive pore water pressure generation in

undrained tests) and at stress ratios larger than image state shows dilative behavior (dilation in drained tests and negative pore water pressure generation in undrained tests).

2.3.3 NorSand Hardening/Softening Law

NorSand hardening law evolves the yield surface with plastic strains. The model adjusts the size of the yield surface in proportion to the mean effective stress at the image state. For loose soils, the effective stress at image state is larger than the effective stress at intersection of yield surface with CSL (the ultimate point soil state moves to). This requires the yield surface to soften (shrink) to reach the critical state. While for dense soils, the reverse applies and yield surface needs to harden (expand) to reach the critical state.

NorSand requires 11 different parameters to represent constitutive soil behavior. Three of them (mean effective stress, soil state and shear modulus) are specific for each test simulation. The remaining parameters are all pre-defined through analyzing test results and calibration of the model. These parameters and the way to find them are explained in detail by Jefferies and Been (2006).

2.4 Dafalias-Manzari Model

Dafalias-Manzari model was developed by Manzari and Dafalias (1997), and was later modified by Dafalias and Manzari (2004). The model (DM04) is based on the constitutive behavior proposed by Y. F. Dafalias and Popov (1975) and Dafalias (1986). DM04 is a stress-ratio controlled, critical state compatible, sand plasticity model. It assumes, only changes of stress ratio can cause the necessary relative shearing and rolling of sand grains which are macroscopically modeled as plastic shear and volume change. The model is made fully compatible with critical state soil mechanics principles by rendering the slopes of the phase transformation and the bounding lines as functions of the state parameter.

The novel aspect of DM04 which can be considered as a significant advancement over other constitutive models is its consideration of fabric change during dilatant phase of plastic deformation. Evolution of fabric change during unloading and reloading cycles is an important soil behavior phenomenon. It can be critical to capture void ratio redistribution within a soil mass

to better evaluate the kinematics of possible failure mechanisms in earth structures. Observations made by many researchers, like Nemat-Nasser (1980), have confirmed that a significant change in the contact-normal orientation distribution of fabric happens during the dilative phase of plastic deformation of soils. This fabric change drastically affects the volume change in the contractive phase of the following reverse loading. Dafalias and Manzari have used a parameter in their model that develops in the dilative phase of shearing and affects the volume change during the contractive phase of the next opposite loading. This capability of the model affects the soil behavior during cyclic loads. However, the focus in this study was put on monotonic loading, thus it has no effect on the simulations performed here. We have thoroughly discussed the fabric evolution mechanism of DM04 in Kamalzare and Olgun (2014a and 2014b).

2.4.1 DM04 Yield Surface

DM04 utilizes a kinematic stress ratio defined yield surface. The yield surface has a wedge shape in the p' - q space and is shown in Figure 2.2. Only elastic strains develop for stress ratios inside the wedge shown by the shaded area. The wedge changes orientation and size for the stress ratio to remain on the yield surface what can be called kinematic (i.e. translates in stress space) and isotropic (i.e. expands and contracts in all directions) hardening.

2.4.2 DM04 Elasto-Plastic Law

DM04 uses a constant Poisson's ratio and a shear modulus suggested by Richart et al. (1970) and Li and Dafalias (2000) which continuously evolves with changes in mean effective stress and void ratio. The model adopts a dilatancy surface that represents the phase transformation surface, shown in Figure 2.2, introduced by Ishihara et al. (1975). Soil at states below this surface shows a contractive behavior, and at states above this surface shows a dilative behavior. For dense soils, the phase transformation surface will be below the critical state surface. Therefore, a dense soil if sheared, will show a contractive behavior until reaching this surface, then a dilative behavior up to failure. For loose soils, this surface will be on or very close to the critical state surface. Therefore, a loose soil, if sheared, will completely or mostly show contractive behavior.

2.4.3 DM04 Hardening/Softening Law

DM04 hardening parameters are defined such that prevent physically impossible unlimited change in volume, and to more properly simulate the strength degradation in dense sands. The model includes a bounding surface, shown in Figure 2.2, that defines the boundary between plastic hardening and softening in soil. Soils, if sheared, show a plastic hardening behavior (expansion of the yield surface – gaining strength) till they reach the bounding surface. Making a peak on it, they continue shearing showing a plastic softening behavior (shrinkage of the yield surface – losing strength) till failure. For dense soils, the bounding surface is always located above the critical state surface. Therefore, dense soils show a more pronounced peak strength during shearing. While for loose soils, the bounding surface is located on or very close to the critical state surface. Therefore, loose soils do not show peak strength before failure.

DM04 maintains compatibility with critical state soil mechanics by adjusting the slope of dilatancy and bounding lines with the state parameter of Been and Jefferies (1985); such that, as the soil state moves toward the critical state line, the dilatancy and bounding lines converge with the critical state line like a closing pair of scissors.

DM04 requires 16 different parameters to represent constitutive soil behavior. Like NorSand, three of these parameters (mean effective stress, soil state and shear modulus) are specific for each test simulation. The remaining parameters are all pre-defined through analyzing tests results and calibration of the model. These parameters and the way to find them are explained in detail by Dafalias and Manzari (2004).

2.5 Implementation and Calibration of the Models

We developed a MATLAB script for each model, and confirmed the scripts by performing simulations in comparison to the ones conducted by Jefferies and Been (2009) and Dafalias and Manzari (2004). Our comparative studies for NorSand and DM04 with soils sheared in drained triaxial compression tests is shown in Figure 2.3. Our scripts were able to replicate the values found by these researchers closely in different relative densities and confinements.

Thereafter, we simulated 8 triaxial tests with Toyoura sand and 9 triaxial tests with Erksak sand in drained and undrained conditions for soils with different relative densities and under a

range of confining pressures. All these tests were performed by Golder Associates Company between the years 1986 to 1988. The test details are shown in Table 2.1. We were not able to calculate soil relative density in these tests. Because using the values of maximum, minimum and actual void ratios provided in the test reports would result in unrealistically low and sometimes negative relative densities. Therefore, in Table 2.1, we have presented initial state parameter for each test which were also provided in the lab reports. Positive values of state parameter show loose of critical state and negative values show dense of critical state. Maximum and minimum void ratios do not take part in calibration and implementation of these models.

In our simulations for Toyoura Sand, we used the parameters suggested by Ghafghazi and Shuttle (2008) for NorSand, and the parameters suggested by Yannis F. Dafalias and Manzari (2004) for DM04. For Erksak sand, we used the parameters suggested by Jefferies and Been (2006) for NorSand, while we calibrated DM04 by analyzing the results of 26 other triaxial tests on Erksak sand. All the parameters used in this study are shown in Tables 2.2 and 2.3.

2.6 Results and Discussion

Model simulations of soil behavior in two of the triaxial tests on Toyoura and Erksak sands in drained and undrained conditions are shown in Figure 2.4. It can be seen that both models were able to realistically predict the general curvature and trend of soil behavior. In simulations for Toyoura sand, NorSand and DM04 predicted peak deviatoric stress with 15% and 20% errors respectively. They predicted better values for pore water pressure with less than 10% error. In simulations for Erksak sand, DM04 was able to predict closer values for deviatoric stress and volumetric strain. NorSand predicted volumetric strain properly at axial strains less than 5%, but thereafter, the differences sharply increased.

To better evaluate capabilities of these models, we compared the results in two different attempts. First, the focus was put on the stresses predicted. Accordingly, the values of deviatoric stress at peak and critical state, and at 2%, 5% and 10% axial strains were compared. Thereafter, the focus was put on volumetric behavior predicted. Thereby, the values of volumetric strain (for drained tests) and pore water pressure (for undrained tests) were compared at the mentioned axial strains.

2.6.1 Evaluation of Predicted Stresses

We compared simulations for deviatoric stress at peak and critical state with the ones measured in the experiments regardless of the strains they were achieved. For the peak stress comparison, we calculated the relative difference (error) between simulated and measured values. For the critical state comparison, we calculated the absolute differences since the values were small, and only a small variation could cause a large relative difference. The results are shown in the Table 2.4. The recorded data for test LIQ-G602 showed unrealistic values for peak stress, and therefore, we did not consider them in our evaluations.

It must be noted that, for simulations of Toyoura sand, the models were calibrated by different people on different series of tests. Still, both models predicted peak values with less than 13% relative difference from the experiments. This shows great capability of models in predicting peak stress values. Because in practice, often there is limited access to detailed laboratory results and consequently default calibrated parameters are used in numerical models.

For Erksak sand, the test results used for calibration and simulation, being different from each other, were from a certain series of experiments. In this case, DM04 predicted peak values with nearly 7% error, and NorSand surprisingly predicted peak values with more than 20% error. In total, over 17 triaxial tests which were modeled, an average of 10% error was found in predictions of DM04 for peak deviatoric stress, whereas this value for NorSand was about 16%.

Figure 2.5 shows the simulation errors for peak deviatoric stress in each test. It can be seen that, the amount of error in undrained simulations are in average greater than the amount of error in drained simulations. This implies that models have better accuracy in drained rather than undrained conditions for predicting peak deviatoric stress. These results also signify the role of calibration. As it was mentioned before, we calibrated DM04 with large amount of experimental results for Erksak sand. Most probably, this has considerably attributed in reducing the errors in predictions. Moreover, it should be noted that when models had the same calibration base (i.e. in Erksak sand), DM04 predicted values with considerably lower error than NorSand.

NorSand predicted closer values to the experiments for deviatoric stress at critical state. Its simulations for Toyoura sand had in average 37 kPa absolute difference with experiments, while this difference for DM04 was in average 93 kPa. For Erksak sand, NorSand and DM04 predicted values in average 66 kPa and 85 kPa different from experiments respectively.

Figure 2.6 demonstrates the comparison of predicted and measured deviatoric stresses at 2%, 5% and 10% axial strains for the test LIQ-G402. Our results for all the simulated tests are shown in Table 2.5. In average, DM04 predicted better values at all three axial strains. DM04 predicted values with 11% to 19% errors at engineering concerned 2% and 5% axial strains, while NorSand model always had more than 19% error.

Finally, we performed a regression analysis to evaluate the dispersal of the predicted values relative to the measured ones. For this purpose, we calculated Coefficient of Determination (R^2) for simulations with respect to experiments. This coefficient indicates how well the data points fit a line or curve (Draper and Smith 1998). In our study, it provides a measure of how well observed behaviors are simulated by the models. The value of R^2 varies between 0 and 1, with larger numbers indicating a better fit and 1 representing a perfect fit. Data dispersal is shown in Figure 2.7, and R^2 values are shown in Table 2.6. As it is seen in Table 2.6, the regression values for both models, except for NorSand at 2% axial strain, are more than 0.93 which indicates a very good fit between predicted and measured values.

2.6.2 Evaluation of Predicted Volumetric Behavior

We compared predicted and measured values of excess pore water pressure (PWP) in undrained tests and volumetric strain in drained tests. For undrained tests, we calculated the relative difference, whereas for drained tests, as the strains were very small and only a small variation would return a large relative difference, we calculated the absolute difference between the predicted and the measured values. Our results are shown in Table 2.7. DM04 predicted excess PWP fairly accurate in all the mentioned axial strains. NorSand, missing the behavior in the beginning of the tests (2% axial strain), predicted closer values at 5% and 10% axial strains. Both models predicted volumetric strain relatively accurate at 2% axial strain. The deviation increased with further loading, but still not being considerably different from the measured values.

We also performed a regression analysis for volumetric behavior to evaluate the dispersal of the predicted values relative to the measured ones. Data dispersal and R^2 values are shown in Figure 2.8 and Table 2.8. The R^2 values for volumetric behavior were smaller than the values found in the stress analyses. This indicates that the models have better accuracy in prediction of stresses than volumetric behavior. In general, DM04 predicted better values for volumetric strain in drained tests, while NorSand predicted better values for excess PWP in undrained tests.

2.7 Conclusions

In this paper, we evaluated capabilities of NorSand and Dafalias-Manzari models in predicting soil behavior under monotonic loading. We simulated Toyoura and Erksak sands behavior in a series of different triaxial compression tests in drained and undrained conditions. The tests were performed on soils with different relative densities and under a range of confining pressures. We investigated the accuracy of the models in predicting the stress-strain and volumetric behavior of soil.

Our results showed that both these models were able to capture general trend and curvature of the soil behavior. DM04 was able to predict slightly better values for deviatoric stress at peak and 2%, 5% and 10% axial strains, while NorSand was able to predict better values at critical state. Dispersion analysis showed that DM04 predictions were in general slightly less scattered from experiments comparing to NorSand predictions.

DM04 was able to predict the volumetric behavior in both drained and undrained conditions with small difference from the measured values. NorSand was also able to predict the volumetric behavior accurately in drained tests. In undrained tests, NorSand had considerable error in simulating the PWP at 2% axial strain, but it was able to predict PWP better at 5% and 10% axial strains. In general, for volumetric behavior, DM04 predicted less scattered values in drained tests and NorSand predicted less scattered values in undrained tests. Dispersion analyses also showed that simulations were more scattered for volumetric behavior than stress-strain behavior.

Results in this study collectively showed that both models are able to predict soil behavior fairly well, while DM04 predicts slightly better values than NorSand. Proper calibration can elevate model predictions and significantly reduce the errors in simulations. The calibrated models can provide realistic estimates of soil strength and deformability and can be used within numerical domains to represent soil behavior during different loading conditions.

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2.10 Tables

Table 2.1: Monotonic triaxial tests numerically modeled in this study

Soil	Test Type	Test ID	p' (kPa)	ψ_0
Toyoura Sand	Drained	CID-G460	200	-0.233
		CID-G461	220	-0.165
		CID-G462	270	-0.061
		CID-G463	200	-0.034
	Undrained	LIQ-G401	300	-0.054
		LIQ-G402	339	0.007
		LIQ-G403	300	0.044
		LIQ-G405	400	0.030
Erksak Sand	Drained	CID-G662	60	-0.084
		CID-G665	130	-0.060
		CID-G667	130	-0.161
		CID-G681	1000	0.052
		CID-G685	200	0.067
	Undrained	LIQ-G601	499	0.025
		LIQ-G602	500	-0.020
		LIQ-G609	500	0.068
		LIQ-G616	1200	-0.017

Table 2.2: Calibrated parameters used in NorSand simulations

Soil	CSL			Plasticity			Elasticity	
	Γ	λ	M_{tc}	N	H	χ	I_r	ν
Toyoura Sand	0.983	0.019	1.28	0.41	100	4.4	Eq. 1	0.2
Erksak Sand	0.817	0.014	1.19	0.25	Eq. 2	3.5	Eq. 3	0.2

$$I_r = 18.35 \times 10^6 \times \frac{(2.17-e)^2}{1+e} \times p'^{-0.47} \text{ (after Chaudhary et al. 2004)} \quad (1)$$

$$H = 103 - 980 \times \psi_0 \quad (2)$$

$$I_r = \frac{3}{4} \times \frac{260}{e - e_{\min}} \times \left(\frac{p'}{100} \right)^{-0.5} \quad (3)$$

Table 2.3: Calibrated parameters used in DM04 simulations

Constant	Variable	Value	
		Toyoura Sand	Erksak Sand
Elasticity	G_0	125	188
	ν	0.05	0.20
Critical State	e_0	0.934	0.911
	λ	0.019	0.158
	ζ	0.700	0.076
	M_{tc}	1.25	1.19
Plastic Modulus	h_0	7.05	1.90
	c_h	0.968	0.975
	n^b	1.1	2.1
Dilatancy	A_0	0.704	1.250
	n^d	3.5	2.2

Table 2.4: Average relative and absolute differences of measured vs. predicted peak and critical state deviatoric stresses

Soil Type	Peak - Rel. Diff. [*] (%)		C.S. – Abs. Diff. (kPa)	
	DM04	NorSand	DM04	NorSand
Toyoura Sand	12.90	11.67	92.94	37.19
Erksak Sand	7.06	20.61	84.65	65.68
Average	9.98	16.14	88.79	51.43

$$^* \text{ Relative Difference} = \frac{|\text{Measured Values} - \text{Predicted Values}|}{\text{Measured Values}} \times 100$$

Table 2.5: Average relative difference between measured vs. predicted deviatoric stresses

Axial Strain (%)	2	5	10
DM04	11.96	18.71	22.63
NorSand	19.91	23.45	24.42

Table 2.6: R² values for predicted vs. measured deviatoric stresses

Model	Peak Value	Axial Strain (%)		
		2	5	10
DM04	0.93	0.95	0.97	0.95
NorSand	0.97	0.88	0.93	0.95

Table 2.7: Comparison of predicted vs. measured volumetric behavior

Model	Axial Strain (%)		
	2	5	10
<i>PWP in Undrained Tests (Rel. Diff.)</i>			
DM04	5.59	6.48	12.36
NorSand	18.99	6.64	7.85
<i>Volumetric Strain in Drained Tests (Abs. Diff.)</i>			
DM04	0.20	0.49	0.62
NorSand	0.16	0.50	1.10

Table 2.8: R^2 values for predicted vs. measured volumetric behavior

Model	Axial Strain (%)		
	2	5	10
<i>PWP in Undrained Tests</i>			
DM04	0.92	0.93	0.85
NorSand	0.71	0.96	0.98
<i>Volumetric Strain in Drained Tests</i>			
DM04	0.78	0.84	0.92
NorSand	0.83	0.79	0.79

2.11 Figures

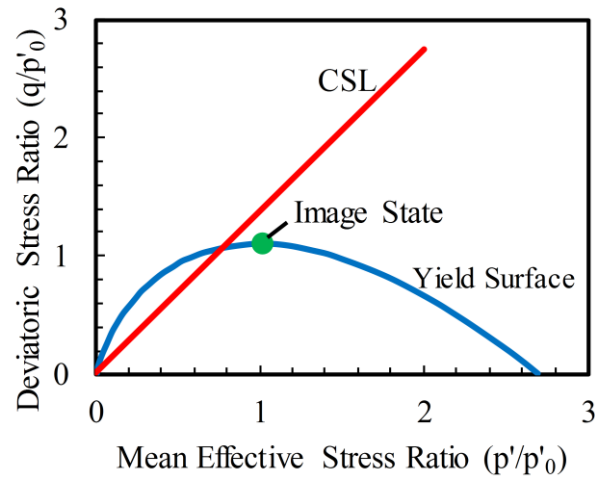


Figure 2.1: Yield surface of NorSand model (after Jefferies and Been 2006)

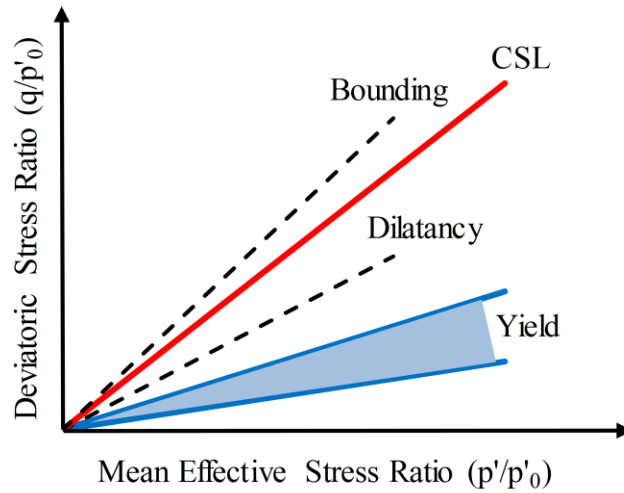


Figure 1.2: Different surfaces of DM04 model (after Manzari and Dafalias 1997)

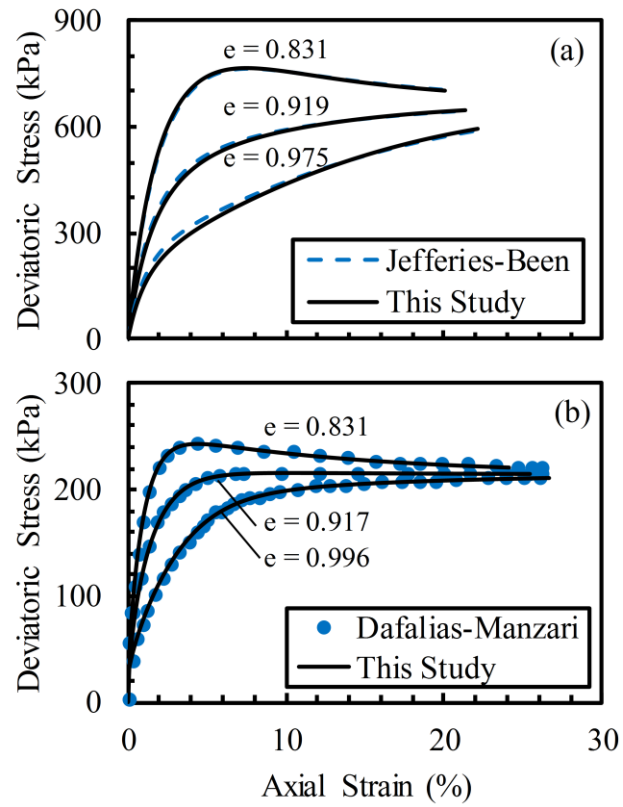


Figure 2.3: Comparison of this study with (a) results of Jefferies and Been (2009) for NorSand; and (b) results of Dafalias and Manzari (2004) for DM04

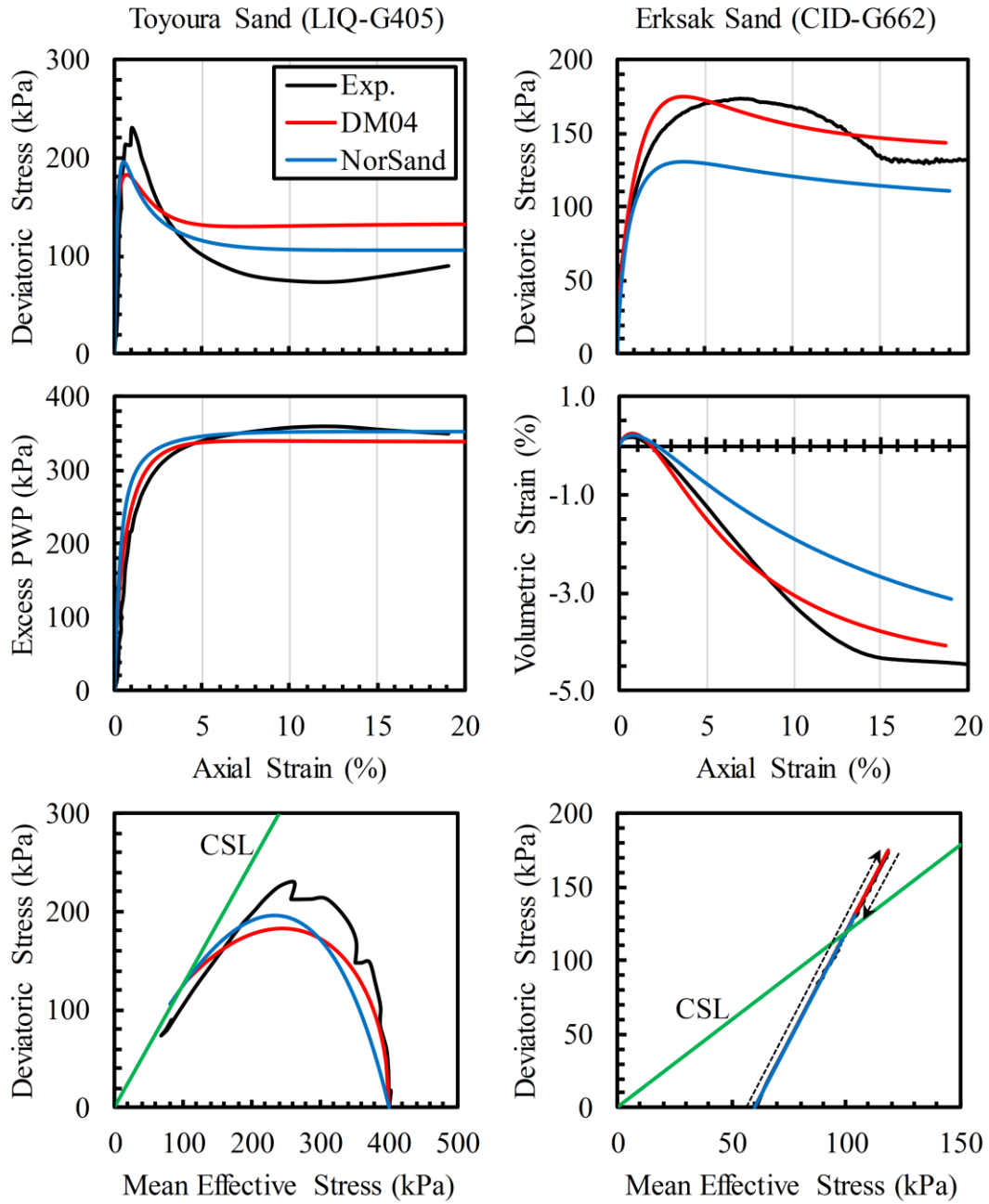


Figure 2.4: Model simulation of soil behavior in drained and undrained triaxial tests

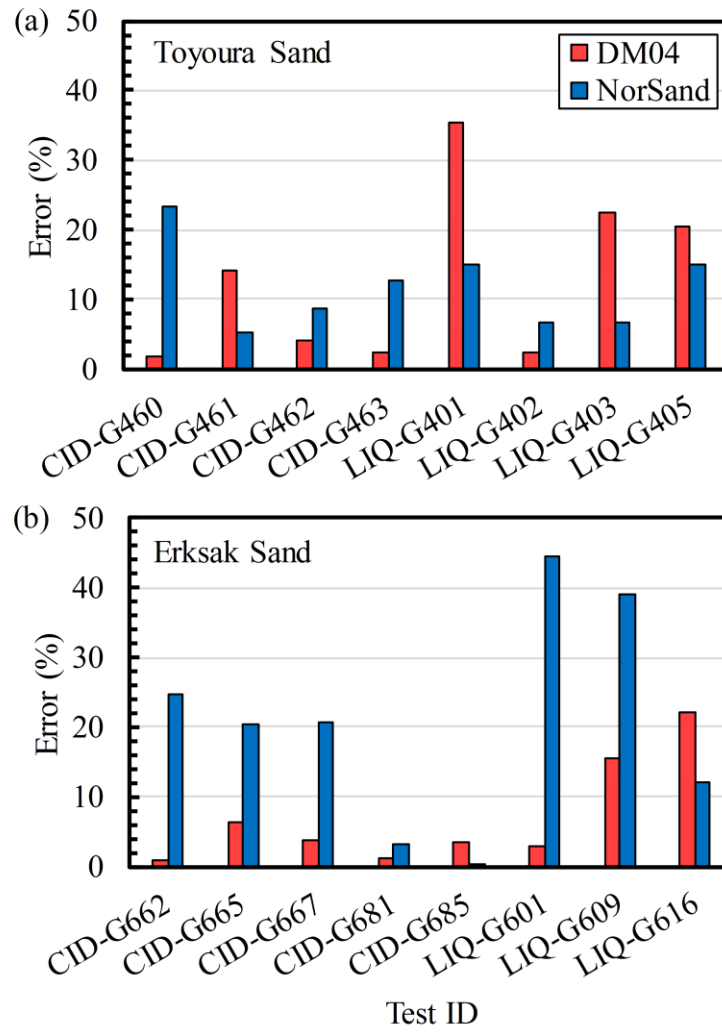


Figure 2.5: Errors in simulations performed by DM04 and NorSand for peak deviatoric stress in triaxial tests performed on (a) Toyoura sand, and (b) Erksak sand

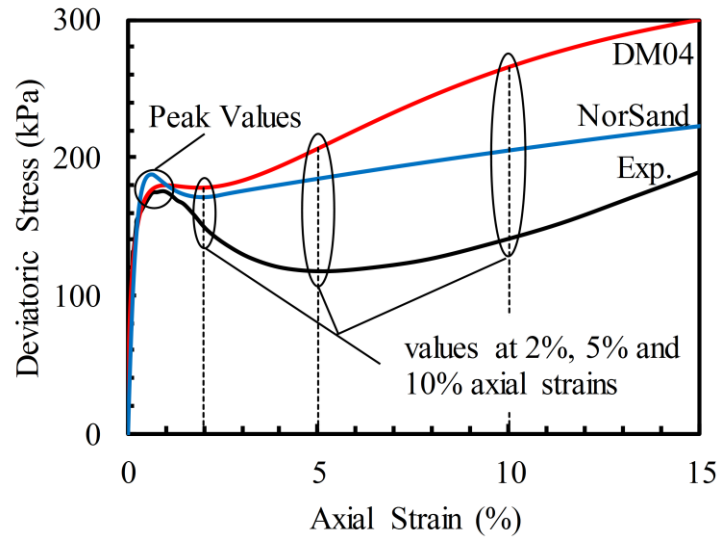


Figure 2.6: Comparison of deviatoric stress at peak and 2%, 5%, and 10% axial strains in test LIQ-G402

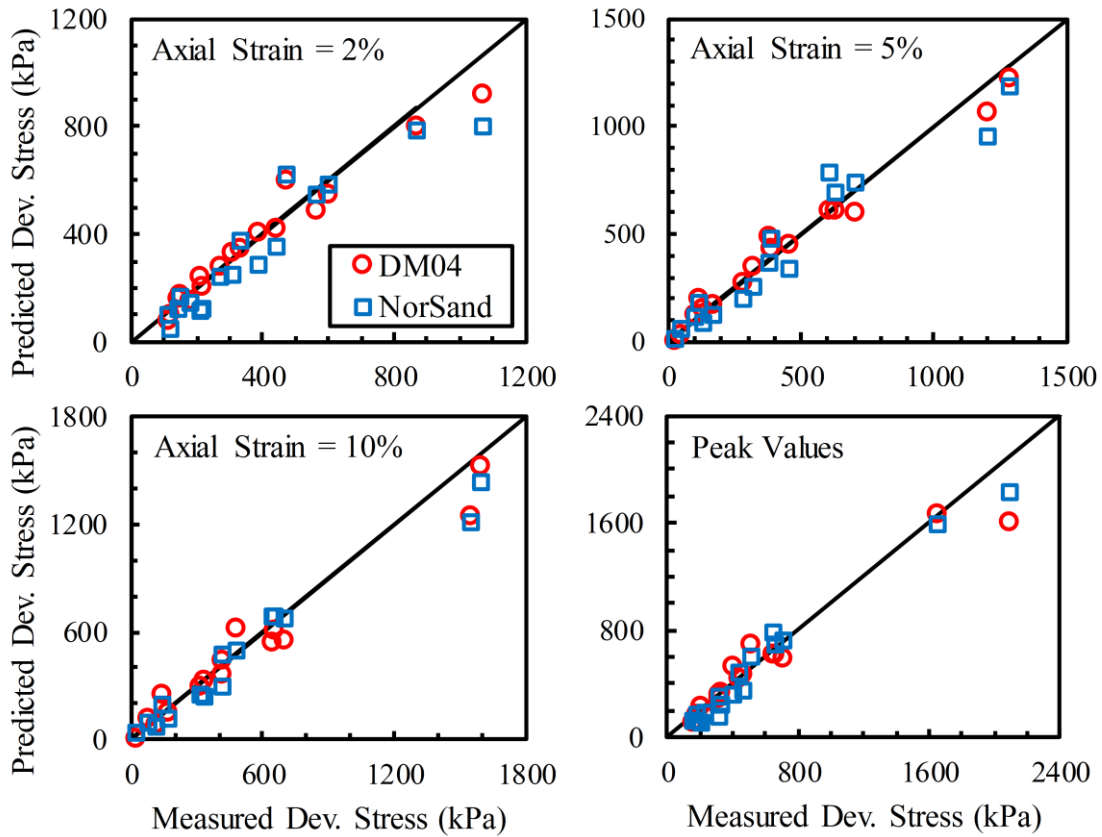


Figure 2.7: Dispersal of predicted vs. measured values of deviatoric stress at different axial strains and peak

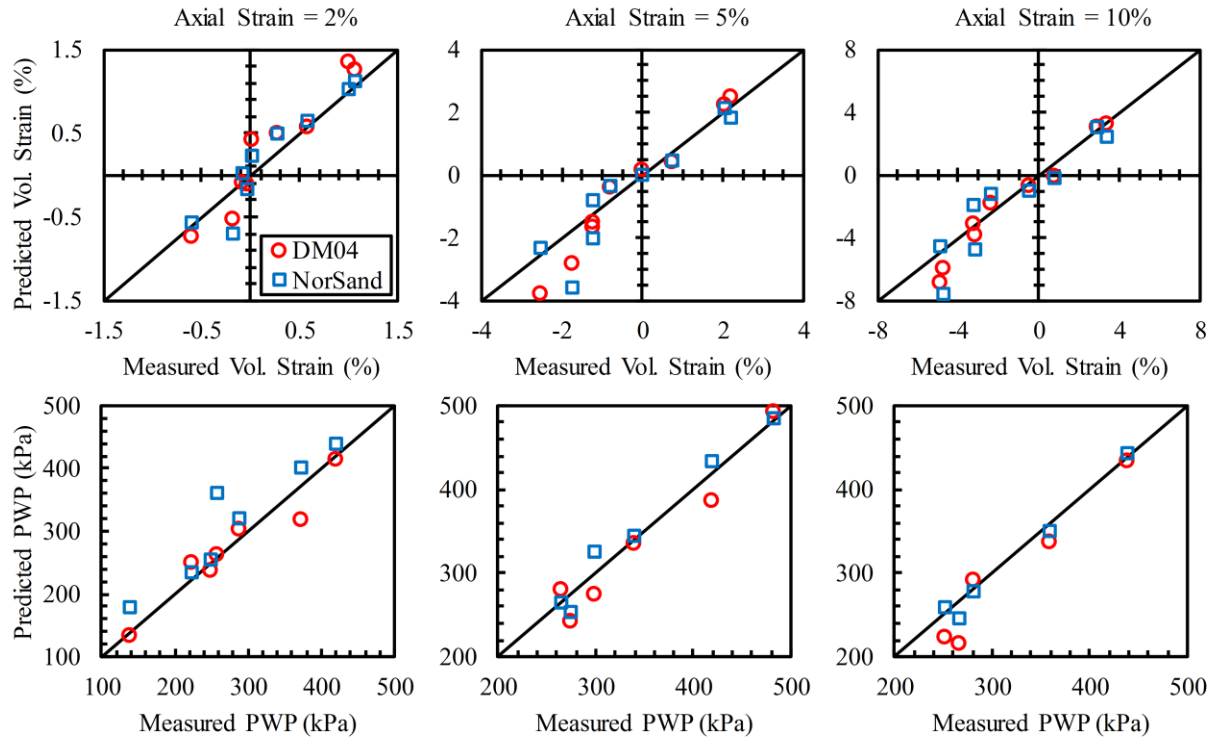


Figure 2.8: Dispersal of predicted vs. measured values for volumetric behavior at different axial strains

Chapter 3

Simulation of Cyclic Soil Behavior Using PM4Sand, a Kinematic Hardening Plasticity Model

The following manuscript has been submitted to the 8th European Conference on Numerical Methods in Geotechnical Engineering.

Soheil Kamalzare made the following contributions to this manuscript:

- Developed two programs for DM04 and PM4Sand models.
- Calibrated the numerical models with results from literature.
- Performed all comparative studies between numerical models and experiments.

Dr. Olgun made the following contributions to this manuscript:

- Supervised the study, and provided valuable feedback.
- Suggested the idea of using constitutive models to evaluate soil resistance against liquefaction.
- Provided underlying ideas for validating the models against experiments.

Simulation of Cyclic Soil Behavior Using PM4Sand, a Kinematic Hardening Plasticity Model

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3.1 Abstract

In this paper we investigate the PM4Sand plasticity model developed by Boulanger and Ziotopoulou in predicting soil behavior under cyclic loading. PM4Sand is a constitutive model based on Critical State Soil Mechanics. It uses a 2-surface kinematic yield surface in modeling soil behavior and more importantly considers the effect of fabric change and void ratio evolution during loading. In this study, we implemented the model in a MATLAB script and further calibrated it with test data in the literature. We simulated the results from earlier experimental studies on Monterey River, Fraser River and Oosterschelde sands at different relative densities and under different confining pressures. Simulated and measured values of number of cycles to liquefaction triggering were compared. The results have shown that, PM4Sand can predict the number of cycles to liquefaction with considerable accuracy for a variety of cyclic load levels for these sands at different relative densities under a wide range of confining pressures.

3.2 Introduction

Predicting soil strength and deformability under different loads continues to be one of the significant challenges in geotechnical engineering. The large body of experimental work in this area has led to a better understanding of complex soil behavior. Researchers in this area have developed different theories in order to explain and predict soil behavior under a variety of loading conditions. One of the major contributions is the Critical State Soil Mechanics (CSSM) theory which was developed over 40 years ago at University of Cambridge in the UK (Schofield and

Wroth 1968). Several other models utilized the concept behind CSSM to predict soil behavior under shearing. One of the models based on this theory is the Dafalias and Manzari model (2004). This model (DM04), which is the modified version of Manzari and Dafalias (1997) model, has been examined closely and it was shown to be capable of predicting soil behavior during monotonic loading in drained and undrained conditions (Vytiniotis 2012). However, it was indicated that, the soil behavior under cyclic loading as modeled by DM04, reaches a state after a couple of cycles and soil strength stops degrading in the following cycles (Vytiniotis 2012). This strongly affects the model's capability in predicting soil strength and deformability under earthquake excitation and other dynamic loads.

Different modifications to DM04 have been suggested (Dafalias et al. 2004, Taiebat and Dafalias 2008) to improve the model, but they have not been successful to capture soil behavior under dynamic loads. Boulanger and Ziotopoulou (2012) utilized the general concept behind DM04 and proposed a model named Plasticity Model for Sands (PM4Sand). They have greatly modified and enhanced different parts of DM04 and indicate that PM4Sand is capable of predicting soil behavior during cyclic loading, especially in terms of the trends for number of cycles to liquefaction. The objective of this paper is to investigate PM4Sand's capability in predicting liquefaction for soils at different relative densities under different confining pressures.

3.3 Plasticity Model for Sands

PM4Sand was first introduced by Boulanger (2010) as a modification to the Dafalias and Manzari (2004) constitutive model. At the time, it was developed as a constitutive model by modifying DM04 to mainly overcome the shortcomings of DM04 in predicting soil behavior during cyclic loading. The model discussed here is the second version of PM4Sand developed by Boulanger and Ziotopoulou (2012). It is based on the constitutive model proposed by Dafalias and Popov (1975) and follows the basic framework of the DM04 model. It is a stress-ratio controlled, critical state compatible, bounding surface sand plasticity model. The model was developed for earthquake engineering applications, with specific goals: (1) to reasonably approximate empirical correlations used in practice, and (2) to be calibrated within a reasonable amount of engineering effort (Boulanger and Ziotopoulou 2012). PM4Sand, uses three main different surfaces similar to DM04.

1. Yield Surface: According to the first basic assumption of the model, a stress ratio defined, two-surface kinematic yield surface is proposed. The yield surface has a wedge shape in the p' - q space and is shown in Figure 3.1(a). Only elastic strains develop for stress ratios inside the wedge shown in the shaded area. The wedge changes orientation and size for the stress ratio to remain on the yield surface what can be called kinematic (i.e. translates in stress space) and isotropic (i.e. expands and contracts in all directions) hardening (Dafalias and Manzari 2004).

2. Dilatancy Surface: PM4Sand has a dilatancy surface that represents the phase transformation surface introduced by Ishihara et al. (1975). The surface is shown in Figure 3.1. Phase transformation surface is largely based on observations from monotonic loading experiments. The soil below this surface shows a contractive behavior (compression in drained tests and positive pore water pressure generation in undrained tests), and it shows a dilative behavior (dilation in drained tests and negative pore water pressure generation in undrained tests) at states above the phase transformation surface. For dense soils, this surface will be below the critical state surface. Therefore, a dense soil if sheared, will show a contractive behavior until reaching this surface, then a dilative behavior up to failure. For loose soils, this surface will be on or very close to the critical state surface. Therefore, a loose soil, if sheared, will completely or mostly show contractive behavior.

3. Bounding Surface: PM4Sand also includes a bounding surface as shown in Figure 3.1. This surface represents the boundary between plastic hardening and softening in soils. Soils, if sheared show a plastic hardening behavior till they reach the bounding surface. Making a peak on it, they continue shearing showing a plastic softening behavior till failure. For dense soils, the bounding surface is always located above the critical state surface. Therefore, dense soils show a more pronounced peak strength during shearing. While for loose soils the bounding surface is located on or very close to the critical state surface. Therefore, loose soils do not show peak strength before failure.

The novel aspect of PM4Sand which can be considered as a significant advancement over other constitutive models is its modified consideration of fabric change during the dilatant phase of plastic deformation. Evolution of fabric change during cyclic loading is an important soil behavior phenomenon. More importantly it can be critical to capture void ratio redistribution within a soil mass to better evaluate the kinematics of possible failure mechanisms in earth

structures. Observations made by many researchers, like Nemat-Nasser (1980), have confirmed that a significant change in the contact-normal orientation distribution of fabric happens during the dilative phase of plastic deformation of soils. This fabric change drastically affects the volume change in the contractive phase of the following reverse loading. Previously, Dafalias and Manzari had used a parameter in their model that develops in the dilative phase of shearing and affects the volume change during the contractive phase of the next opposite loading (Dafalias and Manzari 2004). However, the parameter used in DM04 loses its efficiency after a number of load cycles, leading the model to predict a stable fixed condition during the following load cycles. This shortcoming has been observed by other researchers as mentioned above (Vytiniotis 2012) and the fabric parameter in its original form has been recognized as the cause of this discrepancy (Boulanger and Ziotopoulou 2012). The fabric parameter in PM4Sand has been modified from its original form in DM04 to enable the undrained cyclic stress-strain response to progressively accumulate shear strains rather than lock-up into a repeating sequence of stress-strain loops.

Moreover, one of the major issues with DM04 was that the models predicts an alternate plasticity behavior after some cycles in which the secant shear modulus does not degrade with further cycles of shearing (Vytiniotis 2012). In addition to the problem with fabric parameter mentioned above, this issue also limits the ability of DM04 to model soil behavior under cyclic loading. As a result, it greatly underestimates the number of cycles of equivalent uniform loading to liquefaction, especially at loadings with low cyclic stress ratios. PM4Sand, overcomes this limitation by (a) modifying the fabric parameter which continuously evolves, (b) making the secant elastic shear modulus dependent on the fabric parameter, and (c) introducing a new dilation ratio for the contractive phase. Our studies reported in this paper have shown that these modifications have greatly enhanced the model's ability in predicting soil behavior during cyclic loading and have led to a more realistic representation of number of cycles to liquefaction at different confining pressures and relative densities.

3.4 Implementation and Calibration of the Model

PM4Sand model uses 22 different parameters to represent constitutive soil behavior. These parameters and the approach to estimate them are explained in Boulanger and Ziotopoulou (2012). In our study, we used the reported calibrated parameters. We implemented the model in a

MATLAB script. We performed simulations in comparison to the ones done by Boulanger and Ziotopoulou (2012) to confirm the script. One of our simulations in comparison to those of Boulanger and Ziotopoulou (B&Z) is shown in Figure 3.2 for a soil with 55% relative density sheared in drained monotonic plane strain loading with initial confining stress of 4 atm. As it can be seen from the comparison, the MATLAB script has simulated soil behavior very similar to what Boulanger and Ziotopoulou reported.

Furthermore, we performed simulations under cyclic loading for several different sands that were reported in the literature. Cyclic shake table tests reported by De Alba et al. (1976) were used for this purpose. These tests were performed on Monterey River sand under 55 kPa vertical stress and at 54%, 68% and 82% relative densities. Undrained cyclic shear loading was applied in sine wave form and the number of cycles to initial liquefaction was counted. These tests were simulated in this study using Boulanger and Ziotopoulou's calibrated parameters with minor changes in one of the parameters. This parameter was introduced by Boulanger and Ziotopoulou (2012) as h_0 and it adjusts the ratio of plastic modulus to elastic modulus to avoid division by zero in the computations. For relative densities of 54%, 68%, and 82%, Boulanger and Ziotopoulou have suggested the value of h_0 to be 0.40, 0.47, and 0.54, but we used 0.25, 0.07, and 0.02 respectively. Simulated soil response for $CSR = 0.144$ are shown in Figure 3.3 where shear stresses and strains, and the stress path are presented.

As shown in Figure 3.3, our simulations predicted 11 cycles to liquefaction, as opposed to 12.5 cycles reported by De Alba et al. Moreover, in Figure 3.3a the butterfly effect has been predicted fairly well. In Figure 3.3b, it is seen that the simulated model response did not lock up and has exhibited strength degradation during cyclic loading. It was not possible to achieve this with the DM04 model. In summary, the predicted behavior is in agreement with the expected behavior for a medium dense soil. Other simulations of these tests are shown in Figure 3.4.

As it is seen in Figure 3.4, the model was able to predict the trend of cyclic soil response for medium and dense soils. In a CSR range from 0.25 to 0.13, measured values of number of cycles to liquefaction were within 2.5 cycles difference compared to our simulations. However, in CSR ranges lower than 0.13 the difference in the number of cycles experimented and simulated extensively increased to more than 15 cycles difference.

Continuing our study, we investigated the effects of confining pressure on the number of cycles to liquefaction. For this purpose, Wijewickreme et al. (2005) experiments on Fraser River sand with as-placed relative density of 34%, and under confining pressures of 50 kPa, 100 kPa, and 200 kPa were simulated. All these tests were performed by using the Norwegian Geotechnical Institute (NGI)-type (Bjerrum and Landva 1966) cyclic Direct Simple Shear (DSS) test device. Liquefaction was considered to have triggered when the single amplitude horizontal shear strain (γ) reached a value of 3.75% in the DSS specimen. In the simulations the calibrated parameters by Boulanger and Ziotopoulou (2012) were used with a minor change in one of the parameters as mentioned above. The results are shown in Figure 3.5.

As seen in Figure 3.5, the model has predicted soil response at different confining pressures very well. It was able to provide a reasonable trend for cyclic response of loose soils under different confining pressures. In comparison to medium and dense soils discussed above, the predicted trend for loose soils is slightly less accurate as the values measured in experiments are showing flatter CSR-N responses. At low and medium confining pressures (50 kPa and 100 kPa) the model was able to capture the number of cycles to liquefaction with less than two cycles difference in CSRs larger than 0.1. At CSRs lower than that the model captured the results with approximately five cycles difference to the experimented values. For tests under high confining pressure (200 kPa) the difference has generally increased. At CSR larger than 0.1, the difference was less than 6 cycles and at CSR smaller than 0.1 the difference was approximately 11 cycles. It is worth emphasizing that, at these tests soil was very loose ($D_r = 34\%$), and loose soils show very fragile behavior. Still, the model was able to provide very good estimates of soil behavior.

Expanding our study on the model, we simulated the behavior of soil in 25 different cyclic DSS tests on Oosterschelde sand performed by NGI (1977). These tests were performed on samples with relative densities between 55% and 66%, and under confining pressures of 49 kPa and 343 kPa. The samples had pre-existing shear stress ratios (τ_{st}/σ'_{vo}) from 0 to 0.10. At these tests the liquefaction was considered to have triggered when single amplitude horizontal shear strain reached a value 3% in the specimen. Some of these tests were showing irregular patterns of behavior. For example, in a few of these tests soils with similar properties and under same loading conditions were liquefying with more than 300 cycles difference, or denser soils were liquefying sooner than looser soils. We simulated only the results with regular patterns of behavior. As these tests were performed on a range of relative densities, confining pressures and initial static shear

stresses, it was not possible to calibrate the model to convey all these tests with a specific set of parameters. Therefore, we stayed with the original parameters suggested by Boulanger and Ziotopoulou (2012). One of our simulations for these tests is shown in Figure 3.6. The sample had a relative density of 55% and it was under vertical stress of 343 kPa. It also had a pre-existing shear stress of 34 kPa, and was sheared with a CSR of 0.10 ($\tau_{cyc} = 34$ kPa).

As it is mentioned above, the cyclic shear stress at this test was equal to the pre-existing static shear stress. This has prevented the sample to experience any stress reversal, and liquefaction has happened because of accumulation of positive shear strain. This kind of liquefaction which is called cyclic mobility is simulated well in Figure 3.6. The stress path and strength degradation while soil state is moving toward and on critical state line are shown in Figures 3.6a and 3.6b. The Pore Water Pressure (PWP) generation during loading is shown in Figure 3.6c, and it can be seen that in contrast to DM04 there is no sudden jump in PWP. In summary, the predicted behavior is in agreement with the expected behavior for a medium dense soil. All our simulations of NGI (1977) tests are shown in Figure 3.7.

In Figure 3.7, it can be seen that for a set of tests that the number of cycles to liquefaction varies from 10 to more than 1000 cycles, the simulated behavior has fitted the experiments very well. While at CSRs higher than 0.07, there is a small difference between predicted and simulated values, at CSRs lower than 0.07 the simulated and experimented trend lines have converged and are showing nearly exact values. This represents the capability of the model in predicting cyclic soil response.

Collectively these results indicate that PM4Sand performs well in capturing cyclic soil response for a variety of stress paths and loading conditions. It is generally quite difficult to represent soil behavior to span a wide range of relative densities and confining pressures. Model parameters were kept general between different simulations and we were able to capture cyclic soil response of soils at different relative densities under low to high confining pressures. The results suggest that PM4Sand has strong capabilities in simulating soil behavior and can be used to simulate complex soil-structure interaction problems after proper calibration.

3.5 Conclusions

In this paper, we investigated the PM4Sand constitutive model by Boulanger and Ziotopoulou (2012) and its capability in predicting soil behavior under cyclic loading. This was done in comparison to the DM04 model developed by Dafalias and Manzari (2004) in an attempt to overcome its limitations. For this purpose, a MATLAB script was developed for the PM4Sand model, and it was used to simulate the experiments of De Alba et al. on Monterey River sand, Wijewickreme et al. on Fraser River sand, and NGI on Oosterschelde sand. The main focus was on simulating soil response under a variety of soil characteristics and loading conditions. In particular, we evaluated the number of cycles to liquefaction at different CSRs. In summary this study showed that:

- In contrast to DM04, after a few cycles, the model does not lock-up and continues to degrade soil strength.
- For medium to dense soils, the model was able to simulate the trend of soil liquefaction resistance (CSR-N) fairly well. It was able to predict the number of cycles to liquefaction with less than 2.5 cycles difference at CSRs higher than 0.13.
- For loose soils the model was able to simulate the trend of soil liquefaction resistance fairly well at different confining pressures. The estimated trend had lower accuracy than the estimated trend for medium and dense soils.
- The model was able to provide good estimates of number of cycles to liquefaction in a variety of soil characteristics, loading conditions, and with or without initial static shear stress while not requiring extensive effort for calibration.
- The model was capable of predicting the excess pore water pressure better and in more realistic form than DM04.

While DM04 is a very competent model in predicting soil behavior under monotonic loading, there are significant shortcomings in capturing soil behavior under dynamic loads. The results suggest that PM4Sand has overcome most of the deficiencies with DM04 in cyclic loading. With minor calibration, PM4Sand can provide a relatively accurate estimate of soil resistance against liquefaction during cyclic loadings at different relative densities and confining pressures.

PM4Sand has strong capabilities in simulating soil behavior and can be used to simulate complex soil-structure interaction problems after proper calibration.

3.6 Acknowledgements

Funding for this study was provided by the National Science Foundation for the project “NEESR: Reduction of Seismic Shaking Intensity on Soft Soil Sites Using Stiff Ground Reinforcement” under award CMMI 1208117. Any opinions, conclusions or recommendations expressed herein are those of the authors and do not necessarily reflect the views of the National Science Foundation.

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3.8 Figures

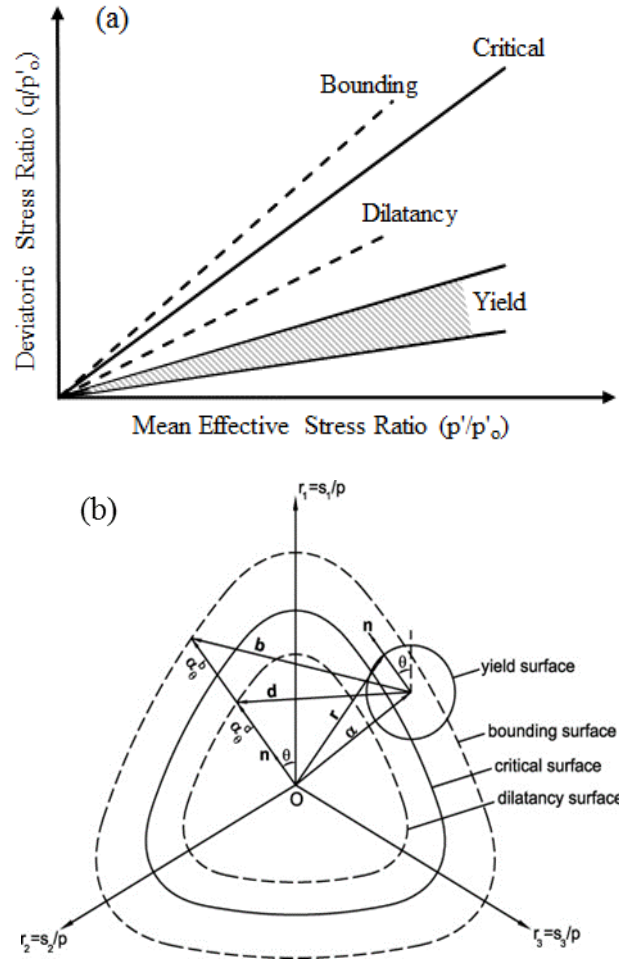


Figure 3.1: PM4Sand Yield Surface; (a) 2D in p' - q surface, and (b) 3D in principal stress ratio space (after Dafalias & Manzari 2004)

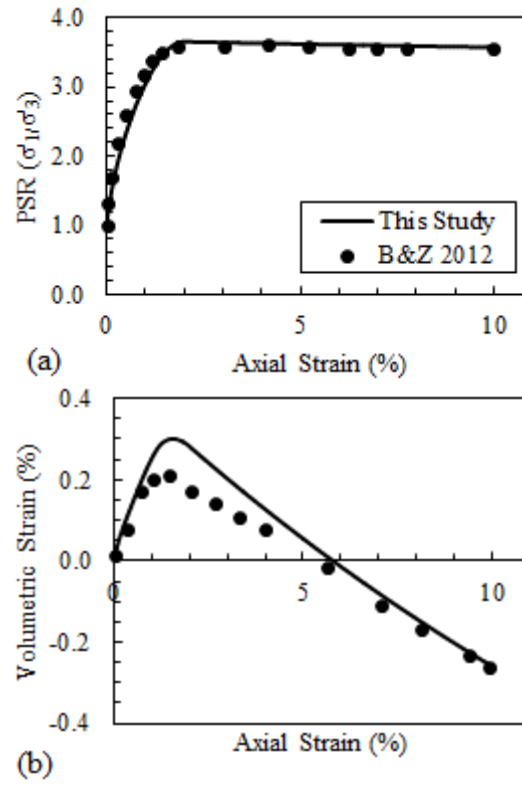


Figure 3.2: Simulation of B&Z results in drained conditions (a) principal stress ratio against axial strain, and (b) volumetric behavior

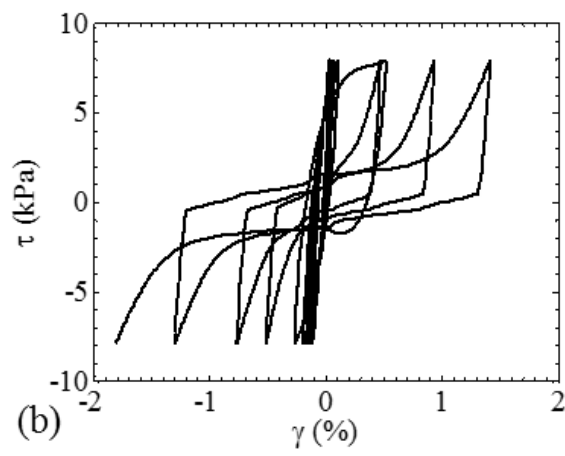
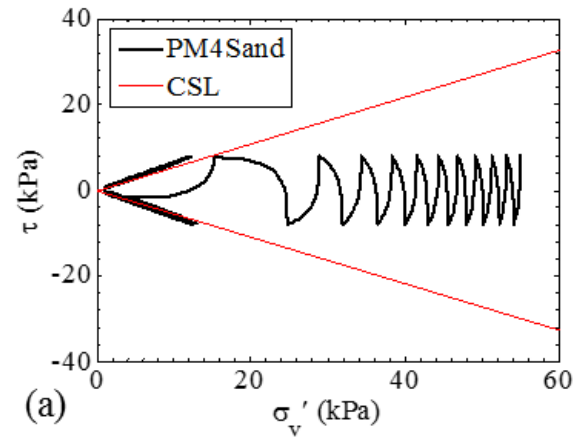


Figure 3.3: Simulated soil behavior in cyclic loading; (a) stress path, (b) shear stiffness degradation of the soil

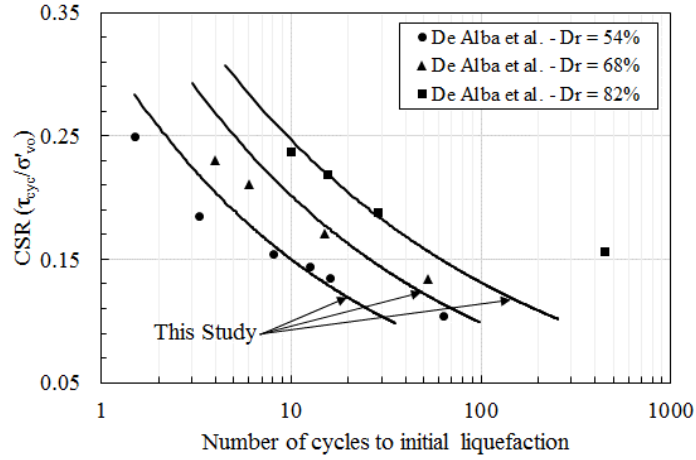


Figure 3.4: De Alba et al. (1976) shake table experiments on Monterey River sands at different relative densities and under 55 kPa vertical stress, and PM4Sand's simulation of these tests.

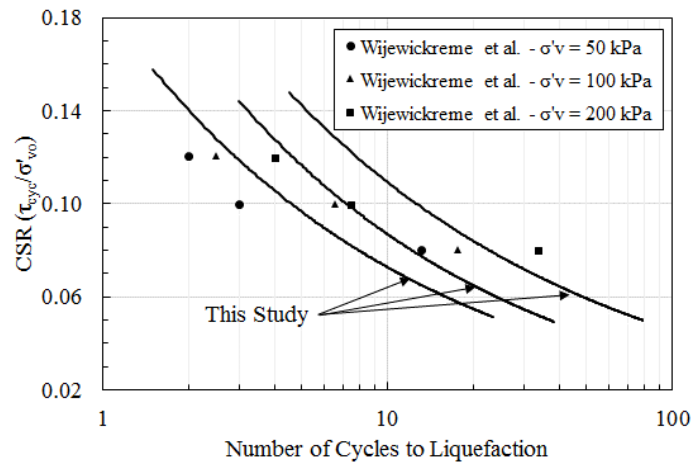


Figure 3.5: Wijewickreme et al. (2005) NGI-DSS experiments on Fraser River sands with as-placed $D_r = 35\%$ and under different vertical stresses, and PM4Sand's simulation of these tests.

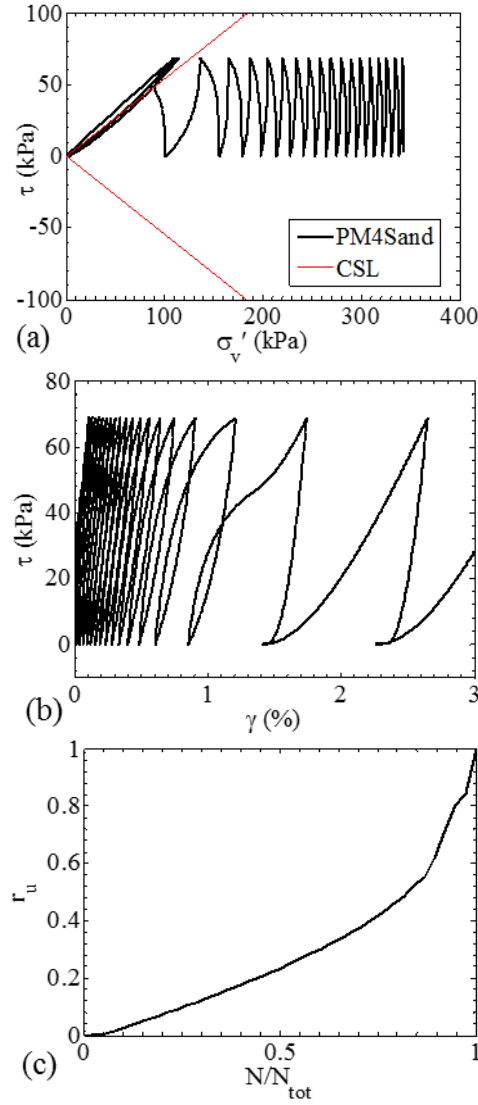


Figure 3.6: Simulated soil behavior in cyclic loading; (a) stress path (b) shear stiffness degradation of the soil, (c) excess PWP ratio generated during different cycles of loading

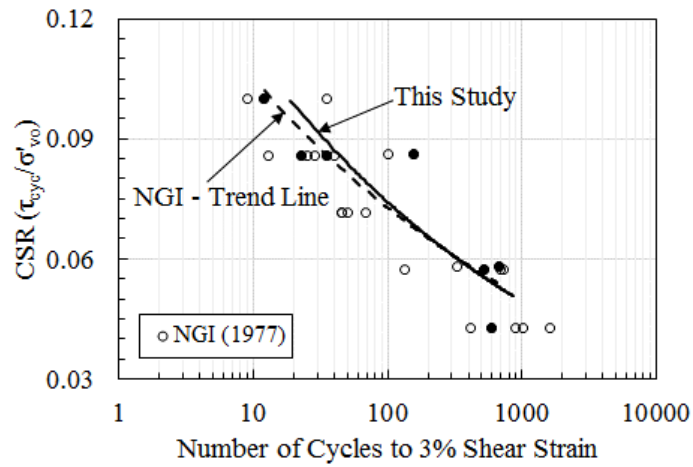


Figure 3.7: NGI (1977) DSS experiments on Oosterschelde sands with $D_r = 55\%$ to 66% and under vertical stresses of 49 kPa and 343 kPa, and with initial shear stress ratios of 0 to 0.1; and PM4Sand's simulation of these tests. The filled points are the NGI experiments which were selected for simulations.

Chapter 4

A Study of Numerical Aspects of Finite Element Modeling in Geotechnical Applications

The authors of the following manuscript intend to submit it to the International Journal of Numerical and Analytical Methods in Geomechanics.

Soheil Kamalzare made the following contributions to this manuscript:

- Developed the finite element models, and the pre- and post-processing tools.
- Performed the analyses with different numerical settings.
- Provided guidelines on proper assignments of numerical settings.
- Evaluated the proposed methodology by performing simulations against recordings in the centrifuge tests.

Dr. Olgun made the following contributions to this manuscript:

- Supervised the study, and provided valuable feedback.
- Suggested the primary idea of using the transfer function to evaluate effect from the numerical settings.
- Recommended the approach to understand and eliminate the numerical errors.

A study of Numerical Aspects of Finite Element Modeling in Geotechnical Applications

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4.1 Abstract

While the finite element method is broadly used in many geotechnical applications today, it has brought to our attention that there is limited practical familiarity with the numerical aspects of this method. Numerical aspects being distinct from geotechnical aspects can have controlling effects to the solution systems. This paper tries to enlighten some of the most influential numerical aspects from a geotechnical stand-point. The focus was put on convergence tolerance, solver time-step, constraint definitions, and, integration, material and Rayleigh damping. For this purpose, the method with different numerical settings was utilized to predict the behavior of a soil site during earthquake excitations. The predictions were compared with analytical approach, and the effects from numerical settings were discussed. The study showed substantial, often adverse, effects from improper numerical settings, and provides guidance on proper assignment of these settings. Furthermore, to evaluate the capability of a well-structured finite element model, a systematic approach was taken to calibrate a nonlinear numerical model, and it was used to predict the behavior of a soil profile during dynamic centrifuge testing. The predicted behavior was compared with recordings for generated pore water pressure, surface settlements and seismic wave propagation throughout soil profile. The finite element model showed great capability in predicting soil response during earthquake excitations with different intensities.

4.2 Introduction

Modern construction and increasing attention toward performance based designs require efficient ways to realistically estimate soil behavior. Predicting soil strength and deformability are two indispensable factors to estimate response and safety of geotechnical structures. Moreover, the continuous distribution of loads due to deformations in soils necessitates incorporation of solution systems that can consider these factors simultaneously. Finite element (FE) analyses incorporated with well calibrated constitutive models provide concurrent predictions for different aspects of soil behavior. However, complexity of the analyses can introduce significant uncertainty into the predictions. These uncertainties can be categorized into two distinct groups: geotechnical and numerical. Geotechnical aspects and the methods to properly build FE Models are comprehensively discussed in literature (e.g. Armstrong 2010, Vytiniotis 2012 and Howell 2013).

This paper tries to enlighten some of the numerical aspects of FE analyses from geotechnical stand-point. The focus is put on numerical convergence tolerance, solver time-step, constraint definitions and damping. These aspects can have improper and sometimes controlling effects on simulations. The paper provides guidance on how to distinguish and avoid improper numerical effects. Thereafter, it introduces a framework to calibrate numerical models to capture pore water pressure (PWP) generation, settlement and seismic wave propagation within soil profile. Finally, it shows how accurate a well built and calibrated model can predict soil behavior during seismic excitations. For this purpose, we have performed nonlinear dynamic finite element modeling to predict results of centrifuge tests with seismic excitations at base.

4.3 Numerical Aspects of Finite Element Modeling

Finite element analysis provides a solution system to estimate how a system reacts to external excitations. System response can be presented by equation of motion, shown in Equation 4.1, with $\ddot{U}_{int}$, $\dot{U}_{int}$ and U_{int} as internal acceleration, velocity and deformation of the system respectively, M as the elemental mass matrix, C as damping matrix, K as stiffness matrix, and F^{ext} as external excitation to the system (e.g. static load, ground motion or heat transfer).

$$M\ddot{U}_{int} + C\dot{U}_{int} + KU_{int} = F^{ext} \quad (4.1)$$

Soil response is represented by stiffness matrix. Recent advancements and introduction of kinematic multi-yield surface constitutive models have provided more realistic predictions for stiffness matrix (e.g. Dafalias and Manzari 2004, Yang and Elgamal 2008, and Boulanger and Ziotopoulou 2013). However, solving equation of motion involves some numerical aspects, which no matter how accurate constitutive models represent soil, can control the response. Our observations in simulations of 2D and 3D multi-layer soil profiles with elasto-plastic materials showed that soil small strain responses such as shear wave propagation and acceleration predictions are considerably affected by FE numerical aspects. This was not the case for soil large strain responses such as PWP generation and settlement, which were dominantly controlled by soil properties within the stiffness matrix.

In our study, the most influential FE numerical aspects were: convergence tolerance, solver time-step, constraint definition and system damping. To better understand the effects of these aspects, numerical modeling was carried out using OpenSees (<http://opensees.berkeley.edu>) finite element program with pre- and post-processing by customized commercial software GiD. OpenSees is an open-source program capable of simulating response of structural and geotechnical systems under static and dynamic loads (Mazzoni et al. 2006).

We studied response of a soil column excited at base under different numerical conditions. For simplicity, soil was modeled as an elastic isotropic material with constant shear wave velocity (V_s) 200 m/s and Poisson's ratio (ν) 0.35. The modeled profile is shown in Figure 4.1. Side nodes above base were tied together to have same displacements and force the column to behave similar to a laterally infinite site. Loma Prieta (1989) earthquake recordings at Yerba Buena Island station (090) was used as the input motion. The model was uniformly excited at its base in the horizontal direction. For numerical aspects, convergence tolerance with norm displacement increment control of $1e-5$, solver time-step 0.005 sec (same as the input motion), constraints with $1e18$ penalties, 1% Rayleigh damping with corner frequencies 5 Hz and 15 Hz, and Newmark integration method with $\gamma = 0.6$ and $\beta = 0.3025$ were used, unless they were changed for parametric studies.

To evaluate effects of FE numerical aspects, we investigated changes in frequency content of motion passing through the soil profile. For this purpose, predicted motion at top surface and input motion at base were converted to frequency domain using fast Fourier transform. Thereafter, transfer function of soil profile was calculated by finding the ratio of frequency amplitudes at

surface to frequency amplitudes at base. The transfer function of an elastic profile is independent of input motion and only represents site response (Roesset 1970). In ideal conditions, we should not see any changes in the transfer function as long as site properties are not changed. Therefore, if different numerical settings change the transfer function, it implies that these settings are affecting (most probably adversely) the solution system. Necessary actions should be then taken to either eliminate or control these effects.

Alternatively, we calculated analytical transfer function, shown in Equation 4.2, from Kramer (1996) to have a baseline for examining FE simulations. In Equation 4.2, f is frequency and a variable, H is profile height, V_s is shear wave velocity, and ζ is damping. The analytical solution for surface motion is shown in Figure 4.1. As the material was elastic, we expected FE simulations to closely replicate the natural frequencies of analytical solutions independent of numerical settings.

$$TF = \frac{1}{\cos\left(\frac{2\pi f H}{V_s \sqrt{1 + 2i\zeta}}\right)} \quad (4.2)$$

4.3.1 Convergence Tolerance

Most FE analyses require solving implicit equation systems. These equations contain an unknown variable in both sides of them which cannot be separated. A common way to solve these equations is to assume a value for the unknown and plug it in to one side of the solution system. Then, the equation will be solved and the unknown will be calculated from the other side of the equation. The convergence test evaluates whether the initially assumed and the latter calculated values are close enough. If they are, the solution system will continue to next step, if not, a better assumption is needed. Tougher convergence limits result in more accurate simulations but require more iterations to converge with longer computation time. Perhaps, the most important factor controlling computation time and stability in implicit solution systems is the convergence tolerance.

Convergence tolerance should be determined based on the finite element application. In our study, large strain predictions such as PWP generation and settlement were realistically obtained by 10^{-3} m tolerance on the norm of displacement residuals (OpenSees: NormDispIncr 10^{-3}) or by

equivalent means. However, this accuracy introduced significant amounts of error in small strain predictions such as shear wave propagation. Improper accuracy limits also amplified computation errors and introduced high frequency noise to the system. In our simulations a minimum of 10^{-5} m tolerance on norm of displacement residuals was required to predict realistic values for shear wave propagation.

To better explain this, we have performed three analyses with convergence tolerances of 10^{-3} , 10^{-5} and 10^{-10} . FE results and the results from analytical solution are shown in Figure 4.2. As it can be seen, all three accuracy limits were able to capture dominant first ($\approx 0.25V_s/H = 5$ Hz) and second ($\approx 0.75V_s/H = 15$ Hz) natural frequencies of the model closely. However, reducing accuracy has introduced high frequency noise to the system. With accuracies 10^{-10} and 10^{-5} noises were not able to reach dominant frequencies. Accordingly, model predictions for Pseudo Spectral Acceleration (PSA) at top surface for these accuracies are almost the same and close to the analytical solution. However, with the accuracy 10^{-3} , noises have reached dominant frequencies and have amplified them. This has affected PSA predictions making the peak PSA almost two times higher than analytical solutions.

These results yield two important outcomes: (1) Adopting very low convergence tolerances and going through long times of computation does not necessarily provide considerably accurate results. (2) Adopting improper convergence tolerances might result in numerical errors in small strain predictions independent of model geotechnical aspects.

It is often impossible from computation time stand-point to adopt high levels of accuracy for large domain systems with numerous degrees of freedom. In these cases, filtering the high frequency noise from predictions can provide more accurate results. For most of geotechnical problems frequencies smaller than 0.05 Hz (period = 20 sec) and larger than 10 Hz (period = 0.1 sec) have negligible effects on system response. These frequencies can be conveniently removed by using band-pass Butterworth filters. However, as it was shown in Figure 4.2c, low accuracies can amplify dominant natural frequencies ($0.05 \text{ Hz} < f < 10 \text{ Hz}$) which are not touched by filters. In these cases, damping natural frequencies can be more effective as discussed later in the paper.

4.3.2 Solver Time-Step

In FE analyses, system response to external excitations is computed at each time-step. In geotechnical transient analyses such as soil sites under dynamic loads, if the solver time-step is larger than the input motion time-step, the solver will miss some of the input frequencies. These frequencies might have controlling effects on small strain predictions. On the other hand, if the solver time-step is too small and not compatible with the convergence tolerance, high frequency noise will be introduced to the system affecting the predictions.

In our study, the input motion had recording $dt = 0.005$ sec (200 Hz). Figure 4.2b shows analysis with solver $dt = 0.005$ sec. We performed two more analyses with $dt = 0.001$ sec (1000 Hz) and 0.01 sec (100 Hz). Results are shown in Figure 4.3. Motion frequency content mirrors after the Nyquist frequency (half of the sampling frequency). Therefore, for the 1000 Hz solver, there will be new amplitudes up to 500 Hz. High level frequencies have negligible effects on site response and for better comparison purposes we are only showing frequencies up to 100 Hz.

Comparing Figure 4.3a with Figure 4.2b, it can be seen that, the 1000 Hz solver has introduced noise in to the system affecting dominant frequencies. This has increased peak PSA predictions, shown in Figure 4.3c, by 14% relative to the analytical solution. This is against the expectation of having more accurate results with smaller time-steps. Reducing time-steps can yield more accurate results only if it is compatible with convergence limits. In other words, smaller time-steps require firmer convergence tolerances to prevent noise generation in solution systems. Otherwise, adding high frequencies to the solution system can cause numerical error in the computation system.

Comparing Figure 4.3b with Figure 4.2b, 100 Hz solver has returned almost crisp transfer function but with smaller amplitudes at dominant frequencies. This has decreased peak PSA predictions by 17% relative to analytical solutions. Moreover, second and particularly third natural frequencies have moved to smaller frequencies. In more complex models, this can have significant effects on small strain predictions. As mentioned before, natural frequencies are only dependent on site properties. Accordingly, inaccurate predictions of natural frequencies implies FE model has not appreciated site profile properly.

These results indicate that, solver time-step should be always equal or smaller than the input motion time-step to capture all the input frequencies. If smaller time-steps are needed, perhaps for convergence requirements, accuracy limits must be set firm enough to prevent noise generations.

The best way to determine whether the time-step is appropriate for a given analysis is to run a second analysis with a reduced time-step. A second analysis with a time-step reduced by 50% should not give a significantly different results (Preisig and Jeremic 2005).

4.3.3 Constraint Definition

In FE analyses, boundary conditions are enforced through constraint definitions. Common methods to define constraints for geotechnical applications are: transformation and penalty methods. Transformation method manipulates the deformation matrix. For example, if a degree of freedom (DOF) is fixed, transformation method puts a zero on that DOF. Computation systems usually recognize these zeros in deformation matrix and remove associated rows and columns to those DOFs from the stiffness matrix. This reduces the size of the stiffness matrix and can considerably help the speed of computations. However, if a node is involved in multiple boundary conditions, simplifying the stiffness matrix can make the computation system unstable. This is one of the most common FE system failures happening in the first step of computations.

The penalty method enforces boundary conditions by adding large values to the stiffness matrix. It does not simplify the stiffness matrix, therefore, the solution system will be mathematically stable, but will take longer time to be processed. Switching boundary definitions from transformations to penalties can be used as a mean to evaluate the stability and the effect of boundary conditions. In the penalty method adding large values to the stiffness matrix can add error to the simulations. Figure 4.2b shows simulations with penalties of 10^{18} . To better demonstrate effect of constraints we have performed three more simulations with 10^{10} and 10^{20} penalties and transformation constraints. Results are shown in Figure 4.4. Comparing Figures 4.4a and 4.4b with Figure 4.2b, it can be seen that decreasing penalties by order of 10^8 has slightly reduced high frequency noise and has almost not affected PSA predictions. However, increasing it only by order of 10^2 has drastically added noise in the solution system. Noises have reached dominant frequencies and have affected PSA predictions by reducing them for about 29% relative to the analytical solution.

In Figure 4.4c, using transformation constraints has substantially reduced the noise in transfer function. This also reveals how much of high frequency noises in penalty methods were originated from constraint definitions. These result show that numerical aspects of constraint definitions can

adversely affect the simulations. If using penalty constraints is mandatory, site transfer function can provide a strong tool to evaluate effects of constraint definitions and ensure proper modeling.

4.3.4 Damping

There are three sorts of damping in FE modeling: material damping, viscous damping and numerical damping. The first two represent physical damping in material. Numerical damping is completely artificial and it is generated from round-off computations. All these damping have considerable and sometimes controlling effects on FE simulations. Herein, we have tried to briefly explain each one of them to distinguish their differences.

4.3.4.1 Material Damping

Material damping or inherent damping emerges from energy dissipation during hysteresis loops. It is a product of soil nonlinear stress-strain behavior, and depends on many factors such as soil tendency to volume change, confining stresses, soil plasticity, and soil particles roll-over and crushing on states without volume change (e.g. critical state). Many of these influential factors continuously change during loading when soil undergoes softening/hardening mostly due to plastic deformations. In numerical simulations, material damping is brought into account directly by constitutive model that represents soil nonlinear behavior. In FE analyses, it is considered through updating stiffness matrix (K) in equation of motion (Equation 4.1). Incorporating stronger well calibrated constitutive models helps to more realistically include this sort of damping. In our analyses, we have used elastic material without any hysteresis loops or energy dissipation. Therefore, this damping does not have any effect on our simulations.

4.3.4.2 Viscous Damping

In numerical analyses, viscous damping is used to compensate inability of the constitutive model to represent inherent material damping completely. In equation of motion, viscous damping is shown as damping matrix (C) and is directly proportional to velocity. In essence, viscous damping is a numerical solution for lack of physical damping from constitutive model. Viscous damping is commonly employed by assuming that the damping matrix is a linear combination of the mass and the stiffness matrices as expressed in Equation 4.3.

$$C = aM + bK \quad (4.3)$$

Where, a and b are viscous damping coefficients. Damping as expressed in Equation 4.3 is often referred to as Rayleigh “proportional” damping. The first term is the mass proportional part and the second term is the stiffness proportional part. Given coefficients a and b , the damping ratio in mode i can be determined as

$$\zeta_i = \frac{a}{2\omega_i} + \frac{b\omega_i}{2} \quad (4.4)$$

Where, ω is angular frequency. In Equation 4.4 damping is dependent on frequency. However, Experiments have shown that inherent damping is not frequency dependent (Ting and Crawley 1992). Therefore, Equation 4.4 must be solved for values of a and b to have damping with minimal changes in concerned frequencies. This is done by solving Equation 4.4 to have certain damping in two corner frequencies. This will lead to

$$a = \frac{2\omega_1\omega_2\zeta}{\omega_1 + \omega_2} \quad (4.5a)$$

$$b = \frac{2\zeta}{\omega_1 + \omega_2} \quad (4.5b)$$

Where, ω_1 and ω_2 are Rayleigh corner angular frequencies. Figure 4.5a shows the Rayleigh damping used in the above simulations with 1% damping (ζ), and corner frequencies 5 Hz and 15 Hz (first and second natural frequencies). As it can be seen, the curvature of Rayleigh damping provides relatively constant damping between corner frequencies and considerably large amount of damping over smaller and larger frequencies. Figure 4.5b shows the same amount of damping but with different frequency corners 10 Hz and 30 Hz. The comparison shows considerable difference in levels of damping over dominant frequencies, emphasizing importance of Rayleigh corners.

The question is always, how to assign best set of corner frequencies? To answer this, one should keep in mind that there is nothing rigorous on choosing corner frequencies, and they are solely chosen to apply a constant damping on interested frequencies. Probably the best way to assign the corner frequencies is to perform a site response analysis or a trial FE modeling and calculate the transfer function of the site. From the transfer function, the dominant natural

frequencies can be determined, and thereafter, Rayleigh corner frequencies should be assigned to provide relatively constant damping over those dominant frequencies.

Proper use of Rayleigh damping can provide considerable advantages for geotechnical applications. Because from one side, we can apply a relatively constant frequency independent damping over dominant frequencies to deal with lack of material damping, and from the other side, we can damp lower and higher frequencies to remove noise from the predictions. Figure 4.5a shows the Rayleigh damping used in the previous simulations with 5 Hz and 15 Hz corners. To better explain the effects of damping, we performed two more simulations, one without Rayleigh damping and one with damping, shown in Figure 4.5b, with 10 Hz and 30 Hz corner frequencies. Results are shown in Figure 4.6. Comparing Figure 4.6a with Figure 4.2b, it can be seen that without Rayleigh damping dominant frequencies have significantly higher amplitudes. This has resulted peak PSA predictions, shown in Figure 4.6c, approximately 44% larger than analytical PSA. Comparing Figure 4.6b with Figure 4.2b, improper assignment of corners has applied higher levels of damping over first natural frequency reducing its amplitude. This has resulted peak PSA predictions approximately 10% smaller than analytical solutions.

A controversial point between results in Figures 4.2b and 4.6b is that stronger damping over large frequencies ($>50\text{Hz}$) in Figure 4.2b has indeed increased the noise in these frequencies. In other words, damping not only has not reduced the noises, but has added to them. This reveals another important aspect of using Rayleigh damping that is damping must be compatible with convergence tolerance over concerned frequencies. As it was shown before, the convergence tolerance $1\text{e-}5$ was not able to accurately capture frequency amplitudes after 50 Hz. Solver inaccuracy has amplified the errors from Rayleigh damping over these frequencies. Moreover, comparing results with convergence tolerance $1\text{e-}10$ (Figure 4.2a) and results shown in Figure 4.6b, it can be seen that, when the solver is accurate, more damping can favorably reduce the noise. In this study, solver accuracy was adequate in dominant frequencies which led to proper PSA predictions. Our further studies with convergence tolerance $1\text{e-}3$ and Rayleigh corners 10 and 30 Hz showed that damping would noise the dominant frequencies reducing the peak PSA by approximately 30%.

Unintended effects of Rayleigh damping can be avoided by continuous monitoring of system frequency response and ensuring dominant frequencies are not improperly affected by damping.

In geotechnical problems, especially when soil undergoes extensive softening (e.g. liquefaction triggering), the most proper way to include damping is through material hysteresis damping. Keeping in mind that hysteretic damping is much more in tune with the actual behavior of system than is viscous damping, which is entirely fictitious (Wilson 2002, Clough and Penzien 2003, and Charney 2008).

4.3.4.3 Numerical Damping

The equation of motion (Equation 4.1) is in terms of acceleration, velocity and displacement. To solve this equation, we need to implement an integration method to convert accelerations and velocities to displacements. One of the most commonly used integration approaches is Newmark (1959) method. Newmark's method is also the basis of many later developed integration methods, e.g. Hilber et al. (1977) method and Chung and Hubert (1993) method. Newmark truncated Taylor's series approximation at third order of differentiation and assumed linear acceleration within a time-step. This resulted in implicit expressions shown in Equations 4.6a and 4.6b.

$$U_{t+\Delta t} = U_t + \Delta t \dot{U}_t + \left(\frac{1}{2} - \beta\right) \Delta t^2 \ddot{U}_t + \beta \Delta t^2 \ddot{U}_{t+\Delta t} \quad (4.6a)$$

$$\dot{U}_{t+\Delta t} = \dot{U}_t + (1 - \gamma) \Delta t \ddot{U}_t + \gamma \Delta t \ddot{U}_{t+\Delta t} \quad (4.6b)$$

Where γ and β are Newmark coefficients that control both the stability of the method and the numerical damping introduced in the system by integration. To better explain this numerical damping, we implemented Newmark's integration method to solve the equation of motion for an **undamped** free vibration system. The equation in terms of displacements will be:

$$U_{t+\Delta t} - (2 - \alpha^2)U_t + U_{t-\Delta t} + \left(\gamma - \frac{1}{2}\right) \alpha^2 (U_t - U_{t-\Delta t}) = 0 \text{ and } \alpha^2 = \frac{\omega_n^2 \Delta t^2}{1 + \beta \omega_n^2 \Delta t^2} \quad (4.7)$$

Where, ω_n is the natural frequency of system. Alternatively, we used finite difference method to the solve equation of motion for a **damped** free vibration system. Again, the equation in terms of displacements will be:

$$U_{t+\Delta t} - (2 - \omega_n^2 \Delta t^2)U_t + U_{t-\Delta t} + \Delta t \zeta \omega_n (U_{t+\Delta t} - U_{t-\Delta t}) = 0 \quad (4.8)$$

Comparing Equations 4.7 and 4.8, it can be seen that the term $(\gamma-1/2)$ in Equation 4.7 is proportional to damping in Equation 4.8. This damping is completely numerical and is different from material hysteresis damping and artificial Rayleigh damping. For $\gamma = 0.5$ there is no numerical damping, for $\gamma < 0.5$ negative (impossible) damping, and for $\gamma > 0.5$ numerical damping is introduced into the system. This damping can be used to effectively reduce computation run-time and damp out high frequency noises (Boulanger et al. 1999). Moreover, when static loads are applied to the system (e.g. gravity analyses), numerical models recognize loads instantly in the first following step of computation, similar to an impact. If loads are strong enough, they can vibrate the system and give acceleration to it. Geotechnical systems require adequate time to dissipate these impacts and reach balance. High values of numerical damping (e.g. $\gamma = 1.5$) can facilitate damping out these unrealistic motions.

Figure 4.2b shows results with slight numerical damping ($\gamma = 0.6$). To better understand effects of numerical damping, we performed two simulations one with no numerical damping ($\gamma = 0.5$) and one with large amount of numerical damping ($\gamma = 1.5$). Results are shown in Figure 4.7. Comparing Figure 4.7a with Figure 4.2b, it can be seen that without numerical damping all natural frequencies have considerably larger amplitudes. Moreover, higher levels of natural frequencies (i.e. 25 Hz, 35 Hz and 45 Hz) are triggered affecting site response. There is also significant amount of noise in high frequencies (>50 Hz). On the other side, in Figure 4.7b, with high levels of numerical damping, only the first natural frequency is distinguishable and all the other natural frequencies are damped out. There is also considerably smaller amount of noise in high frequencies. In Figure 4.7c, the case without numerical damping have resulted a PSA similar to the analytical predictions but with peak value 28% larger. The case with large numerical damping was not able to capture PSA correctly and has predicted the peak PSA 53% smaller than the analytical predictions.

To properly use numerical damping, a user must monitor the site transfer function and ensure that numerical damping is only adjusting frequencies in concerned range (e.g. 0.05 Hz to 10 Hz) and damping out higher levels of frequencies. In our further studies on 2D and 3D non-reinforced and reinforced soil profiles, using slight numerical damping ($\gamma = 0.6$) showed promising results helping us to replicate experimental findings.

4.4 Finite Element Simulation of a Site Response

To further evaluate the capability of a well-structured FE model, following the suggested numerical guidelines, we developed a nonlinear numerical model to capture the results in the centrifuge tests by Rayamajhi et al. (2014). Centrifuge testing was performed at NEES @ UC Davis, Center for Geotechnical Modeling. Model prototype dimensions were 20 m $\times$ 10 m $\times$ 10 m (length, width, height), and it consisted of an unreinforced soil profile with 2 m dense Monterey sand ($D_r \approx 60\%$), underlain by 6 m loose Nevada sand ($D_r \approx 35\%$), underlain by 2 m very dense Monterey sand ($D_r \approx 90\%$). Water table was 1.5 m below ground surface. The model was built in a hinge plate container developed by Fiegel et al. (1994) with negligible friction for lateral movement to represent an infinite free-field soil profile. Chi-Chi (1999) earthquake motion recording at station TCU078E was selected as input motion. The motion was scaled to different amplitudes from 0.02g to 0.74g and applied to the base of the model in the longitudinal direction. In between each earthquake excitation a small amplitude (~ 0.02 g) sine-sweep was applied to the model to test the elastic response of site profile and evaluate effects of earthquake shakings. The sine sweeps were small enough to not generate notable excess PWP or cause settlement. Instrumentations included accelerometers and PWP transducers throughout the profile with settlement transducers on surface. More details about the experiments can be found at Rayamajhi et al. (2014 and 2015).

4.4.1 Finite Element Model Preparation

The focus of numerical modeling was put on key observations from centrifuge tests as pointed out by Kamai and Boulanger (2013): (1) the dynamic response and onset of liquefaction, (2) the amount and pattern of surface deformations, and (3) the pattern of pore pressure dissipation and void ratio redistribution. In our preliminary studies, we compared results from 2D simulations with full scale 20 m $\times$ 10 m mesh and simulations with only one column 0.5 m $\times$ 10 m mesh. The results were close to each other with negligible differences. This was in agreement to our expectations as the model from numerical concerns was homogenous in horizontal direction and had a simple shear dominated behavior. Moreover, Afacan et al. (2014) showed that hinge plate containers impose minimal effects on soil models. They compared recordings from sensors located on soil and on container, and found almost the same measurements. Therefore, we decided to continue

our study by columnar modeling of soil profile to have the advantage of faster computation speed. The FE model is shown in Figure 4.8. Similar to the model used in numerical studies, side nodes above the base were tied together to have same displacements and force the column to behave like a laterally infinite site with simple shear deformations. The base nodes were constrained against any movements.

Soil was modeled using 2D four-node quadUP elements that model dynamic response of solid-fluid fully coupled material based on Biot's theory of porous media. These elements are not appropriate for cases where incompressible behavior is expected due to mesh locking (Oden and Carey 1984). However, mesh locking is not anticipated to have significant effects herein, because soil deformations are dominated by a simple shear mechanism that is less prone to mesh locking. To ensure ineffectiveness of mesh-locking, we performed simulations with nine-node quadUP elements, and the results were the same.

In FE analyses to represent a traveling wave accurately about 10 nodes per wave length are required. Fewer than 10 nodes can lead to numerical damping as the discretization might miss certain peaks of the wave (Preisig and Jeremic 2005). This requirement is shown in Equation 4.9, where Δh is node spacing, λ is wave length, $V_{s,min}$ is minimum shear wave velocity, and f_{Max} is maximum motion frequency.

$$\Delta h \leq \frac{\lambda}{10} = \frac{V_{s,min}}{10f_{Max}} \quad (4.9)$$

The input motion was refined by a Butterworth band-pass filter of order four with 0.05 Hz and 8 Hz corners. This resulted in considerable frequencies up to 10 Hz. Using Equation 4.9 with the maximum frequency of 10 Hz in the input motion, and assuming minimum shear wave velocity of 100 m/s, the maximum length of elements will be 1 m. In our simulations, we tried elements with 0.5 m, 0.25 m and 0.1 m lengths, and the results were not considerably different without missing any relevant frequencies. Therefore, for ease of computation we stayed with element length of 0.5 m. Out-of-plane thickness of soil elements are usually set very large to enforce plane strain condition. In our study we varied thickness from 1 m to 10 m (centrifuge model width), 100 m and 1000 m. Similar to Chang et al. (2013), there was negligible difference between results, and hence the element thickness was set to 1 m.

The Pressure-Dependent-Multi-Yield-02 (PDMY02) constitutive model developed by Yang and Elgamal (2008) was used to predict soil behavior. PDMY02 incorporates various features to more realistically predict soil dynamic behavior, such as multiple yield surfaces to capture nonlinear soil behavior, phase transformation surface to properly predict soil volumetric behavior, pressure dependent shear modulus (G) which is continuously updated during the simulations to more realistically predict soil strength (Prevost 1985), and fabric change to enhance contractiveness on reversal of load after dilative phases (see Dafalias and Manzari 2004 for a detailed discussion). In our simulations we used calibrated parameters suggested by Khosravifar (2012) with further calibrations as discussed later.

During the centrifuge test, 16 different motions (scaled Chi-Chi motion with increasing amplitude and Sine-Sweeps with very small amplitudes) were applied to the model with enough time in between for PWP to dissipate. Kamai and Boulanger (2013) investigated two possible approaches for running simulations of consecutive shaking events: (1) simulate all shakes in sequence with time for dissipation between them (as occurred in centrifuge test itself) or (2) simulate each shake individually using a newly generated mesh with a best estimate on the geometry and other properties in the beginning of that shaking event. They concluded overall the two approaches yielded comparable results. Moreover, field experiments conducted by Baez and Martin (1992) revealed that when PWP ratio does not exceed 0.7, there will be little noticeable volume change with no improvement in densification. In this study, we simulated model behavior under motions shown in Table 4.1. All these motions had amplitudes less than 0.13g and were not able to change the geometry of the model significantly or cause PWP ratios more than 0.7 to considerably densify or loosen the soil. Therefore, we simulated model behavior by using individual newly generated meshes. The acceleration recordings for EQ1 and EQ2 at base and surface of centrifuge model are shown in Figure 4.9. The recordings at the base were directly input to FE model in form of uniform horizontal excitations at the base.

For FE numerical aspects, we used convergence tolerance of $1e-5$ on norm displacement residuals to reduce high frequency noise and more properly simulate small strain responses. Solver time-step was set equal to input motion 0.01 sec recording time-step to consider all input frequencies and not introduce higher levels of frequency into the solution system. Constraints definitions with the transformation method failed to assemble the model because some of the nodes had multiple boundary conditions (e.g. periodic boundary conditions with zero PWP). Therefore,

we used the penalty method with $1e18$ enforcements on stiffness matrix. PDMY02 provides the model with proper material hysteresis damping. Therefore, we used only 1% Rayleigh damping proportional to mass and initial stiffness matrices with first and second natural frequencies of profile as corners to minimize convergence problems at lower levels of shaking. Newmark integration method with $\gamma = 1.5$ - $\beta = 1.0$ was used in gravity analyses to extensively damp out oscillations from impacts of weight loads. Values of $\gamma = 0.6$ - $\beta = 0.3025$ were used for dynamic analyses to expedite computations and help damping out high frequency noises. Before and after all these numerical assignments, the transfer function of the model was evaluated to avoid any detrimental numerical effects.

4.4.2 Model Calibration

4.4.2.1 Small Strain Calibrations

Soil small strain parameters are controlled by elastic properties. To evaluate these properties, we studied centrifuge model response to small amplitude sine sweeps. These motions with 0.02g Peak Base Accelerations (PBA) were not able to generate notable PWP and deformations in the model. Soil behavior was predominantly elastic, and therefore, first natural frequency (f_1) of model can be approximated as $f_1 \approx 0.25V_s/H$. In other words, f_1 can provide us a sense on site shear wave velocity, and through that, an estimation on shear modulus since $V_s = (G/\rho)^{0.5}$. To find f_1 , acceleration recordings at base and on top of the model were converted to frequency domain, and model transfer function was calculated. Results, shown in Figure 4.10, indicate two important outcomes: (1) model f_1 was approximately 3 Hz, and (2) it did not change after stronger shakings. Therefore, numerical elastic properties must be calibrated to have $f_1 \approx 3$ Hz, and they should be kept the same for all simulations.

Using suggested parameters by Khosravifar (2012) in FE analyses, f_1 was simulated as 3.74 Hz. This resulted inaccurate predictions of PSA on top surface. Therefore, recognizing Nevada sand layer as the dominant soil in the profile with largest thickness and loosest relative density, we reduced its shear modulus to 60% of the default values to have $f_1 \approx 3$ Hz like experiments. Figure 4.11 shows FE PSA predictions with suggested and calibrated parameters against measurements for a Chi-Chi earthquake motion with PBA = 0.02g. As it can be seen, suggested parameters resulted almost two times greater peak PSAs and in different periods from experiments. Favorably,

calibrated parameters closely captured peak PSAs and their periods. Therefore, for rest of the simulations we used our calibrated shear modulus.

4.4.2.2 Large Strain Calibrations

Soil volumetric behavior is represented by constitutive model. PDMY02 has six parameters to simulate volumetric behavior. Three parameters for contraction phase and three parameters for dilation phase to control volume change rate, pore water pressure generation, fabric damage and overburden stress effects on volumetric behavior (Khosravifar, 2012). In essence these parameters control coefficient of volume compressibility (m_v) in Equation 4.10.

$$c_v = \frac{k}{m_v \gamma_w} \quad (4.10)$$

Where c_v is coefficient of consolidation, k is permeability and γ_w is unit weight of water. c_v is the key parameter controlling volume change and pore water pressure generation in FE simulations. To calibrate c_v we either need to calibrate the six parameters for m_v or adjust k to result a proper c_v . Calibrating m_v requires a set of laboratory tests to find best fitting values, thus, it is more convenient to calibrate k . Permeability, itself, plays an important role in determining the liquefaction response of homogenous sand deposits, affecting especially the depth of liquefaction, the speed at which excess PWP dissipates after shaking, and the character of the ground surface settlement (Sharp et al. 2003). Therefore, calibrating it requires great attention to evaluate gains and losses in simulations.

The Nevada sand used in centrifuge model is a relatively uniform, clean, fine sand. It has a specific gravity (G_s) of 2.64, a mean grain size (D_{50}) of 0.17 mm (model scale), a uniformity coefficient (C_u) of 1.64, and a permeability of 0.002 cm/s (Kamai et al. 2008 and Arulmoli et al. 1992). For a medium sand, typical values of m_v are generally on the order of $4e-5$ to $10e-5$ (1/kPa) (Pestana et al. 1997), and vary based on relative density, confining pressure, and the peak excess pressure in the soil (Lee and Albaisa 1974). Howell (2013) comprehensively studied PDMY02 model and found out that the equivalent m_v values modeled by PDMY02 are 1/10 to 1/3 smaller than those measured in experiments. Therefore, permeability of FE model must be set to 0.002/10 cm/s to 0.002/3 cm/s.

Centrifuge tests were performed under 50g centrifugal acceleration with a fluid 30 times more viscous than water. A model saturated with water and tested at N g would have permeability N times greater than the same soil in prototype saturated with water and tested at 1g (Kutter 1995). Therefore, the permeability of soil was 1.67 ($=50/30$) times higher than actual values, resulting in 0.0003 cm/s ($=0.002/10 \times 1.67$) to 0.0011 cm/s ($=0.002/3 \times 1.67$). Seed et al. (1975) suggested that for a loose sand ($D_r = 30\%$ to 40%), m_v can increase by as much as a factor of 10 as liquefaction is approached. Since liquefaction happened very fast during the tests, and PDMY02 maintains a relatively constant permeability during simulations, the permeability of numerical model should at most be set to 0.003 cm/s to 0.011 cm/s. Based on these, we chose the permeability range of 0.0003 cm/s to 0.01 cm/s and performed simulations to find best PWP generation and settlement relative to experiments. Our model performed best with permeability 0.01 cm/s.

Top and bottom Monterey sands were built as drainage boundaries for Nevada sand layer. The top layer was almost dry (except 0.5 m) and did not experience liquefaction. The bottom layer was very dense ($D_r = 90\%$) and did not experience liquefaction either. None of these layers deformed considerably relative to loose Nevada sand layer during all the excitations. We chose permeability of 0.05 cm/s for Monterey sand layers to have similar PWP distribution to recordings.

After calibrating permeability of layers and ensuring close capture of PWP recordings, we calibrated PDMY02 contraction rate parameter (c_1) to better capture settlements. Khosravifar (2012) has suggested $c_1 = 0.06$. We chose $c_1 = 0.08$ to have more contractive behavior in Nevada sand layer and better replicate settlements.

4.4.3 Acceleration Response

Computed and measured pseudo spectral accelerations throughout the model profile are shown in Figure 4.12. As seen by comparison, FE model was able to closely capture peaks and their periods, and the general trend of PSAs at all depths. The discrepancies increased with stronger earthquakes and with wave upward propagation.

Idriss (1990) indicated that ground motions with moderate acceleration levels (less than 0.4g) will be amplified while passing through soft soil profiles. Findings in centrifuge tests are consistent with Idriss (1990). EQ1 and EQ2 with accelerations less than 0.4g were amplified while passing through loose Nevada sand. As it was shown, the FE model quite accurately predicted this

phenomenon. Moreover, similar to Seed et al. (1976) and Armstrong et al. (2013) findings, motion passing through loose soil profile acquired longer-period PSA on top of soil surface. The FE model was able to closely capture this phenomenon too.

4.4.4 PWP and Settlement Responses

EQ1 was not strong enough to generate notable PWP and settlement. Computed and measured PWP and surface settlements for EQ2 is shown in Figures 4.13 and 4.14. In our FE simulations like Howell (2013), PWP dissipated immediately after motion intensity decreased to small amplitudes. This was in contrast with recordings which show limited PWP dissipations even after motion stopped. It was possible to reduce permeability of numerical model to delay PWP dissipation, but this would adversely affect settlement predictions. In Figure 4.13, it can be seen that except for predictions at 4 m depth, numerical predictions have reasonable agreement with measurements. Moreover, FE model was able to replicate liquefaction triggering accurately both for time and PWP generation. In Figure 4.14, it can be seen that the FE model was able to properly predict the amount and the occurrence of surface settlement. Measured and predicted values have about 5% difference in total amount of settlements.

4.5 Summary

In this study, we discussed numerical aspects of finite element analyses from a geotechnical stand point. The primary focus was put on convergence tolerance, solver time-step, constraint definition and system damping. We modeled a soil column and studied its response to earthquake base excitations with different numerical settings. Response sensitivity was evaluated by comparing model transfer function and surface pseudo spectral accelerations from FE analyses with analytical solutions.

Our investigations showed that FE numerical aspects being distinct from geotechnical aspects can have substantial and deterministic effects on simulations. Convergence tolerance should be compatible with numerical model applications. Low tolerance limits cannot necessarily increase the accuracy of simulations, while high tolerance limits can drastically affect small strain behaviors. Solvers with time-steps smaller than input motion time-step can introduce high frequency noise to the solution system affecting the results. While, solvers with larger time-steps

can miss some of the influential frequencies. Manipulating the stiffness matrix to enforce constraints can utterly disturb the results, and should be avoided if possible. Whereas, manipulating the displacement matrix can de-stabilize the solution system particularly if some nodes are tied in different constraints.

Rayleigh damping can be used to complement lack of damping from material hysteresis energy dissipations, and damp out very low and high frequency noises. However, improper use of Rayleigh damping can affect proportionality between dominant natural frequencies and severely affect the simulations. Numerical damping from computation round-offs can help preventing unintended model oscillations, stimulate computations and damp out high frequency noises. Yet, imprudent round-offs can damp model dynamic response adversely.

Results collectively showed the significance of finite element numerical aspects on simulations of geotechnical systems. A user can easily monitor error in his/her simulations and blame the constitutive model to not representing soil well enough. While the main reason might be behind numerical structure of model and using complex advanced constitutive models would not have any effects on predictions.

Results also show that the model transfer function can be used as a strong tool to evaluate effects of numerical settings on finite element simulations. Numerical settings alter frequency amplifications in the transfer function, and can have controlling effects when they affect dominant natural frequencies. Improper settings can be avoided by continuous evaluation of the transfer function and ensuring that the dominant frequencies are not adversely disturbed by these settings.

Finally, to evaluate well-built and calibrated finite element models, we developed a model to replicate soil response in centrifuge testing. Our model showed great capability in predicting soil response during different seismic intensities. Acceleration predictions closely fitted measurements throughout soil profile. Pore pressure developments and surface settlements were captured with negligible differences. Finite element analyses were in good agreements with literature on effects of soft soil profiles on propagating waves.

4.6 Acknowledgements

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4.7 References

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4.8 Tables

Table 4.1: Simulated input motions in this study

Motion	Title	Peak Base Acceleration (g)
Sine Sweep	S1	0.02
Chi-Chi Earthquake	EQ1	0.02
Sine Sweep	S2	0.02
Chi-Chi Earthquake	EQ2	0.13
Sine Sweep	S3	0.02

4.9 Figures

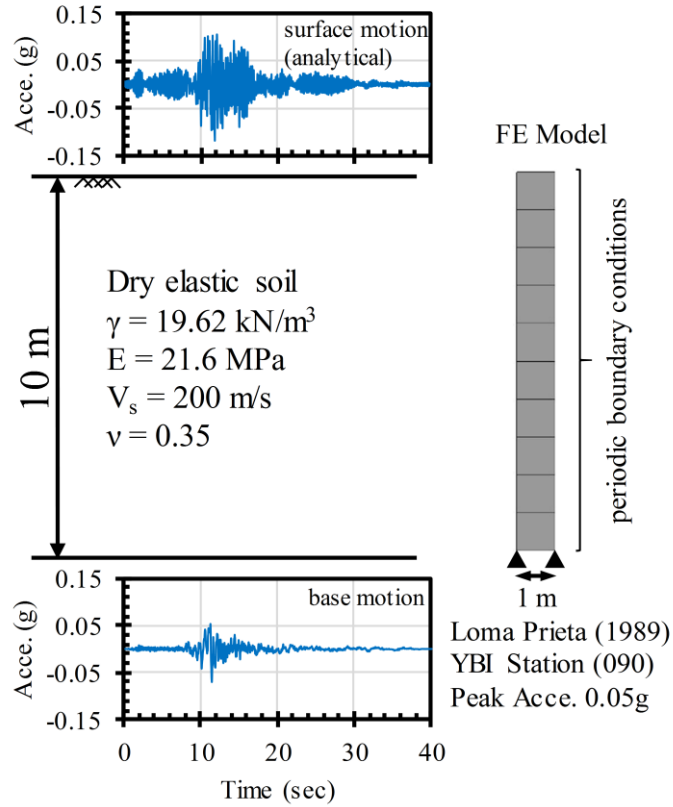


Figure 4.1: Studied soil profile for evaluating numerical aspects of FE analyses with analytical solutions for surface motion.

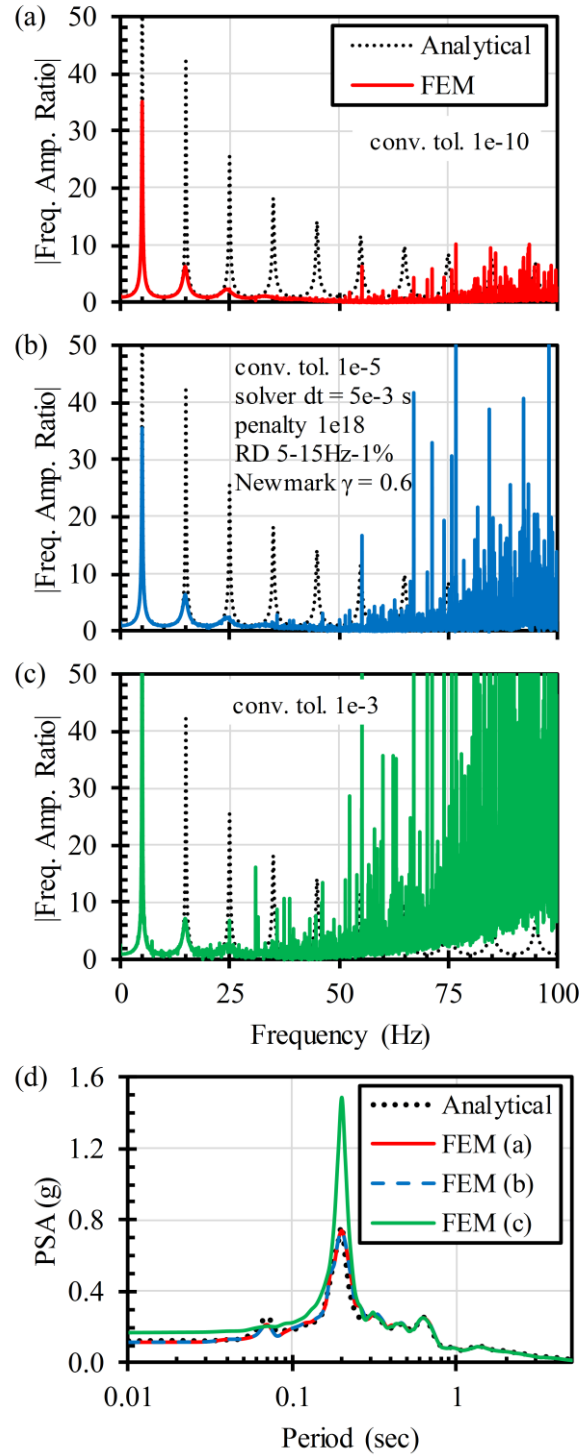


Figure 4.2: Comparison of FE simulations with different convergence tolerances on norm displacement increments: a) 10^{-10} m, b) 10^{-5} m, c) 10^{-3} m with analytical solutions, and PSA predictions at soil surface

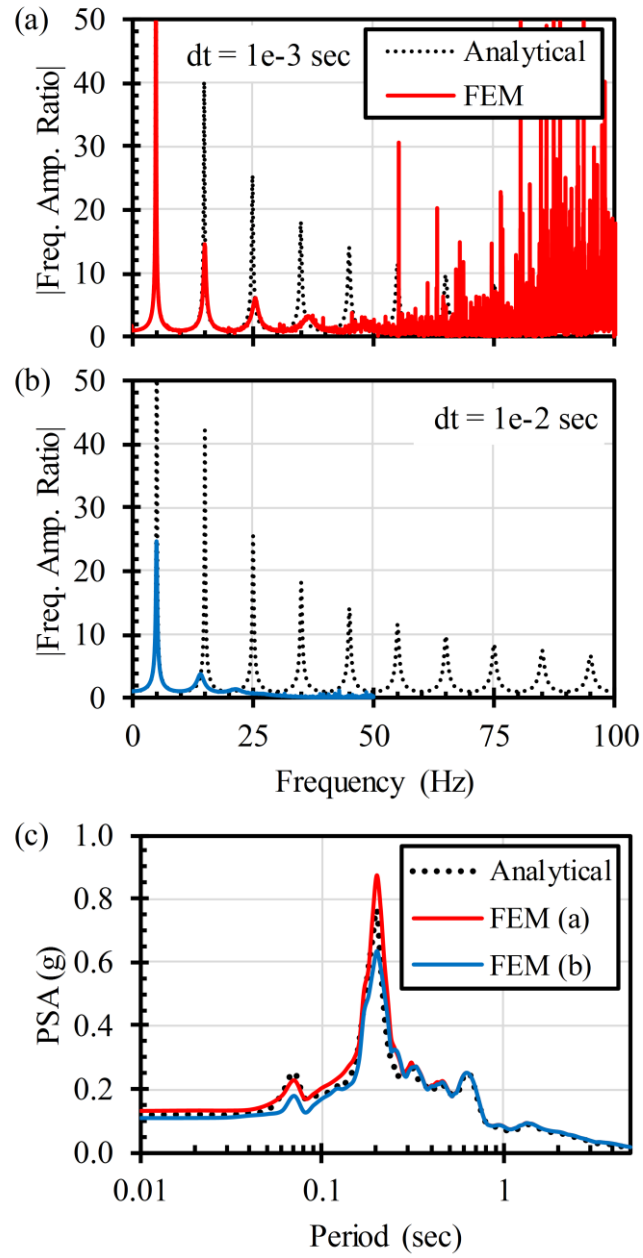


Figure 4.3: Comparison of FE simulations with different solver time-steps: a) 0.001 sec, b) 0.01 sec with analytical solutions, and PSA predictions at soil surface.

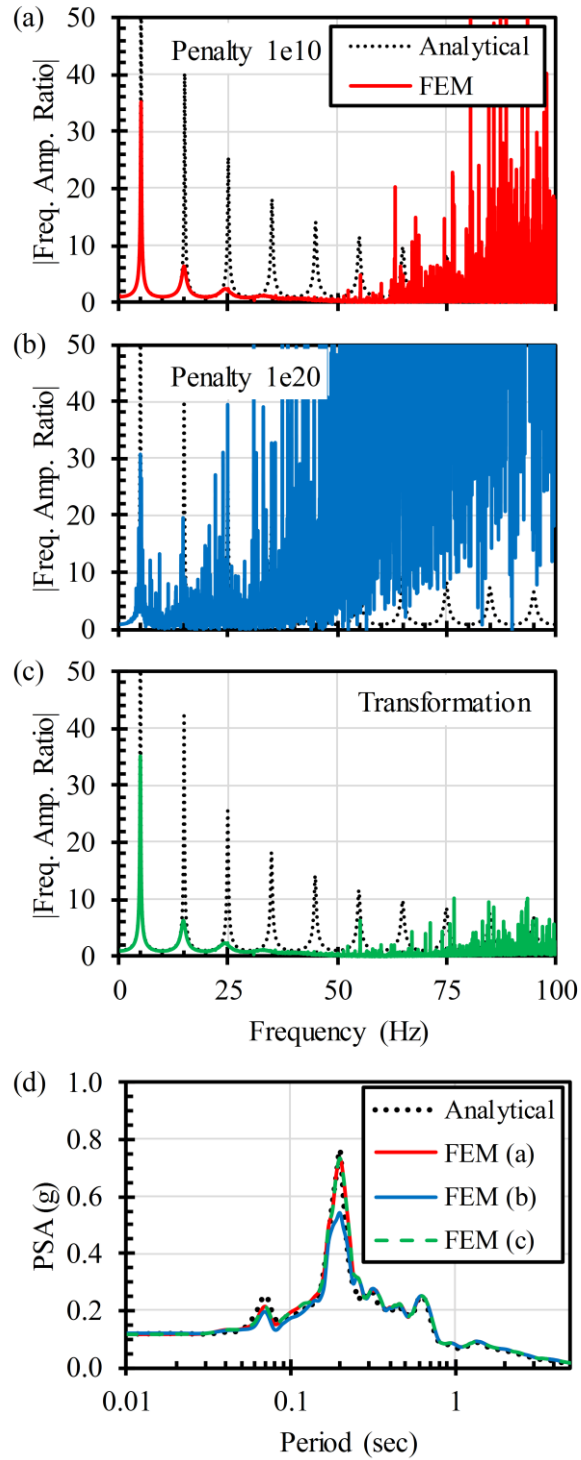


Figure 4.4: Comparison of FE simulations with different constraint definitions: a) penalties 10^{10} , b) penalties 10^{20} , c) transformations with analytical solutions, and PSA predictions on soil surface.

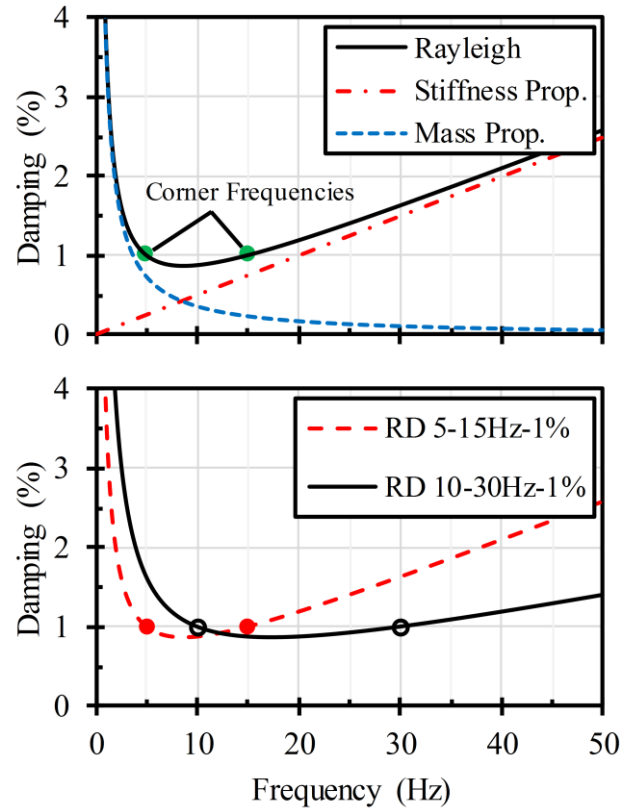


Figure 4.5: (a) Mass and stiffness proportional components of Rayleigh damping, (b) comparison of Rayleigh damping curves with different corner frequencies.

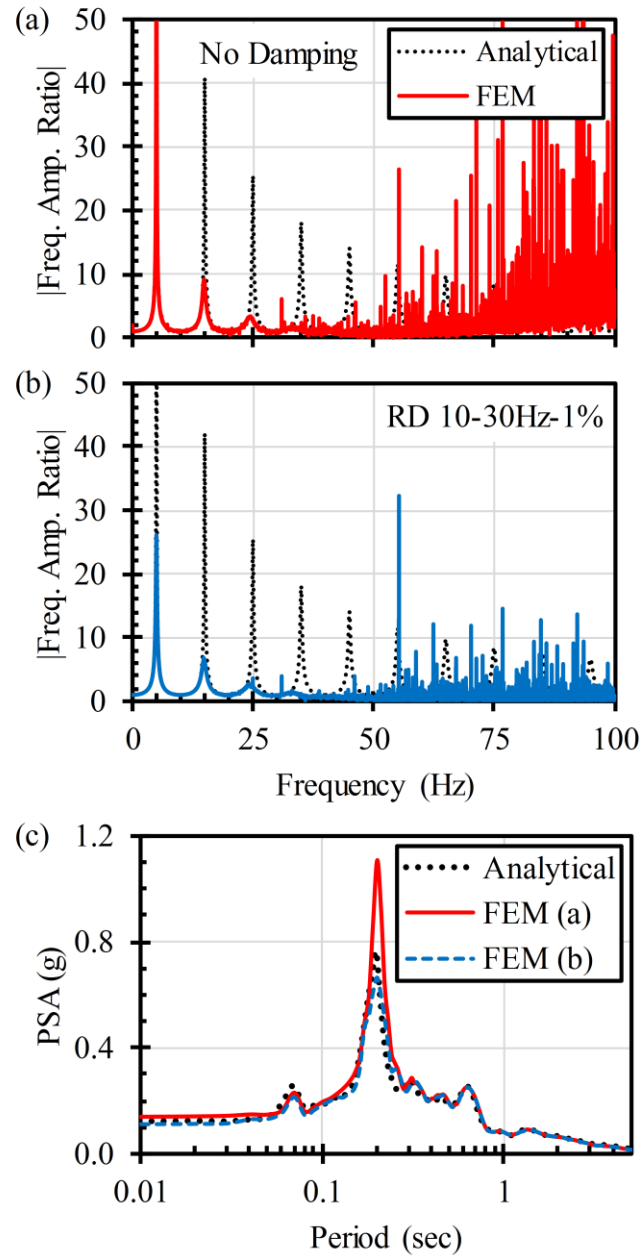


Figure 4.6: Comparison of FE simulations (a) without Rayleigh damping, (b) with frequency corners 10 Hz and 30 Hz against analytical solutions, and PSA predictions on soil surface

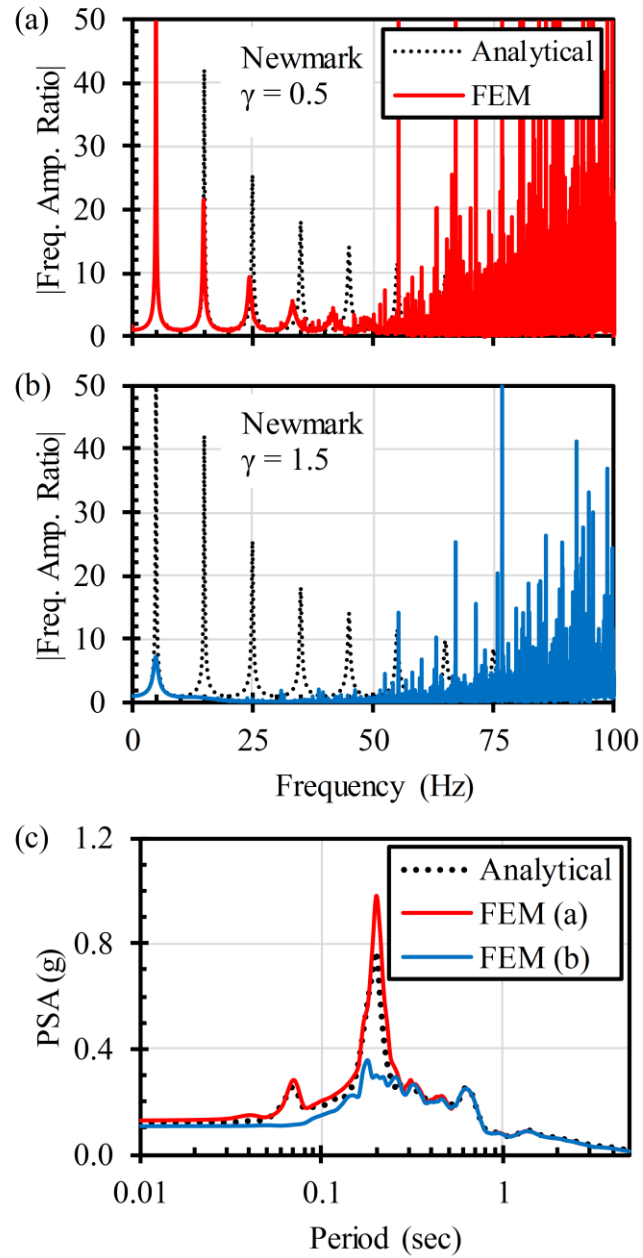


Figure 4.7: Comparison of FE simulations with different Newmark γ coefficients: a) 0.5 b) 1.5 against analytical solutions, and PSA predictions on soil surface.

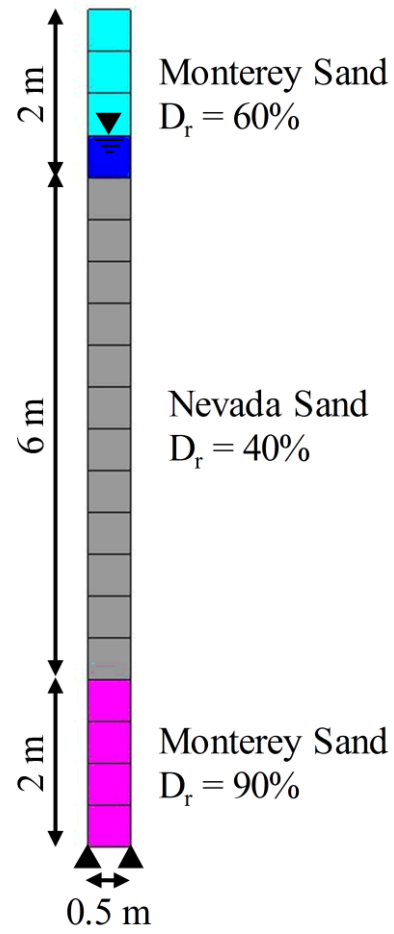


Figure 4.8: FE model used to study centrifuge tests

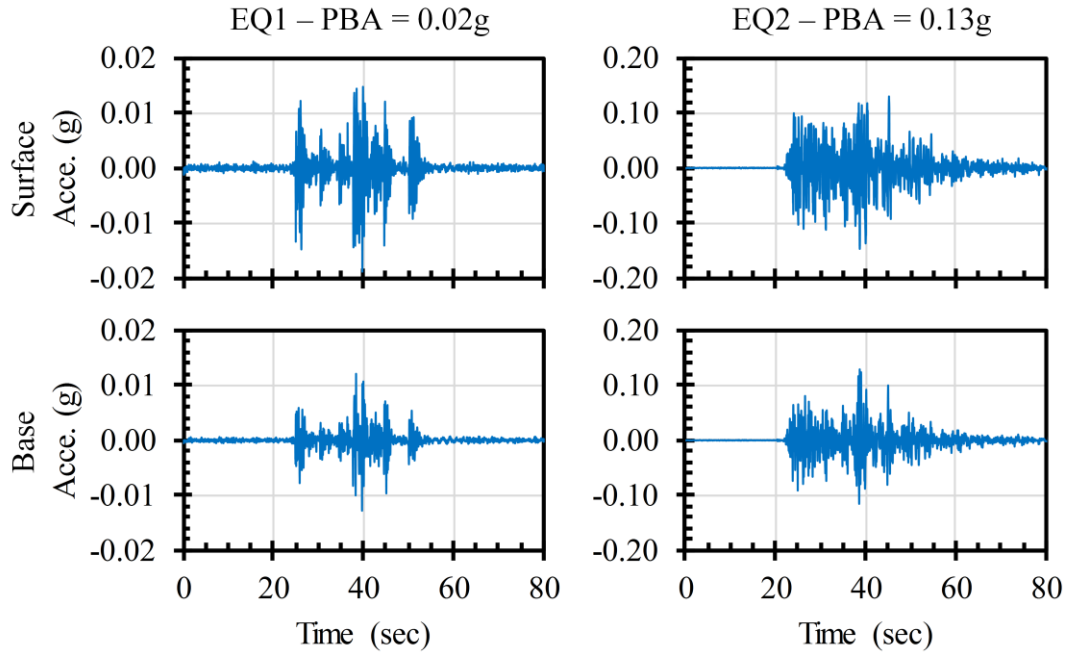


Figure 4.9: Acceleration recordings at top surface and base of the centrifuge model for Chi-Chi motion with 0.02g and 0.13g peak base accelerations.

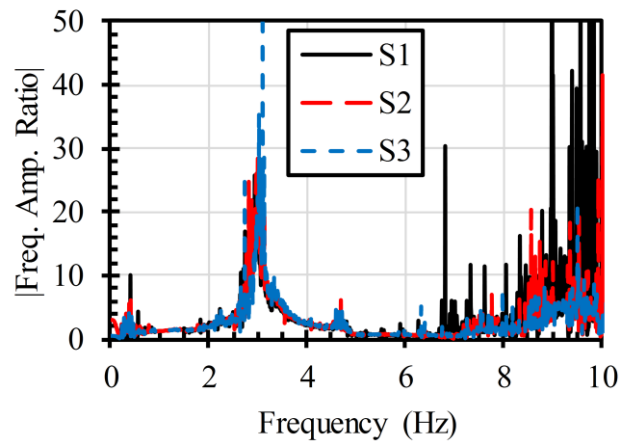


Figure 4.10: Transfer function of centrifuge model with sine sweep input motions.

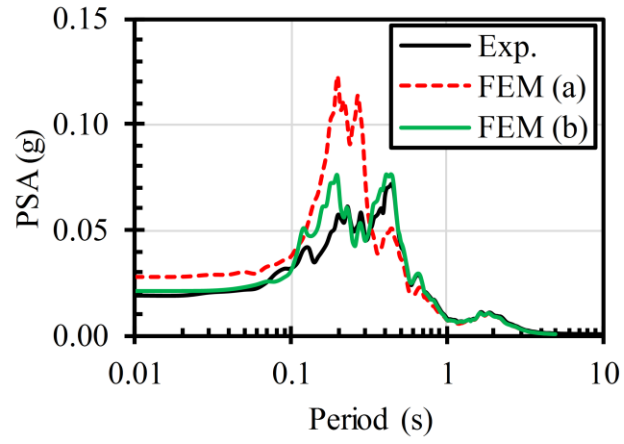


Figure 4.11: Predicted PSA at top soil surface with (a) literature suggested parameters, and (b) calibrated parameters versus PSA from experiments.

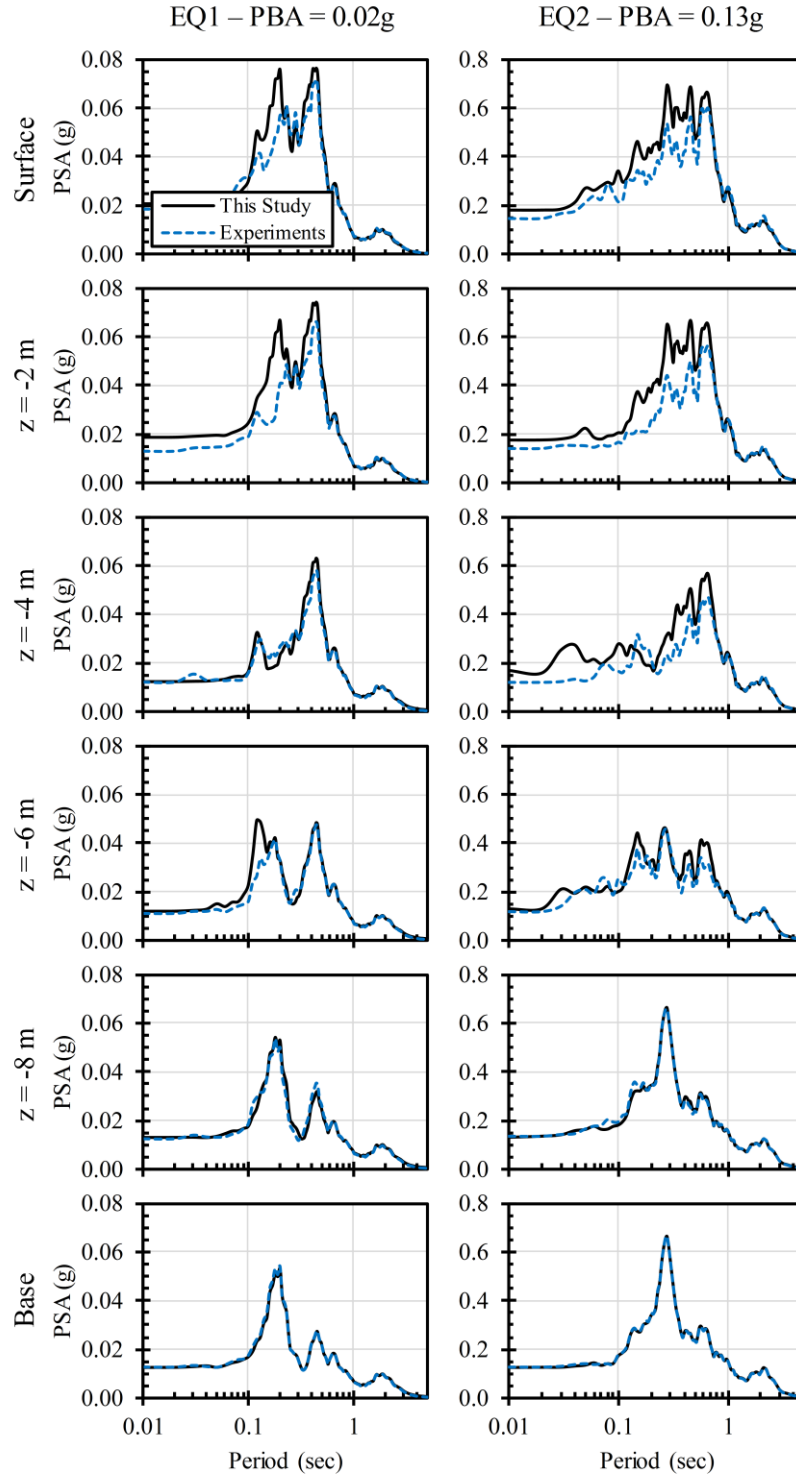


Figure 4.12: FE simulated pseudo spectral accelerations vs. centrifuge test recordings throughout soil profile for Chi-Chi motion with peak accelerations of 0.02g (EQ1) and 0.13g (EQ2).

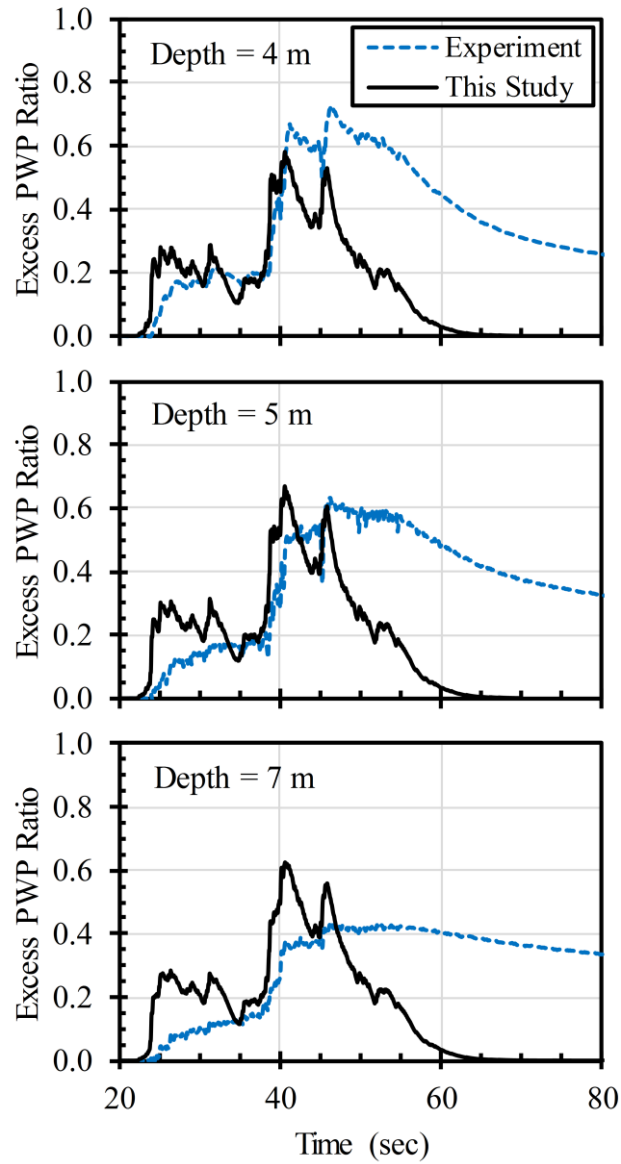


Figure 4.13: FE simulated excess PWP vs. centrifuge test recordings in Nevada sand layer for Chi-Chi motion with peak acceleration of 0.13g (EQ2).

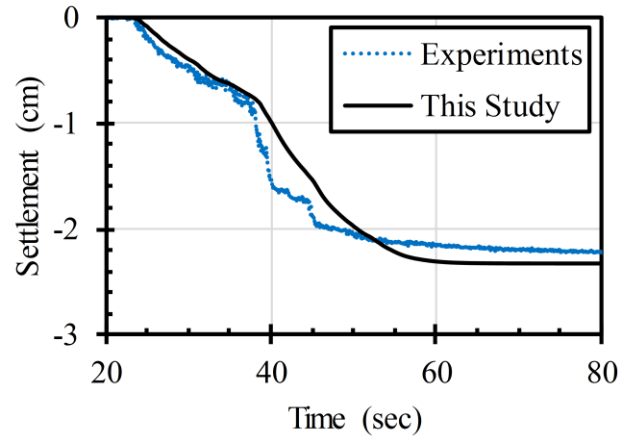


Figure 4.14: FE simulated settlement vs. centrifuge test recordings on top soil surface for Chi-Chi motion with peak accelerations of 0.13g (EQ2).

Chapter 5

Numerical Modeling of Columnar-Reinforced Ground Behavior during Dynamic Centrifuge Testing

The following manuscript has been submitted to the ASCE conference of Geotechnical Frontiers 2017 (with permission from ASCE).

Soheil Kamalzare made the following contributions to this manuscript:

- Developed the finite element models for free-field and reinforced profiles.
- Calibrated the numerical models with laboratory results.
- Performed simulations against recordings in the free-field and reinforced centrifuge tests.
- Evaluated the effect of reinforcements on seismic performance of the loose ground.

Dr. Olgun made the following contributions to this manuscript:

- Supervised the study, and provided valuable feedback.
- Suggested several approaches to identify the mechanism governing the seismic behavior of the reinforced ground.

Numerical Modeling of Columnar-Reinforced Ground Behavior during Dynamic Centrifuge Testing

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5.1 Abstract

Predicting the response of soil profiles during earthquakes is one of the major challenges in geotechnical earthquake engineering. The presence of reinforcing elements such as stiff columns adds further complexity to the problem due to the interaction of these stiff elements with the surrounding ground. This research presents the results of advanced numerical simulations of dynamic centrifuge tests performed on a columnar reinforced model with a loose sandy profile. The model was subjected to earthquake base motions of varying intensities to investigate the reinforcing mechanisms of soil-cement columns. Numerical simulations were performed using the finite element computational platform OpenSees with pressure dependent multi yield (PDMY02) constitutive model. Simulated and measured values were compared for seismic intensity, excess pore water pressure and ground settlement at different locations within soil profile. The calibrated numerical model was able to realistically predict the response of reinforced ground.

5.2 Introduction

Deep Soil Mixing (DSM) is a ground improvement technique whereby soil is mechanically mixed with cementitious materials to make columns of uniform diameter (Mitchell 2008). Cemented soil material generally has a higher strength, lower permeability and lower compressibility than the native soil, although total unit weight may be less (Bruce 2000). Extensive amount of research has been contributed to different aspects of DSM approach (e.g. Kitazume and Terashi 2013, Adalier et al. 1998, and Baez 1995). This has resulted in better understanding of

reinforcing mechanism of soil-cement columns and more efficient techniques for constructing them.

In this study, we have evaluated the capability of numerical modeling in capturing the response of a columnar reinforced ground. For this purpose, at first, we calibrated a nonlinear Finite Element (FE) model with results from unreinforced free-field centrifuge tests. Thereafter, the calibrated model was implemented to predict the response of a dynamic centrifuge test with columnar reinforcements.

We compared numerical predictions with recordings from centrifuge tests for Pore Water Pressure (PWP) generation, surface settlement and seismic wave propagation throughout soil profile. Numerical simulations were able to closely capture the experimental results and were consistent with literature on seismic behavior of reinforced loose soil profiles.

5.3 Centrifuge Tests

Centrifuge testing was performed by Rayamajhi et al. (2015) at NEES @ UC Davis, Center for Geotechnical Modeling. In this study, we looked into two models with free-field and columnar reinforced profiles. Tests were performed under 50g centrifugal acceleration and model prototype dimensions were 20 m $\times$ 10 m $\times$ 10 m (length, width, height). Standard scaling laws were followed when transferring model dimensions to prototype dimensions (Kutter 1995). Model consisted of soil profile with 2 m dense Monterey sand ($D_r \approx 60\%$), underlain by 6 m loose Nevada sand ($D_r \approx 35\%$), and underlain by 2 m rigid cemented base. Soil was saturated 1.5 m below ground surface with methyl-cellulose and de-ionized water. The fluid was 30 times more viscous than water to more properly represent pore fluid diffusion and dynamic response as discussed later in the paper. The model was built in a hinge plate container developed by Fiegel et al. (1994) with negligible friction for lateral movement to represent an infinite free-field soil profile. Chi-Chi (1999) earthquake motion recordings at station TCU078E was scaled to different amplitudes from 0.02g to 0.79g and applied to the base of the model in the longitudinal direction. In between each earthquake excitation, a small amplitude sine-sweep was applied to the model to test the elastic response of site profile and evaluate effects of earthquake shakings. The sine sweeps were small

enough to not generate notable excess PWP nor cause settlement. Instrumentations included accelerometers and PWP transducers throughout the profile with settlement transducers on surface.

Figure 5.1 shows the reinforced model profile. Reinforcements were rigidly fixed within cemented base, and had an area replacement ratio of $\sim 31\%$. Soil-cement columns were constructed by mixing ordinary Portland cement with Nevada sand to reach 1 MPa unconfined compressive strength in seven days. Average shear modulus of columns were estimated to be 174 MPa. Detailed information about these experiments can be found in Rayamajhi et al. (2015).

5.4 Numerical Model

Numerical modeling was carried out using OpenSees finite element program. OpenSees is an open-source program capable of simulating response of structural and geotechnical systems under static and dynamic loads (Mazzoni et al. 2006). The focus of numerical modeling was put on excess PWP generation, settlements and seismic wave propagation throughout soil profile. We performed 2D analyses for free-field profile since the model was homogenous in the horizontal direction and had a simple shear dominated behavior. Reinforced profile was modeled in 3D analyses to better capture out-of-plane effects of reinforcements and PWP dissipation.

Pressure-Dependent-Multi-Yield-02 (PDMY02) constitutive model by Yang and Elgamal (2008) was used to represent soil behavior. PDMY02 incorporates various features to realistically predict soil dynamic response, such as, multiple yield surfaces to capture nonlinear soil behavior, phase transformation surface to properly predict soil volumetric behavior, pressure dependent shear modulus (G) which is continuously updated during the simulations to better predict soil strength, and fabric evolution to enhance contractiveness on reversal of load after dilative phases.

For soil-cement material, we used Pressure-Independent-Multi-Yield (PIMY) constitutive model by Yang et al. (2003). PIMY is an elasto-plastic material whose shear behavior is insensitive to the confinement pressure. It uses nested Von Mises yield surfaces and an associative flow rule. Plasticity is exhibited in the deviatoric stress-strain response, while the volumetric stress-strain response is linear-elastic and independent of the deviatoric response (Mazzoni et al. 2006). This is consistent to our expectations for the soil-cement columns as their stiffness was largely greater than the surrounding soils and their behavior was predominantly elastic. In our simulations we

used calibrated parameters suggested by Khosravifar (2012) with further calibrations as discussed later in the paper for both PDMY02 and PIMY materials.

In terms of FE numerical aspects, convergence tolerance was set to $1e-5$ on the norm of the displacement residuals to reduce high frequency noise and more properly simulate small strain responses. We were not able to manipulate displacement matrix to enforce constraints since some of the nodes had multiple boundary conditions (e.g. periodic boundary conditions with zero PWP). Therefore, we used penalties with $1e18$ enforcements on the stiffness matrix to define constraints. PDMY02 provided the model with proper material hysteresis damping. Therefore, we used only 1% Rayleigh damping proportional to mass and initial stiffness matrices with corners on first and second natural frequencies of the centrifuge profile to minimize convergence problems at lower levels of shaking. Newmark integration method with $\gamma = 1.5$ - $\beta = 1.0$ was used in gravity analyses to extensively damp out oscillations from impacts of weight loads. Values of $\gamma = 0.6$ - $\beta = 0.3$ were used for dynamic analyses to expedite computations and help damping out high frequency noises. Before and after all these numerical assignments, transfer function of the model was evaluated to avoid any detrimental numerical effects.

5.5 Model Calibration

Results from free-field centrifuge tests were used to calibrate the parameters of constitutive models. We developed two 2D FE models, one with full scale $20\text{ m} \times 10\text{ m}$ mesh, and one with single column $0.5\text{ m} \times 10\text{ m}$ mesh. The preliminary studies showed that these models return almost the same results with negligible differences. This was in agreement to our expectations as the centrifuge model had a simple shear dominated behavior similar to a laterally infinite site. Moreover, Afacan et al. (2013) showed that hinge plate containers impose minimal effects on soil models. They compared recordings from sensors located on soil and on container, and found almost the same measurements. Therefore, we decided to continue our calibration by utilizing the smaller mesh for soil profile to have the advantage of faster computation speed.

5.5.1 Small Strain Calibration

Soil small strain parameters are controlled by elastic properties. To evaluate these properties, we studied centrifuge model response to small amplitude sine sweeps. These motions had $0.02g$

Peak Base Accelerations (PBA), and were not able to generate notable PWP nor deformations in the model. Soil behavior was predominantly elastic, and therefore, first natural frequency (f_1) of the model can be approximated as $f_1 \approx 0.25V_s/H$. Value of f_1 can provide us a sense on site shear wave velocity (V_s), and through that, an estimation of shear modulus since $V_s = (G/\rho)^{0.5}$. To find f_1 , acceleration recordings at base and on top of the model were converted to frequency domain, and model transfer function was calculated. Results for four sine-sweeps, which were applied to the model in between earthquakes are presented in Figure 5.2. These graphs show two important points: (1) model f_1 was approximately 3 Hz, and (2) the stronger shakings in between these sine sweeps were not able to change f_1 . Therefore, numerical elastic properties must be calibrated to have $f_1 \approx 3$ Hz, and they should be kept the same for all simulations.

Using suggested parameters by Khosravifar (2012) in FE analyses, f_1 was simulated as 3.72 Hz. This resulted inaccurate predictions for PSA on top surface. Therefore, recognizing Nevada sand layer as the dominant soil in the profile with largest thickness and loosest relative density, we reduced its shear modulus to 60% of suggested values to have $f_1 \approx 3$ Hz like experiments. Figure 5.3 shows FE prediction for surface PSA with suggested and calibrated parameters against measurements during Chi-Chi motion with peak base acceleration 0.02g. As it can be seen, suggested parameters resulted almost two times greater peak PSAs and in different periods from experiments. Favorably, calibrated parameters closely captured peak PSAs and their periods. Therefore, for rest of the simulations we used the calibrated shear modulus.

5.5.2 Large Strain Calibration

Soil volumetric behavior is represented by constitutive model. PDMY02 has six parameters to simulate volumetric behavior. Three parameters for contraction phase and three parameters for dilation phase to control volume change rate, pore water pressure generation, fabric damage and overburden stress effects on volumetric behavior (Khosravifar 2012). In essence these parameters control coefficient of volume compressibility (m_v) in equation $c_v = k/(m_v\gamma_w)$. Where c_v is coefficient of consolidation, k is permeability and γ_w is unit weight of water. The parameter c_v is the key parameter controlling volume change and pore water pressure generation in FE simulations. To calibrate c_v , we either need to calibrate the six parameters for m_v or adjust k to result a proper c_v . Calibrating m_v requires a set of laboratory tests to find best fitting values, thus, it is more convenient to calibrate k . Permeability, itself, plays an important role in determining the

liquefaction response of homogenous sand deposits, affecting especially the depth of liquefaction, the speed at which excess PWP dissipates after shaking, and the character of the ground surface settlement (Sharp et al. 2003). Therefore, calibrating it requires great attention to evaluate gains and losses in simulations.

The Nevada sand used in centrifuge model is a relatively uniform clean fine sand. It has a specific gravity (G_s) of 2.64, a mean grain size (D_{50}) of 0.17 mm (model scale), a uniformity coefficient (C_u) of 1.64, and a permeability of 0.002 cm/s (Kamai et al. 2008 and Arulmoli et al. 1992). For a medium sand, typical values of m_v are generally on the order of $4e-5$ to $10e-5$ (1/kPa) (Pestana et al. 1997), and vary based on relative density, confining pressure, and the peak excess pressure in the soil (Lee and Albaisa 1974). Howell (2013) comprehensively studied PDMY02 model and found out that the equivalent m_v values modeled by PDMY02 are 1/10 to 1/3 smaller than those measured in experiments. Therefore, permeability of FE model must be set to 0.002/10 cm/s to 0.002/3 cm/s to include the effects of m_v through permeability.

As it was mentioned before, the tests were performed under 50g centrifugal acceleration with a fluid 30 times more viscous than water. A model saturated with water and tested at N g would have permeability N times greater than the same soil in prototype saturated with water and tested at 1g (Kutter 1995). Therefore, the permeability of soil was 1.67 ($=50/30$) times higher than actual values, resulting in 0.0003 cm/s ($=0.002/10 \times 1.67$) to 0.0011 cm/s ($=0.002/3 \times 1.67$). Seed et al. (1975) suggested that for a loose sand ($D_r = 30\%$ to 40%), m_v can increase by as much as a factor of 10 as liquefaction is approached. Since liquefaction triggered very fast during the tests, and PDMY02 maintains a relatively constant permeability during simulations, the permeability of numerical model should at most be set to 0.003 cm/s to 0.011 cm/s. Based on these, we chose the permeability range of 0.0003 cm/s to 0.01 cm/s and performed simulations to find closest PWP generation and settlement to experiments. Our model performed best with permeability 0.01 cm/s.

Top and bottom Monterey sands were built as drainage boundaries for Nevada sand layer. The top layer was almost dry (except 0.5 m) and did not experience liquefaction. The bottom layer was very dense ($D_r = 90\%$) and did not experience liquefaction either. None of these layers deformed considerably relative to loose Nevada sand layer during all the excitations. We chose permeability of 0.05 cm/s for Monterey sand layers to have similar PWP distribution to recordings.

After calibrating permeability of layers and ensuring close capture of PWP recordings, we calibrated PDMY02 contraction rate parameter (c_1) of Nevada sand to better capture settlements. Khosravifar (2012) has suggested $c_1 = 0.06$. We chose $c_1 = 0.08$ to have more contractive behavior in Nevada sand layer and better replicate settlements.

Figure 5.4 shows two simulations with permeability of 0.01 cm/s and 0.001 cm/s. It can be seen that changing water diffusion in the numerical model can extensively affect trend and amount of PWP generation and settlement. Simulation with $k = 0.001$ cm/s showed almost two times more excess PWP and half settlements comparing to recordings in experiment. Whereas, simulation with $k = 0.01$ cm/s was able to predict better trend and values for both PWP generation and settlement.

In our simulations, similar to Howell (2013), PWP dissipated immediately after motion intensity decreased to small amplitudes. This was in contrast with recordings which show limited PWP dissipations even after motion stopped. We expect to observe settlements when PWP dissipates. On the contrary, during the experiments settlements occurred while the PWP was building up. The reason why this behavior is monitored or why there is still a considerable amount of PWP in the model even long after the motion stopped, is not clear at this point. There is a possibility of sensors malfunctioning against the saturation viscous fluid used in the experiments. More study is required to fully understand the observed behavior.

5.6 Simulation of Soil Response in the Reinforced Profile

We developed a 3D nonlinear finite element model to simulate soil response in the reinforced profile. The 3D set-up was to more properly include effects of columns on surrounding soil, PWP dissipation and surface settlements. We used OpenSeesPL program by Lu et al. (2010) as graphical pre- and post-processing program to generate mesh and interpret the simulated response. Owing to symmetry and to simplify computations, we developed a model for only one of the columns and its surrounding soil. The modeled area is highlighted in Figure 5.1 and the numerical model is shown in Figure 5.5. We did not model the cemented base, because it was significantly stiffer than the overlying soils and acted as a rigid boundary.

Ground motion was applied to the model in the longitudinal direction. Side nodes above base at $x = -1.4$ m and 1.4 m were tied together for longitudinal and vertical displacements, and side

nodes at $y = 0$ m and 1.4 m were fixed against transverse displacements to enforce shear beam mechanism. Base nodes were constrained against movements in all directions. FE numerical aspects were assigned like 2D analyses with only difference of adjusting Rayleigh damping corners to target first and second natural frequencies of the reinforced profile.

We performed simulations with peak base excitations of 0.06g and 0.11g. The recorded motions on top of the cemented base and at surface of the model are shown in Figure 5.6. The recordings on top of the cemented base were applied to the base of numerical model.

We used calibrated soil parameters from free-field studies for reinforced profile analyses since the same soils were used in both tests. Reinforcement materials were calibrated following same procedure as discussed above by studying the transfer function of model and matching the natural frequencies. Our simulations showed that only 50% of estimated value (≈ 87 MPa) should be used to have same f_1 in numerical model and experiment. For large strain calibrations, we increased permeability of all sands by two times to better capture recordings for PWP and surface settlements.

5.6.1 Reinforced Profile Acceleration Response

Computed and measured PSAs throughout reinforced profile are shown in Figure 5.7. As seen by comparison, FE model was able to fairly accurately predict peaks and their periods and the general trend of PSAs at all depths. The discrepancies increased with stronger earthquake and with wave upward propagation specially within periods 0.1 to 0.2 seconds. Still, the errors were insignificant with almost always being smaller than 0.1g difference. The centrifuge recordings were consistent with Idriss (1990) recommendations. These motions with amplitudes less than 0.4g amplified while passing through loose soil profile. Numerical model was able to closely capture this phenomenon.

5.6.2 Reinforced Profile PWP and Settlement Responses

Computed and measured PWP at the middle of Nevada sand layer and surface settlement are shown in Figure 5.8. Similar to free-field results PWP dissipated faster in FE simulations than recordings. In general, FE simulations were able to fairly predict PWP generation and peaks during both motions.

In terms of settlements, FE simulations were able to capture the response with less than 0.5 cm difference during both motions. However, the trends of settlements were considerably different between experiments and simulations. In experiments majority of settlements occurred as PWP was building up, whereas in numerical simulations majority of settlements occurred after developed PWP started to dissipate. This discrepancy can be due to a difference in permeability and dissipation regimes between numerical model and experiment. While the total values of settlements are independent of permeability, their developments are much dependent on permeability. In the similar tests by Sharp et al. (2003), low permeability sands showed little settlements during shaking, with most of total settlement a long period after shaking. However, high permeability sands showed most of settlement take place during shaking. This might be the reason behind the deviation in settlement rates herein. Probably, centrifugal scaling effects (i.e. small sand particles finding 50 times larger dimensions), and as suggested by Rayamajhi et al. (2015), the formation of preferential seepage paths along sides of the columns have facilitated PWP dissipation during the tests causing the settlements occur during the shaking.

5.7 Summary

In this study, we evaluated the capability of finite element numerical modeling in predicting the response of a columnar reinforced ground during dynamic centrifuge testing. For this purpose, we developed a 2D FE model incorporated with a nonlinear multi-surface constitutive model, and calibrated it with results from free-field centrifuge tests. Thereafter, we implemented the calibrated parameters in a 3D FE model to predict response of a columnar reinforced ground. Simulations and measured behavior for generated PWP, surface settlement and accelerations throughout profile were compared. In total, results showed that carefully calibrated numerical model can provide realistic predictions for complex behavior of a reinforced ground under seismic excitations with different intensities.

5.8 Acknowledgements

Funding for this study was provided by the National Science Foundation for the project “NEESR: Reduction of Seismic Shaking Intensity on Soft Soil Sites Using Stiff Ground

Reinforcement” under award CMMI 1208117. Any opinions, conclusions or recommendations expressed herein are those of the authors and do not necessarily reflect the views of the National Science Foundation.

5.9 References

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5.10 Figures

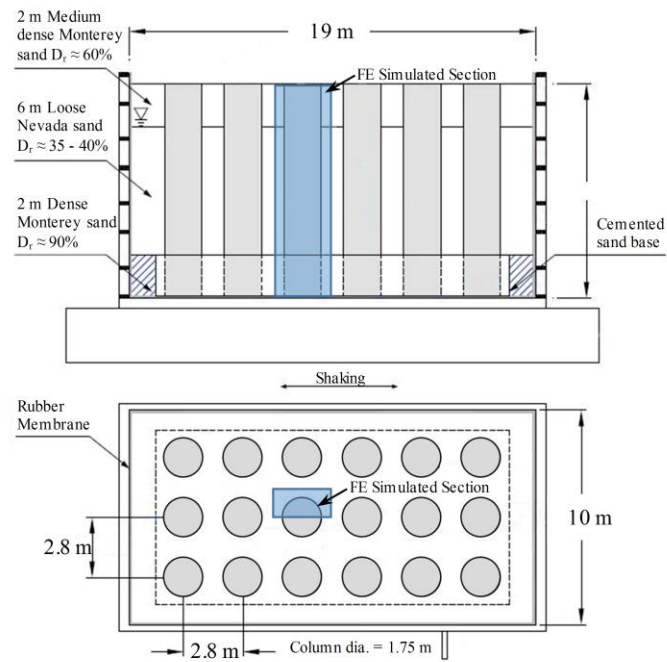


Figure 5.1: Centrifuge model configuration for reinforced profile (after Rayamajhi et al. 2015)

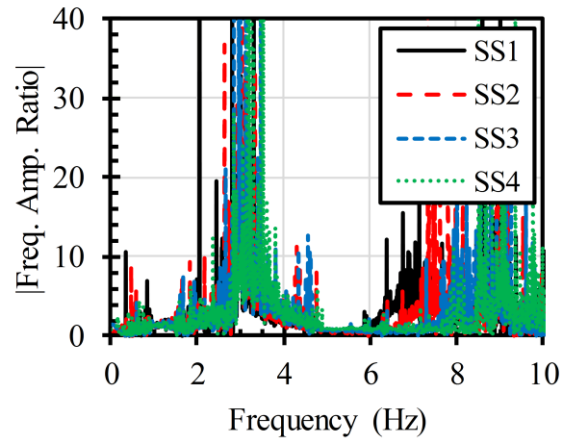


Figure 5.2: Transfer functions of the free-field centrifuge model with sine sweep input motions

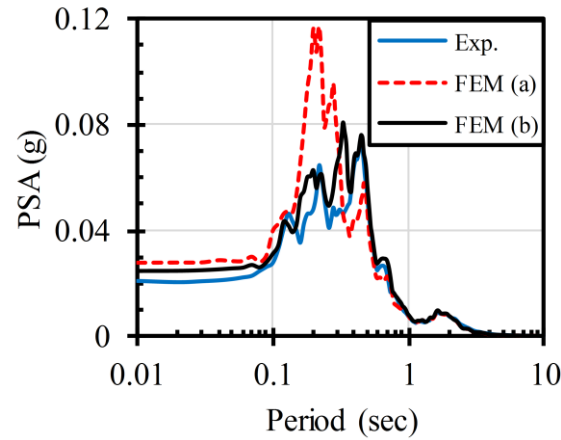


Figure 5.3: Predicted PSA at top soil surface with (a) literature suggested, and (b) calibrated parameters in comparison with PSA from experiments

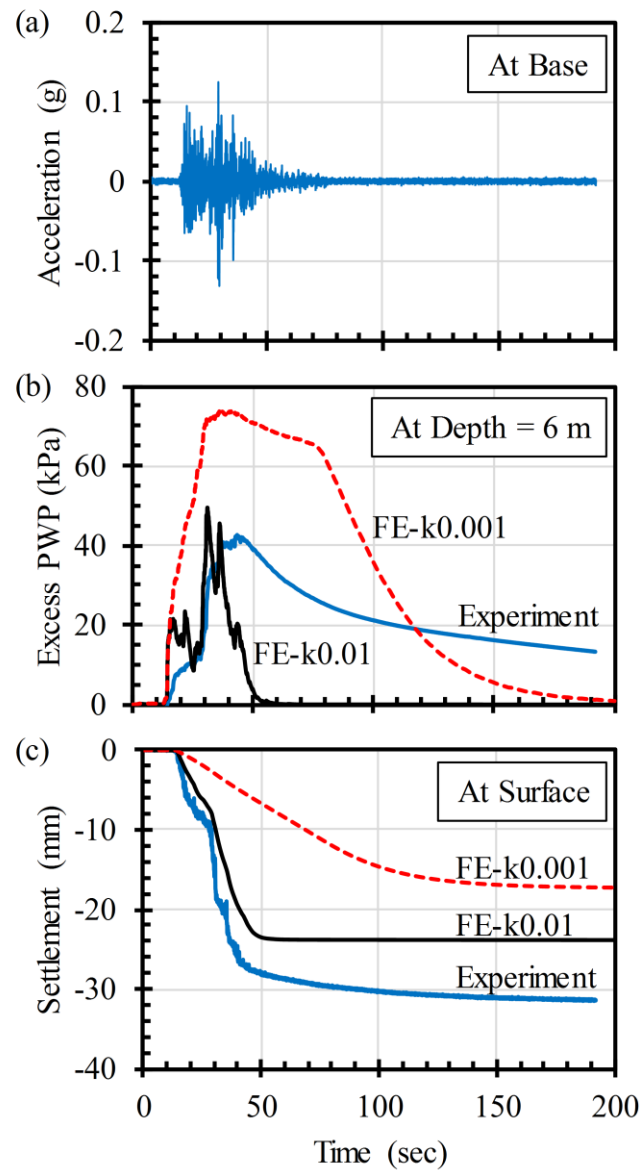


Figure 5.4: Comparison of dynamic centrifuge test results and numerical simulations (a) ground motion applied at the base of the model, and comparison of measured and simulated cases with permeability values of 0.01 cm/s and 0.001 cm/s (b) excess PWP and (c) surface settlement.

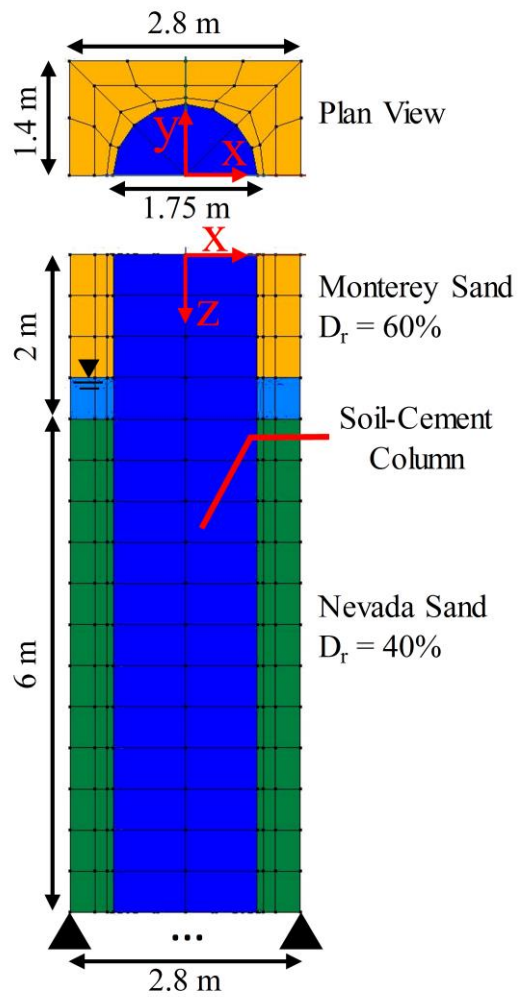


Figure 5.5: FE model developed to study the centrifuge test with columnar reinforcements

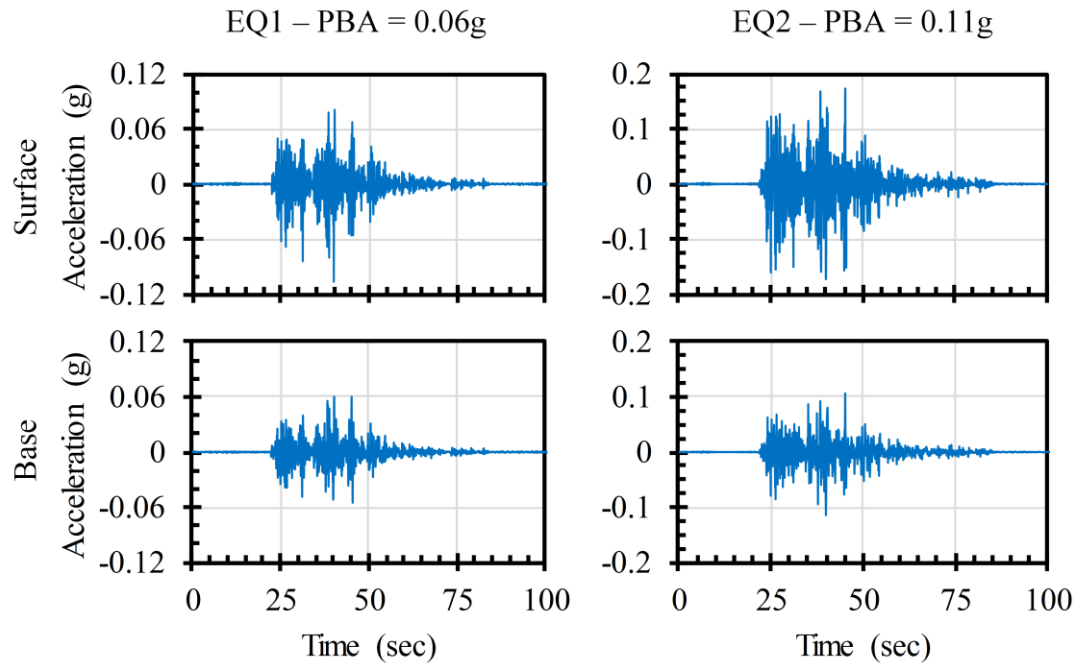


Figure 5.6: Recorded accelerations on top of the soil-cemented base and at surface of the reinforced centrifuge model

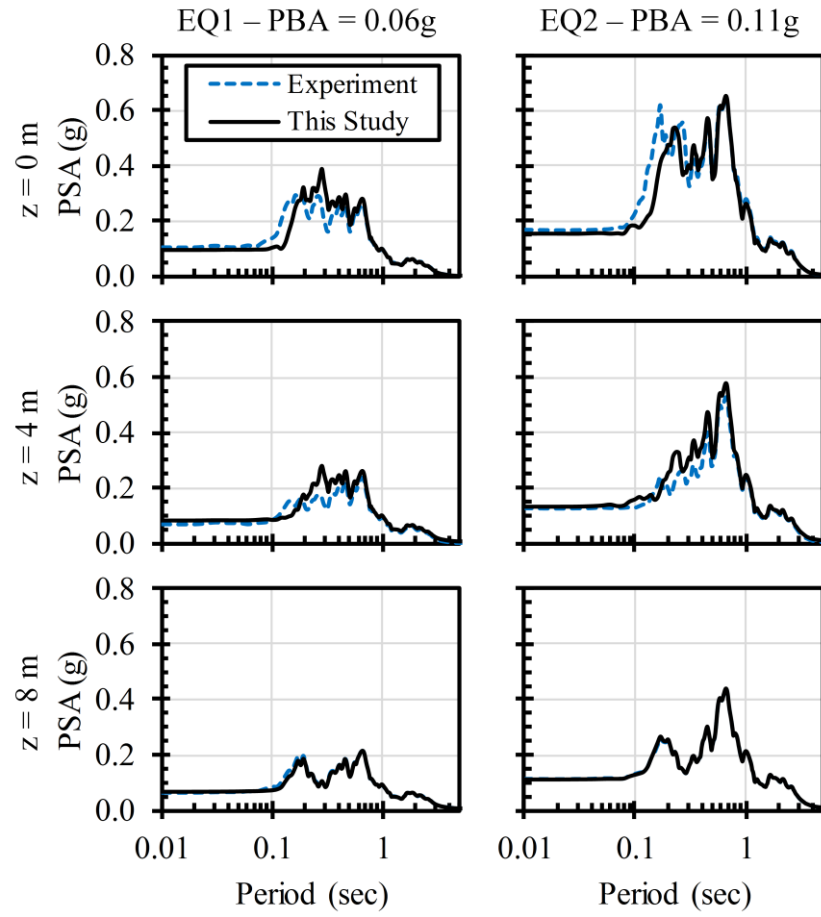


Figure 5.7: FE simulated pseudo spectral accelerations vs. centrifuge recordings

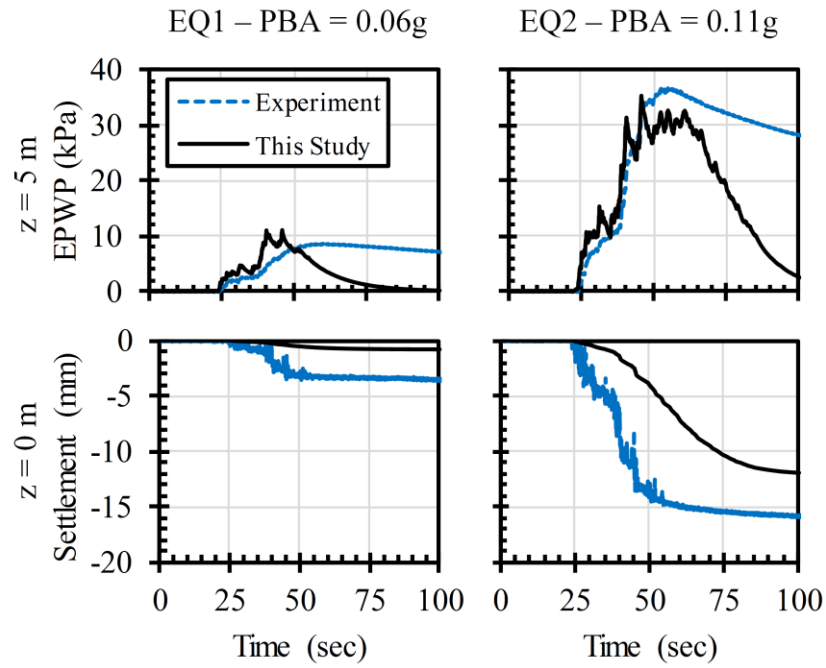


Figure 5.8: FE simulated excess PWP and surface settlement vs. centrifuge recordings

Chapter 6

Seismic Behavior of Columnar-Reinforced Ground: Centrifuge Testing and Numerical Modeling

The authors of the following manuscript intend to submit it to the ASCE Journal of Geotechnical and Geoenvironmental Engineering.

Soheil Kamalzare made the following contributions to this manuscript:

- Developed the finite element models for the free-field and reinforced profiles.
- Calibrated the numerical models with the laboratory results.
- Performed simulations against recordings in the free-field and reinforced centrifuge tests.
- Evaluated the effect of reinforcements on seismic performance of the loose ground.

Dr. Olgun made the following contributions to this manuscript:

- Supervised the study, and provided valuable feedback.
- Suggested the underlying ideas on the purpose, scope and depth of the study.
- Suggested several approaches to identify the mechanism governing the seismic behavior of the reinforced ground.

Seismic Behavior of Columnar-Reinforced Ground: Centrifuge Testing and Numerical Modeling

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6.1 Abstract

Deep soil mixing to construct stiff columns is one of the methods used today to improve performance of loose grounds and remediate liquefaction problems. Stiff elements can reduce the deformation of surrounding soils during earthquakes and provide a firm support underneath foundations. However, they can also adversely affect motion intensity passing through these profiles. This research focuses on seismic performance of columnar reinforcements and investigates their effect on pore water pressure generation, surface settlements and seismic intensity in loose sandy profiles. For this purpose, at first, a nonlinear finite element model was calibrated with results from an unreinforced dynamic centrifuge test, and thereafter, the calibrated model was utilized to predict the behavior of a centrifuge test with essentially the same profile but reinforced with soil-cement columns during base excitations with different intensities and liquefaction triggering. Simulations were compared with recordings in both unreinforced and reinforced tests and the observed behavior was discussed. Overall, numerical simulations were consistent with experiments and aligned with findings in literature. Results showed that columnar reinforcement were rather inefficient in reducing pore water pressure generation and preventing liquefaction triggering. Soil settled more in between columns while the columns stayed steady with little settlements, possibly, providing a firm base for above structures. Columns prevented the de-amplification of waves passing through and more seismic intensity was observed at soil surface.

6.2 Introduction

Deep Soil Mixing (DSM) is a ground improvement technique whereby soil is mechanically mixed with cementitious materials to make columns of uniform diameter (Mitchell 2008). Cemented soil material generally has a higher strength, lower permeability and lower compressibility than the native soil, although total unit weight may be less (Bruce 2000). Constructing soil-cement columns is relatively fast, with minimal ground vibration and noise pollution. Material strength can be conveniently calibrated by varying the ratio of solidifying material. The method gained reliability after new advancements in mixing equipment, alignment control and integrated systems for real-time monitoring during installation. Moreover, it does not require any particular material such as gravels to be imported to the site especially in areas with remote access (Porbaha 1998). All these and many other advantageous have made DSM an attractive ground improvement method particularly in urban constructions on soft soil profiles.

Over the last five decades, extensive amount of research has been contributed to different aspects of DSM approach (e.g. Kitazume and Terashi 2013, Adalier et al. 1998, and Baez 1995). This has resulted in better understanding of reinforcing mechanism of soil-cement columns and more efficient techniques for constructing them. However, there has been somewhat controversial debates about cyclic performance of columnar reinforcements. It is often assumed that columns and surrounding soils have shear strain compatible deformations (Baez and Martin 1993 and Durgunoglu 2006). If pure shear beam behavior was occurring, the shear stresses in stiff column relative to the soil mass would be in proportion to the column-to-soil stiffness ratio. In other words, if the column is 100 times stiffer than the surrounding soil, it will absorb 100 times more shear stress than surrounding soil during pure shear behavior. Therefore, columns can play an important role in reducing seismic intensity in site and prevent liquefaction triggering.

Yet, theoretical analyses (e.g. Goughner and Pestana 1998) and numerical simulations (e.g. Olgun and Martin 2008, Green et al. 2008, Rayamajhi et al. 2012 and 2014) suggest that discrete columns deform in combination of both shear and flexure during seismic loading, such that, columns carry substantially smaller amounts of shear stress. Therefore, they might not be beneficial as expected from previous studies in reducing seismic intensity and mitigating liquefaction.

In this study, we have performed numerical modeling to evaluate seismic performance of DSM columnar reinforcements. For this purpose, at first, nonlinear Finite Element (FE) models were calibrated with results from unreinforced free-field centrifuge tests. Thereafter, the calibrated models were employed to simulate soil response during centrifuge testing of a columnar reinforced ground. We compared numerical predictions with experimental results for Pore Water Pressure (PWP) generation, liquefaction triggering, surface settlement and seismic wave propagation throughout soil profile. Numerical simulations were able to capture experimental results and were consistent with literature on seismic behavior of loose soil profiles. In total, seismic intensity increased in reinforced profile because columns reduced system nonlinearity and softening. Reinforced profile had similar PWP generation to free-field and columns were not able to prevent liquefaction. Although settlements were smaller on columns, they increased on soil surface in between columns.

6.3 Centrifuge Tests

Centrifuge testing was performed by Rayamajhi et al. (2014) at NEES @ UC Davis, Center for Geotechnical Modeling. In this study, we looked into two models with free-field and columnar reinforced profiles. Tests were performed under 50g centrifugal acceleration and model prototype dimensions were 20 m $\times$ 10 m $\times$ 10 m (length, width, height). Standard scaling laws were followed when transferring model dimensions to prototype dimensions (Kutter 1992 and Wood 2004). The model consisted of soil profile with 2 m dense Monterey sand ($D_r \approx 60\%$), underlain by 6 m loose Nevada sand ($D_r \approx 35\%$), and underlain by 2 m very dense Monterey sand ($D_r \approx 90\%$). Soil was saturated 1.5 m below ground surface with methyl-cellulose and de-ionized water. The fluid was 30 times more viscous than water to more properly represent pore fluid diffusion and dynamic response as discussed later in the paper. The model was built in a hinge plate container developed by Fiegel et al. (1994) with negligible friction for lateral movement to represent an infinite free-field soil profile. Chi-Chi (1999) earthquake motion recording at station TCU078E was scaled to different amplitudes from 0.02g to 0.74g and applied to the base of the model in the longitudinal direction. In between each earthquake excitation, a small amplitude sine-sweep was applied to the model to test the elastic response of site profile and evaluate effects of earthquake shakings. The sine sweeps were small enough to not generate notable excess PWP or settlement. Instrumentations

included accelerometers and PWP transducers throughout the profile with settlement transducers on surface.

Reinforcements were in form of 3×6 columns extended from surface to bottom of Nevada sand layer. Reinforced profile is shown in Figure 6.1. Columns had 1.75 m diameter with 2.8 m spacing in rectangular grid setting. Soil-cement columns were constructed by mixing ordinary Portland cement with 10% by weight Nevada sand at water-cement ratio of 2.6. Several trials were carried out to achieve 1 MPa unconfined compressive strength in seven days. Average shear modulus of columns were estimated as 116 MPa. In further studies performed by Rayamajhi et al. (2015), they increased this value by 50% (≈ 174 MPa) to account for limitations of unconfined compression tests. Detailed information about these experiments can be found in Rayamajhi et al. (2014 and 2015).

6.4 Numerical Model

Numerical modeling was carried out using OpenSees (<http://opensees.berkeley.edu>) finite element program with pre- and post-processing by customized commercial software GiD. OpenSees is an open-source program capable of simulating response of structural and geotechnical systems under static and dynamic loads (Mazzoni et al. 2006). The focus of numerical modeling was put on key observations from centrifuge tests as pointed out by Kamai and Boulanger (2013): (1) the dynamic response and onset of liquefaction, (2) the amount and pattern of surface deformations, (3) the pattern of pore pressure dissipation and void ratio redistribution, and (4) the difference in response between free-field and reinforced profiles. We performed 2D analyses for free-field profile since the model was homogenous in the horizontal direction and had a simple shear dominated behavior. Reinforced profile was modeled in 3D analyses to better capture out-of-plane effects of reinforcements and PWP dissipation.

Pressure-Dependent-Multi-Yield-02 (PDMY02) constitutive model by Yang and Elgamal (2008) was used to represent soil behavior. PDMY02 incorporates various features to more realistically predict soil dynamic response, such as, multiple yield surfaces to capture nonlinear soil behavior, phase transformation surface to properly predict soil volumetric behavior, pressure dependent shear modulus (G) which is continuously updated during the simulations to more

realistically predict soil strength (Prevost 1985), and fabric evolution to enhance contractiveness on reversal of load after dilative phases (see Dafalias and Manzari 2004 for a detailed discussion).

For soil-cement material, we used Pressure-Independent-Multi-Yield (PIMY) constitutive model by Yang et al. (2003). PIMY is an elasto-plastic material whose shear behavior is insensitive to the confinement pressure. It uses nested Von Mises yield surfaces and an associative flow rule. Plasticity is exhibited in the deviatoric stress-strain response, while the volumetric stress-strain response is linear-elastic and independent of the deviatoric response (Mazzoni et al. 2006). This is consistent to our expectations for soil-cement columns as their stiffness was largely greater than surrounding soils and their behavior was predominantly elastic. In our simulations we used calibrated parameters suggested by Khosravifar (2012) with further calibrations as discussed later for both PDMY02 and PIMY materials.

In terms of FE numerical aspects, we followed the recommendations by Kamalzare and Olgun (2017). Convergence tolerance was set to $1e-5$ on norm displacement residuals to reduce high frequency noise and more properly simulate small strain responses. We were not able to manipulate displacement matrix to enforce constraints since some of the nodes had multiple boundary conditions (e.g. periodic boundary conditions with zero PWP). Therefore, we used penalties with $1e18$ enforcements on stiffness matrix to define constraints. PDMY02 provides model with proper material hysteresis damping. Therefore, we used only 1% Rayleigh damping proportional to mass and initial stiffness matrices with corners on first and second natural frequencies of the centrifuge profile to minimize convergence problems at lower levels of shaking. Newmark integration method with $\gamma = 1.5$ - $\beta = 1.0$ was used in gravity analyses to extensively damp out oscillations from impacts of weight loads. Values of $\gamma = 0.6$ - $\beta = 0.3$ were used for dynamic analyses to expedite computations and help damping out high frequency noises. Before and after all these numerical assignments, transfer function of the model was evaluated to avoid any detrimental numerical effects.

During the centrifuge tests, 16 different motions (scaled Chi-Chi motion with increasing amplitude and Sine-Sweeps with very small amplitudes) were applied to both models with enough time in between for PWP to dissipate. Kamai and Boulanger (2013) investigated two possible approaches for running simulations of consecutive shaking events: (1) simulate all shakes in sequence with time for dissipation between them (as occurred in centrifuge test itself) or (2)

simulate each shake individually using a newly generated mesh with a best estimate on the geometry and other properties in the beginning of that shaking event. They concluded overall that the two approaches yield comparable results. Moreover, field experiments conducted by Baez and Martin (1992) revealed that when PWP ratio does not exceed 0.7, there will be little noticeable volume change with no improvement in densification. In this study, we simulated model behavior under motions shown in Table 6.1. All these motions except the final ones had amplitudes less than 0.13g and were not able to change the geometry of the model significantly or cause PWP ratios more than 0.7 to considerably densify or loosen the soil. Therefore, we simulated model behavior by using individually newly generated meshes. Acceleration recordings at the base of centrifuge model were directly input to FE model in form of uniform horizontal excitations at base.

6.5 Model Calibration

Results from the free-field centrifuge test were used to calibrate the parameters of constitutive models. The free-field profile with recordings at base and surface of the centrifuge model for one of the Chi-Chi motions are shown in Figure 6.2. We developed two 2D FE models, one with full scale $20\text{ m} \times 10\text{ m}$ mesh, and one with single column $0.5\text{ m} \times 10\text{ m}$ mesh. The preliminary studies showed that these models return almost the same results with negligible differences. This was in agreement to our expectations as the centrifuge model had a simple shear dominated behavior similar to a laterally infinite site. Moreover, Afacan et al. (2013) showed that hinge plate containers impose minimal effects on soil models. They compared recordings from sensors located on soil and on container, and found almost the same measurements. Therefore, we decided to continue our study by utilizing the smaller mesh for soil profile to have the advantage of faster computation speed.

The FE model is also shown in Figure 6.2. Side nodes above base were tied together to have same displacements and force the column to behave like a laterally infinite site with shear beam deformations. The base nodes were constrained against any movements. Soil was modeled using four-node quadUP elements that model dynamic response of solid-fluid fully coupled material based on Biot's theory of porous media. These elements are not appropriate for cases where incompressible behavior is expected due to mesh locking (Oden and Carey 1984). However, mesh locking is not anticipated to have significant effects herein, because soil deformations are

dominated by a simple shear mechanism that is less prone to mesh locking. To ensure ineffectiveness of mesh-locking, we performed simulations with nine-node quadUP elements, and the results were the same.

In FE analyses to represent a traveling wave accurately about 10 nodes per wave length are required. Fewer than 10 nodes can lead to numerical damping as the discretization might miss certain peaks of the wave (Preisig and Jeremic 2005). This requirement is shown in Equation 6.1, where Δh is node spacing, λ is wave length, $V_{s,min}$ is minimum shear wave velocity, and f_{Max} is maximum motion frequency.

$$\Delta h \leq \frac{\lambda}{10} = \frac{V_{s,min}}{10f_{Max}} \quad (6.1)$$

The input motion was refined by a Butterworth band-pass filter of order four with 0.05 Hz and 8 Hz corners. This resulted in considerable frequencies up to 10 Hz. Using Equation 6.1 with the maximum frequency of 10 Hz in input motion, and assuming minimum shear wave velocity of 100 m/s, maximum length of elements will be 1 m. In our simulations, we tried elements with 0.5 m, 0.25 m and 0.1 m lengths, and the results were not considerably different without filtering any relevant frequencies. Therefore, for ease of computation we stayed with element length of 0.5 m. Out-of-plane thickness of soil elements are usually set very large to enforce plane strain conditions. In our study we varied thickness from 1 m to 10 m (centrifuge model width), 100 m and 1000 m. Similar to Chang et al. (2013), there was negligible difference between results, and hence the element thickness was set to 1 m.

6.5.1 Small Strain Calibration

Soil small strain parameters are controlled by elastic properties. To evaluate these properties, we studied centrifuge model response to small amplitude sine sweeps. These motions had 0.02g Peak Base Accelerations (PBA), and were not able to generate notable PWP and deformations in the model. Soil behavior was predominantly elastic, and therefore, first natural frequency (f_1) of model can be approximated as $f_1 \approx 0.25V_s/H$. Value of f_1 can provide us a sense on site shear wave velocity, and through that, an estimation of shear modulus since $V_s = (G/\rho)^{0.5}$. To find f_1 , acceleration recordings at base and on top of the model were converted to frequency domain, and model transfer function was calculated. Results for four sine-sweeps, which were applied to the

model in between earthquakes (shown in Table 6.1), are presented in Figure 6.3. These graphs show two important points: (1) model f_1 was approximately 3 Hz, and (2) the stronger shakings in between these sine sweeps were not able to change f_1 . Therefore, numerical elastic properties must be calibrated to have $f_1 \approx 3$ Hz, and they should be kept the same for all simulations.

Using suggested parameters by Khosravifar (2012) in FE analyses, f_1 was simulated as 3.74 Hz. This resulted inaccurate predictions for PSA on top surface. Therefore, recognizing Nevada sand layer as the dominant soil in the profile with largest thickness and loosest relative density, we reduced its shear modulus to 60% of suggested values to have $f_1 \approx 3$ Hz like experiments. Figure 6.4 shows FE prediction for surface PSA with suggested and calibrated parameters against measurements during FFP-EQ1 motion. As it can be seen, suggested parameters resulted almost two times greater peak PSAs and in different periods from experiments. Favorably, calibrated parameters closely captured peak PSAs and their periods. Therefore, for rest of the simulations we used our calibrated shear modulus.

6.5.2 Large Strain Calibration

Soil volumetric behavior is represented by constitutive model. PDMY02 has six parameters to simulate volumetric behavior. Three parameters for contraction phase and three parameters for dilation phase to control volume change rate, pore water pressure generation, fabric damage and overburden stress effects on volumetric behavior (Khosravifar 2012). In essence these parameters control coefficient of volume compressibility (m_v) in equation $c_v = k/(m_v \gamma_w)$. Where c_v is coefficient of consolidation, k is permeability and γ_w is unit weight of water. The parameter c_v is the key parameter controlling volume change and pore water pressure generation in FE simulations. To calibrate c_v , we either need to calibrate the six parameters for m_v or adjust k to result a proper c_v . Calibrating m_v requires a set of laboratory tests to find best fitting values, thus, it is more convenient to calibrate k . Permeability, itself, plays an important role in determining the liquefaction response of homogenous sand deposits, affecting especially the depth of liquefaction, the speed at which excess PWP dissipates after shaking, and the character of the ground surface settlement (Sharp et al. 2003). Therefore, calibrating it requires great attention to evaluate gains and losses in simulations.

The Nevada sand used in centrifuge model is a relatively uniform clean fine sand. It has a specific gravity (G_s) of 2.64, a mean grain size (D_{50}) of 0.17 mm (model scale), a uniformity coefficient (C_u) of 1.64, and a permeability of 0.002 cm/s (Kamai et al. 2008 and Arulmoli et al. 1992). For a medium sand, typical values of m_v are generally on the order of $4e-5$ to $10e-5$ (1/kPa) (Pestana et al. 1997), and vary based on relative density, confining pressure, and the peak excess pressure in the soil (Lee and Albaisa 1974). Howell (2013) comprehensively studied PDMY02 model and found out that the equivalent m_v values modeled by PDMY02 are 1/10 to 1/3 smaller than those measured in experiments. Therefore, permeability of FE model must be set to 0.002/10 cm/s to 0.002/3 cm/s to include the effects of m_v through permeability.

As it was mentioned before, centrifuge tests were performed under 50g centrifugal acceleration with a fluid 30 times more viscous than water. A model saturated with water and tested at N g would have permeability N times greater than the same soil in prototype saturated with water and tested at 1g (Kutter 1995). Therefore, the permeability of soil was 1.67 ($=50/30$) times higher than actual values, resulting in 0.0003 cm/s ($=0.002/10 \times 1.67$) to 0.0011 cm/s ($=0.002/3 \times 1.67$). Seed et al. (1975) suggested that for a loose sand ($D_r = 30\%$ to 40%), m_v can increase by as much as a factor of 10 as liquefaction is approached. Since liquefaction happened very fast during the tests, and PDMY02 maintains a relatively constant permeability during simulations, the numerical model permeability should at most be set to 0.003 cm/s to 0.011 cm/s. Based on these, we chose the permeability range of 0.0003 cm/s to 0.01 cm/s and performed simulations to find best PWP generation and settlement relative to experiments. Our model performed best with permeability 0.01 cm/s.

Top and bottom Monterey sands were built as drainage boundaries for Nevada sand layer. The top layer was almost dry (except 0.5 m) and did not experience liquefaction. The bottom layer was very dense ($D_r = 90\%$) and did not experience liquefaction either. None of these layers deformed considerably relative to loose Nevada sand layer during all the excitations. We chose permeability of 0.05 cm/s for Monterey sand layers to have similar PWP distribution to recordings.

After calibrating permeability of layers and ensuring close capture of PWP recordings, we calibrated PDMY02 contraction rate parameter (c_1) of Nevada sand to better capture settlements. Khosravifar (2012) has suggested $c_1 = 0.06$. We chose $c_1 = 0.08$ to have more contractive behavior in Nevada sand layer and better replicate settlements.

6.6 Simulation of Soil Response in the Free-Field Profile

6.6.1 Data Refinement

High frequency noise was detected in some of the sensor recordings. For example, FFP-EQ3 input motion was scaled to 0.54g, while the recordings at base were showing peak acceleration of 0.74g. The noise faded out in recordings at higher elevations. High frequency noise was also detected in FE simulations with increasing trend at higher elevations. Recordings and numerical noises were not significant for FFP-EQ1 and FFP-EQ2, whereas they became influential during FFP-EQ3 at depths above 6 m. To remove these noises, we refined the input motion and the predicted values by a second order Butterworth band-pass filter with 0.05 Hz and 8 Hz corner frequencies. The rest of results and recordings were not filtered and are exact as they were recorded and predicted.

6.6.2 Free-Field Acceleration Response

Computed and measured Pseudo Spectral Accelerations (PSA) throughout model profile are shown in Figure 6.5. As seen by comparison, FE model was able to closely capture peaks and their periods, and the general trend of PSAs at all depths. The discrepancies increased with stronger earthquakes and with wave upward propagation. The only exception is predictions for FFP-EQ3 at depth of 4 m, where simulated values are substantially smaller than measurements. This is happening at boundaries of liquefied area in Nevada sand layer. Shown in Figure 6.6, this area had high levels of PWP ($r_u > 0.6$) with little effective stress. Soil behavior was mostly nonlinear, and nonlinearity can significantly de-amplify motion. Numerical models become sensitive at low effective stresses and might not be able to predict proper values for acceleration. Moreover, PSAs at this elevation are even smaller than PSAs at above elevations indicating that motion was amplified above this elevation while passing through liquefied soil. This is against our expectations and what was recorded during the tests. Therefore, the over-estimated reduction in PSA might be attributed to an error in numerical model and soil nonlinearity at that elevation.

Idriss (1990) indicated that ground motions with moderate acceleration levels (less than 0.4g) will be amplified while passing through soft soil profiles. However, at higher acceleration levels, the low stiffness and nonlinearity of soft soils often de-amplify motions. Findings in centrifuge

tests are consistent with this. FFP-EQ1 and FFP-EQ2 with accelerations less than 0.4g were amplified, and FFP-EQ3 with accelerations higher than 0.4g was de-amplified while passing through the loose Nevada sand. As it was shown, FE model quite accurately predicted this phenomenon. Moreover, similar to Seed et al. (1976) and Armstrong et al. (2013) findings, motion passing through loose soil profile acquired longer-period PSA on top soil surface. FE model was able to closely capture this phenomenon too.

6.6.3 Free-Field PWP and Settlement Responses

FFP-EQ1 was not strong enough to generate notable PWP and settlement. Computed and measured PWP and surface settlements for FFP-EQ2 and FFP-EQ3 are shown in Figs. 6 and 7. In our simulations, like Howell (2013), PWP dissipated immediately after motion intensity decreased to small amplitudes. This was in contrast with recordings which show limited PWP dissipations even after motion stopped. It was possible to reduce permeability of the numerical model to delay PWP dissipation, but this would adversely affect settlement predictions. In Figure 6.6, FFP-EQ2 was not able to trigger liquefaction but FFP-EQ3 liquefied the soil. It can be seen that except for predictions at 4 m depth, numerical predictions have reasonable agreement with measurements. Moreover, FE model was able to replicate liquefaction triggering accurately both for time and PWP generation.

In Figure 6.7, it can be seen that FE model was able to properly predict the amount of surface settlements. Measured and predicted values have about 5% and 8% relative difference for FFP-EQ2 and FFP-EQ3 respectively. Furthermore, it can be seen that in experiments majority of settlements occurred as PWP was building up, whereas in numerical simulations majority of settlements occurred after PWP was built and in low effective stresses. The reason why this behavior is monitored or why there is still a considerable amount of PWP in the model even long after the motion stopped, is not clear at this point. However, the discrepancy can be attributed to two main reasons: (1) there is a possibility of sensors malfunctioning against the saturation viscose fluid used in the experiments; and (2) due to a difference in permeability and dissipation regimes between numerical model and experiments. While the total values of settlement are independent of permeability, their developments are much dependent on permeability. In the similar tests by Taboada (1995) and Sharp et al. (2003), low permeability sands showed little settlements during shaking, with most of total settlement occurring a long period after shaking. However, high

permeability sands showed most of settlement take place during shaking. During centrifuge testing with 50g acceleration, all the dimensions become 50 times larger. This directly affects the size of particles and the voids in between them, which in turn, affects the overall permeability and PWP dissipation of the system. Probably, this phenomenon has facilitated PWP dissipation during the shaking. More study is required at this time to fully understand the observed behavior.

6.7 Simulation of Soil Response in the Reinforced Profile

We developed a 3D nonlinear finite element model to simulate soil response in the reinforced profile. The 3D set-up was to more properly include effects of columns on surrounding soil, PWP dissipation and surface settlements. We used OpenSeesPL program by Lu et al. (2010) as graphical pre- and post-processing program to generate mesh and interpret the simulated response. Owing to symmetry and to simplify computations, we developed a model for only one of the columns and its surrounding soil. The modeled area is highlighted in Figure 6.1 and the numerical model is shown in Figure 6.8.

Ground motion was applied to the model in the longitudinal direction. Side nodes above base at $x = -1.4$ m and 1.4 m were tied together for longitudinal and vertical displacements, and side nodes at $y = 0$ m and 1.4 m were fixed against transverse displacements to enforce shear beam mechanism. Base nodes were constrained against movements in all directions. Nine-node brick elements were utilized to model soil and column. Smaller elements were used around the column to have an interfacial transition area between very stiff column elements and soil elements. Similar to 2D analyses, we used elements with less than 1 m dimensions to have adequate nodes per wave length. FE numerical aspects were assigned like 2D analyses with only difference of adjusting Rayleigh damping corners to target first and second natural frequencies of reinforced profile.

We used calibrated soil parameters from free-field studies for reinforced profile analyses since the same soils were used in both tests. Reinforcement materials were calibrated following same calibration procedure as discussed above. Studying transfer function of centrifuge model during small amplitude sine sweeps revealed that reinforcements have increased the first natural frequency of model to 3.5 Hz (comparing to 3 Hz in the free-field profile). Rayamajhi et al. (2015) estimated shear modulus of soil-cement columns as 174 MPa. Using this value in numerical

analyses resulted in $f_1 = 4.1$ Hz with significant deviations in PSA predictions from recordings. Therefore, we reduced this value step by step to have f_1 close to 3.5 Hz. Our simulations showed that only 30% of estimated value (≈ 52 MPa) should be used to have $f_1 = 3.5$ Hz. This value was considerably closer to suggested value by Khosravifar (2012) for shear modulus of a very stiff clay as 50 MPa.

For large strain calibrations, we increased permeability of all sands by five times to closer replicate recordings for PWP and surface settlements. During second earthquake, soil in between columns settled for more than 3 cm. As occupied space by soil was relatively small, this settlement might have caused densification in Nevada sand. Therefore, to be able to use a newly generated mesh for third motion, we reduced compressibility coefficient of Nevada sand (c_1) to 0.02 to account for densification and geometry changes. This value was found through trial and error for best response.

Following ground improvement, loose sites become stiff and this can affect the dynamic site response. In this study with area replacement ratio $\sim 30\%$, f_1 or V_s of the site was increased by 17%. This was consistent with field tests by Stuedlein et al. (2015). They measured field shear wave velocities before and after installing stone-columns with 21% area replacement ratio. Their results showed approximately 13% increase in V_s of the site. However, these findings were significantly different from Ishikawa and Asaka (2006) shake table tests. Column-type improvements with 20% area replacement ratio showed 80% increase in V_s of the site. It should be noted that Ishikawa and Asaka (2006) tests were performed under very high frequency (30 Hz to 300 Hz) and very large amplitude (3g to 5g) motions. These motions were far from our concerned motions with dominant frequencies between 0.05Hz to 10Hz and amplitudes of less than 0.6g.

6.7.1 Reinforced Profile Acceleration Response

Computed and measured PSAs throughout reinforced profile are shown in Figure 6.9. As seen by comparison, FE model was able to fairly accurately predict peaks and their periods and the general trend of PSAs at all depths especially during RP-EQ1 and RP-EQ2. The discrepancies increased with stronger shakings and in higher elevations. The differences became significant during RP-EQ3 in depths 2 m, 4 m and eminently in 6 m. The reason behind this can be found in

Figure 6.10 from PWP predictions. In Figure 6.10, recordings at depth 9 m in the middle of very dense sand ($D_r = 90\%$) are showing liquefaction triggering with $r_u \approx 95\%$. Whereas, FE simulations are barely reaching $r_u = 60\%$ without soil liquefaction. FE simulations are considerably closer to our expectations from a very dense sand at high levels of confining pressure excited with a medium intensity earthquake (PBA = 0.46g). Essentially, the very dense sand should not have liquefied. Nonetheless, liquefaction was triggered at depth 9 m in experiments and soil nonlinearity has de-amplified the motion passing through. Whereas, in FE simulations liquefaction was triggered around depth 6 m, and there was not much nonlinearity and de-amplification below that. The deviation between PSA recordings and predictions can be attributed to liquefaction triggering at lower depths in experiments than simulations.

Comparing free-field with reinforced profile results, it can be seen that low amplitude earthquakes (i.e. PBA = 0.02g and 0.1g) were amplified more in reinforced profile. During RP-EQ1, peak PSA was increased for about 0.05g at surface, while during FFP-EQ1, this increase was 0.02g. During RP-EQ2, peak PSA was increased for 0.4g, whereas during FFP-EQ2, this increase was only 0.06g. The discrepancy became more significant between medium amplitude motions (i.e. PBA $\approx$ 0.5g). In free-field profile, FFP-EQ3 was de-amplified with peak PSA decreasing for about 2.3g. Whereas in reinforced profile, RP-EQ3 was amplified with peak PSA increasing for about 0.3g. Since liquefaction was triggered in both profiles, soil contribution was the same. Therefore, the amplification must have come from rigid columns in reinforced profile. Stiff columns did not undergo any softening nor nonlinearity and should have been able to transfer the wave energy almost intact from base to the surface. This is in contrast with Baez and Martin (1993) and the idea that using stiff columns can reduce seismic intensity on surface.

The recording and simulations for RP-EQ1 and RP-EQ2 motions were consistent with Idriss (1990) recommendations. These motions with amplitudes less than 0.4g amplified while passing through loose soil profile. Soil-cement columns restricted system nonlinearity and RP-EQ3 with amplitude more than 0.4g, was not de-amplified as Idriss (1990) suggested. During all motions, results did not show considerable period elongation. It should be emphasized that FE analyses were able to closely capture all these phenomena in experiments.

6.7.2 Reinforced Profile PWP and Settlement Responses

RP-EQ1 was not strong enough to generate notable PWP nor cause considerable settlement. Computed and measured PWP for RP-EQ2 and RP-EQ3 are shown in Figure 6.10. Similar to free-field results PWP dissipated faster in FE simulations than recordings. In general, FE simulations were able to fairly predict PWP generation and peaks. The discrepancy increased in stronger motion with in average about 15% PWP ratio difference. Perhaps, the most important deviation was in the vicinity of liquefaction. In experiments, almost entire model was liquefied. Recordings show high levels of PWP ratio from 9 m depth extended to 3 m depth. Whereas, FE simulations show only high levels of PWP ratio in the middle of loose sand at depths 5 m and 6 m. As it was mentioned before, this has considerably affected intensity of waves propagating through the model.

Comparing the results from free-field with reinforced profile, it can be seen that during experiments PWP dissipated faster in reinforced profile. We expected impermeable columns to reduce overall site permeability and delay PWP dissipations consistent to Adalier (1996) centrifuge tests. FE simulations were in agreement to our expectations and showed delayed dissipations in reinforced profile. Rayamajhi et al. (2015) suggest that preferential seepage paths might have formed along the sides of columns resulting in faster rates of dissipations in experiments. Other than this, there was not much difference in PWP generation between two models. Soil liquefied almost the same in both models, and columns were not able to provide any effective sorts of lateral restriction or resistance to mitigate liquefaction. It should be emphasized that this likeness between free-field and reinforced profiles was closely captured by FE analyses.

In Figure 6.11 measured and simulated settlements on soil surface and top of the column are shown. FE simulations were able to closely capture settlements on soil surface with approximately 3% and 6% errors during RP-EQ2 and RP-EQ3 respectively. Settlements were also captured accurately on top of soil-cement columns during RP-EQ2 with 0.1 cm difference. However, recorded and predicted values were considerably different (≈ 3 cm) on soil-cement column for RP-EQ3. During the tests, very stiff columns did not undergo any permanent deformations, and the settlements recorded herein must be from columns sinking in to the very dense sand layer underneath. As mentioned before, bottom sand layer was liquefied during the test and has probably opened room for columns to settle in. Our FE simulations were considerably closer to other tests

by Rayamajhi et al. (2014) with columns resting on rigid base. The facts that bottom sand layer liquefied and showed approximately 4 cm settlements during a medium amplitude earthquake ($PBA = 0.46g$) is against our expectations from a very dense sand layer. Most probably, the air pluviation used in the laboratory was not able to make a sand layer with 90% relative density and we are dealing with a sand layer considerably looser with 65% to 75% relative densities in accordance to observed behavior. Our further simulations showed that, if we model bottom sand layer with $D_r = 75\%$, settlements would be predicted substantially better both for soil surface and top of soil-cement columns.

Comparing the responses from free-field with reinforced profile, it can be seen that, the overall settlements on soil surface have increased in reinforced profile. Settlements increased from about 2.2 cm in the free-field profile to 3.2 cm in the reinforced profile during the earthquakes with 0.1g peak accelerations, and from 7.4 cm to 12 cm during the earthquakes with 0.5g peak accelerations. However, settlements on top of soil-cement columns were 0.5 cm and 4 cm (77% and 46% reduction respectively relative to free-field settlements) with respect to mentioned motions. These values being considerably smaller than the free-field settlements indicate that, if soil-cement columns were strong enough to hold the loads from foundations, they can be used to reduce the settlement underneath the structure. Our results were consistent with Miller et al. (2012) full scale blast tests on DSM columnar reinforced ground. Field measurements showed that reinforcements were able to reduce settlements by 50% and ensure uniform settlement of the site.

However, it should be noted that the large differences between soil surface and top of soil-cement columns can be detrimental for structures placed on top of these improved grounds. If bearing columns of a structure are placed on both soil and soil-cement columns, structure might suffer from drastic differential settlements and angular rotations at foundation level.

6.8 Summary

In this study, we performed finite element numerical modeling to evaluate effects of soil-cement columnar reinforcements on seismic response of loose soil profiles. For this purpose, we built a nonlinear numerical model and calibrated it with results from centrifuge testing of a free-field profile. Thereafter, the calibrated model was used to capture the behavior of a reinforced

profile during centrifuge testing with base excitations of different intensities. The main focus was put on seismic wave propagation, PWP generation and surface settlements.

In the free-field profile, FE simulations were able to closely capture PSAs throughout profile in terms of peak, periods and trends. A few exceptions occurred at depths where soil was liquefied and was under very low effective stresses. Recordings and Simulations were in agreement with literature, where small amplitude earthquakes ($PBA < 0.4g$) were amplified whereas large amplitude earthquakes ($PBA > 0.4g$) were de-amplified while passing through loose soil profile. Moreover, loose soil profile elongated PSAs on top surface. FE simulations provided realistic simulations for PWP generation and peaks, but with faster dissipation rates. Liquefaction triggering was captured accurately both for time and trend of generation. Total values of surface settlement were predicted closely with negligible differences in trends of settlements. In experiments, most of settlements occurred during excitations and when PWP was building up, whereas in FE simulations most of settlements happened after motion intensity dropped and when PWP started to dissipate.

In reinforced profile, FE simulations were able to closely capture PSAs throughout profile during small amplitude motions. The differences increased during the medium amplitude motion mostly because liquefaction was triggered at deeper depths in experiments, which considerably de-amplified the accelerations on above recordings. Simulations for PWP were in reasonable agreements with recordings. The main discrepancy was in vicinity of liquefaction. In experiments soil liquefied in deeper depths and extended to higher elevations than simulations. FE analyses were also able to provide fairly accurate results for settlements on soil surface during all motions and on soil-cement columns during small amplitude motion. During the large amplitude motion, studied column sank into the underlain “unexpectedly liquefied soil” and showed considerably different settlements from simulations.

Comparing the response from free-field with reinforced profile showed that soil-cement columns increased seismic amplification on soil surface. Columns became detrimental during medium amplitude motion by reducing system nonlinearity and preventing seismic de-amplification. PWP generation was almost the same in both models, and reinforcements were not able to limit PWP generation nor stop liquefaction triggering. Reinforced profile showed larger settlements on soil surface and smaller settlements on columns comparing to the free-field profile.

Results collectively included that soil-cement columns were not efficient in liquefaction remediation and reducing seismic intensity on ground surface.

6.9 Acknowledgements

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6.10 References

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6.11 Tables

Table 6.1: Simulated motions in free-field and reinforced centrifuge tests

Model	Motion	Peak Base Acceleration (g)
Free-Field Profile	FFP-EQ1	0.02
	FFP-EQ2	0.13
	FFP-EQ3	0.54
Reinforced Profile	RP-EQ1	0.02
	RP-EQ2	0.12
	RP-EQ3	0.46

6.12 Figures

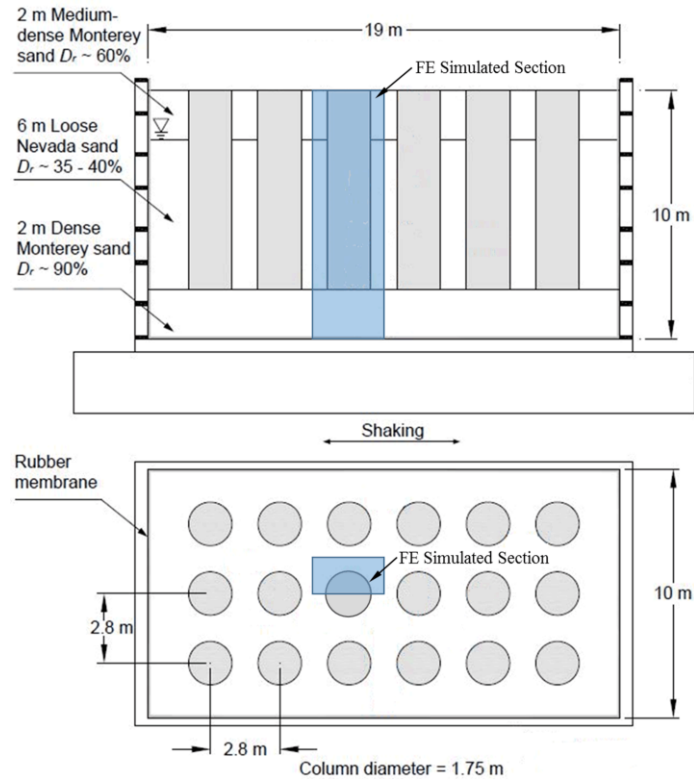


Figure 6.1: Centrifuge model configuration for reinforced profile (after Rayamajhi et al. 2015)

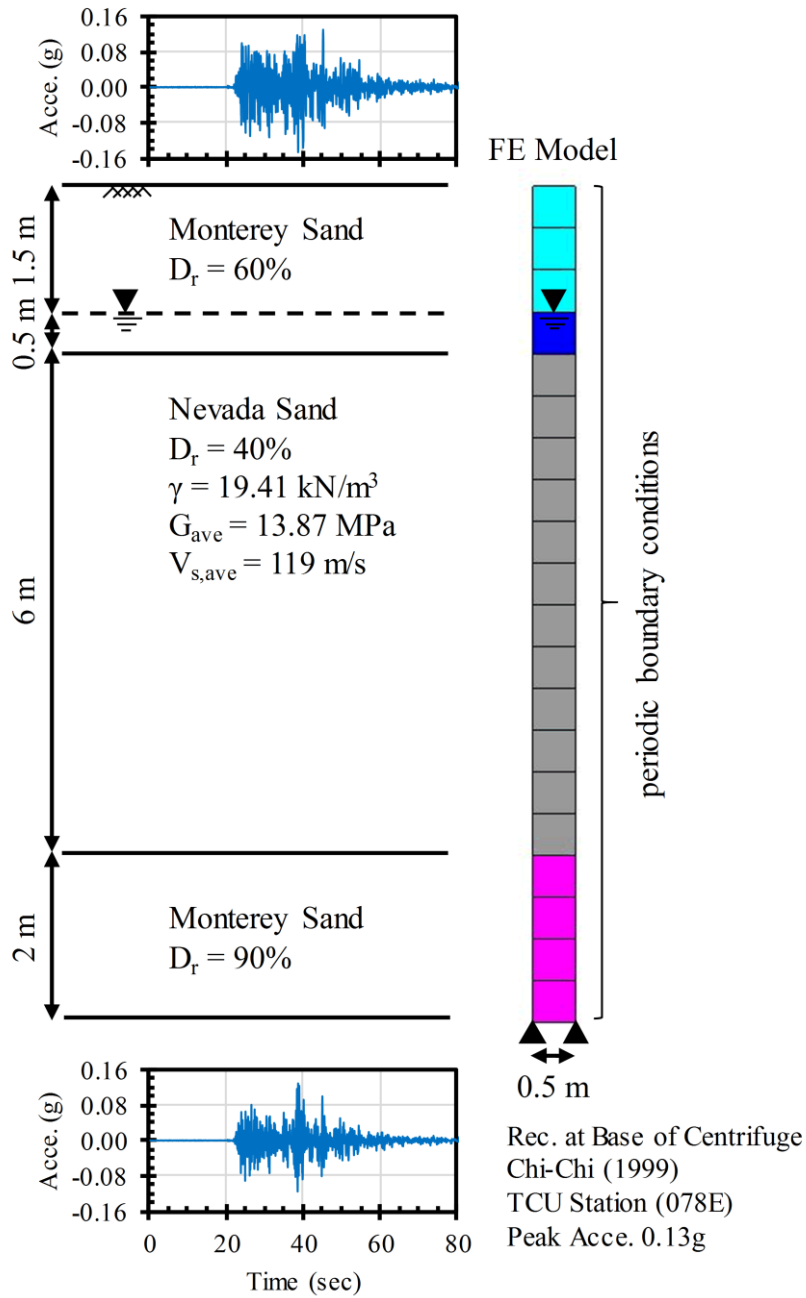


Figure 6.2: Free-Field profile with acceleration recordings at base and surface of the centrifuge test, and the FE model used in simulations

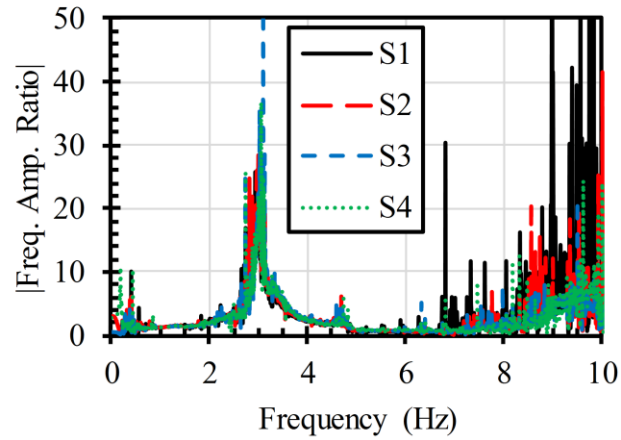


Figure 6.3: Transfer function of free-field centrifuge model with sine sweep input motions.

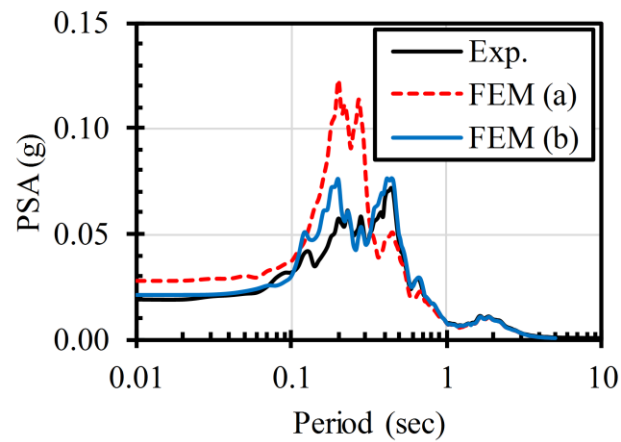


Figure 6.4: Predicted PSA at top soil surface with (a) literature suggested parameters, and (b) calibrated parameters in comparison with PSA from experiments.

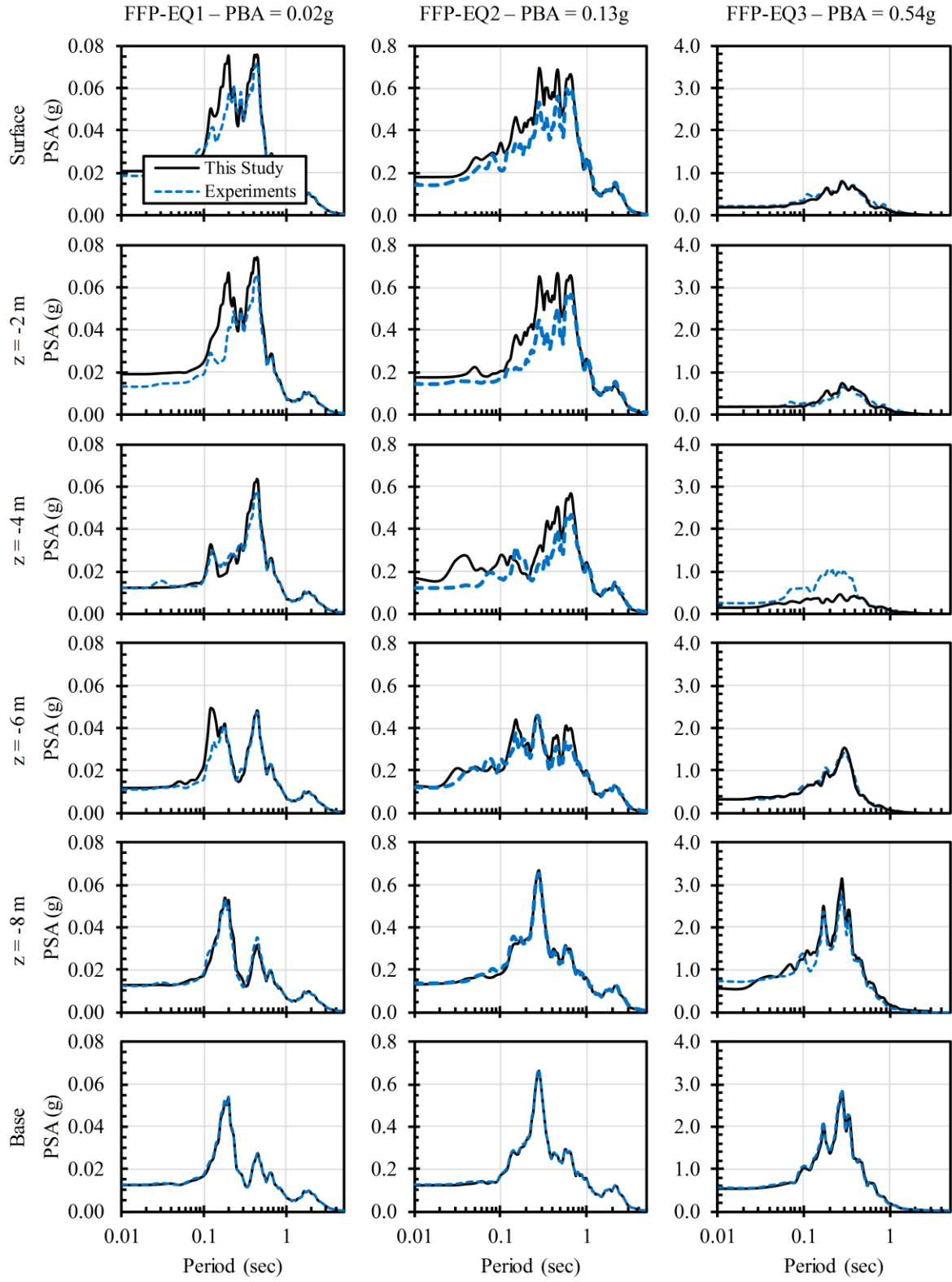


Figure 6.5: FE simulated pseudo spectral accelerations vs. centrifuge free-field test recordings throughout soil profile for Chi-Chi motion with different peak base accelerations.

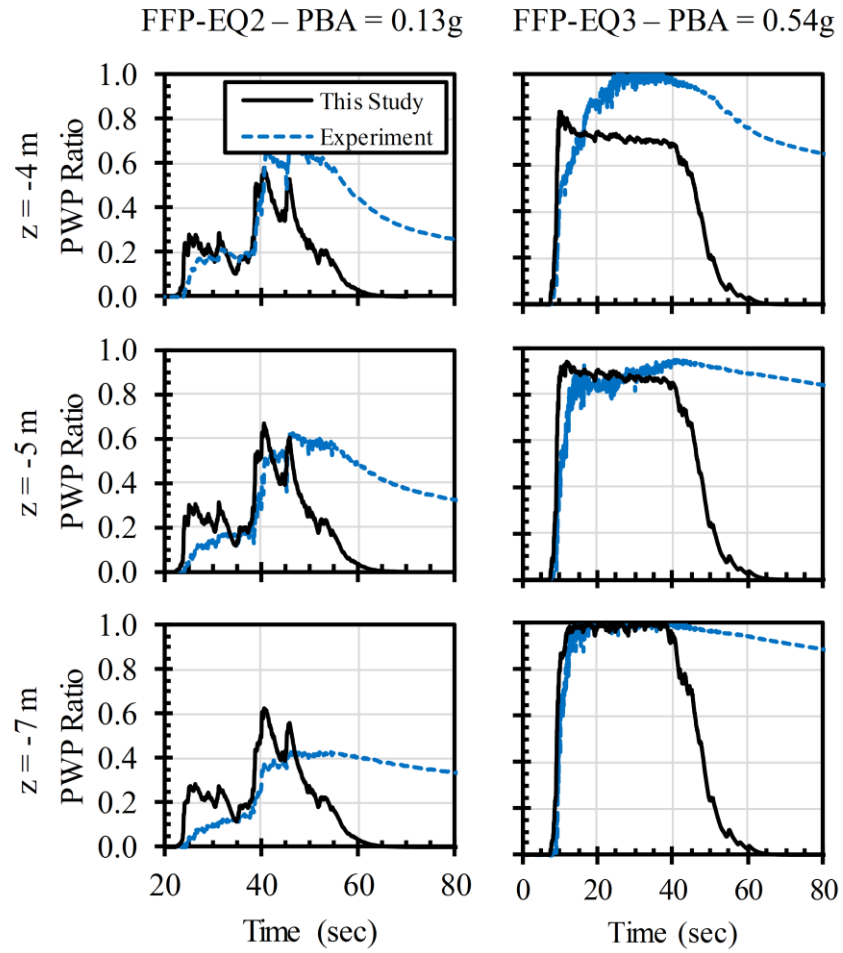


Figure 6.6: FE simulated excess PWP vs. centrifuge free-field test recordings in Nevada sand layer for Chi-Chi motion with peak base accelerations of 0.13g and 0.54g.

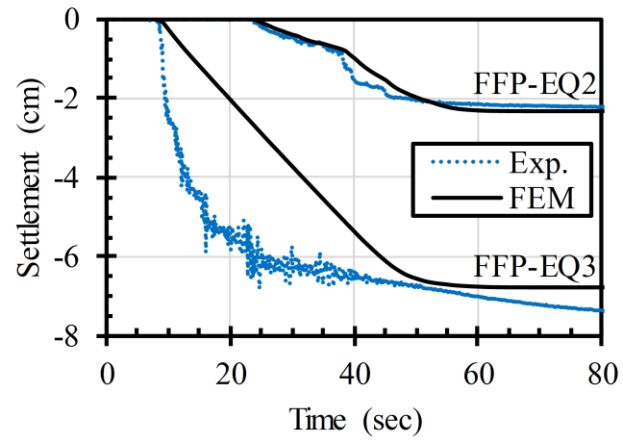


Figure 6.7: FE simulated settlement vs. centrifuge free-field test recordings on top soil surface for Chi-Chi motion with peak base accelerations of 0.13g (FFP-EQ2) and 0.54g (FFP-EQ3).

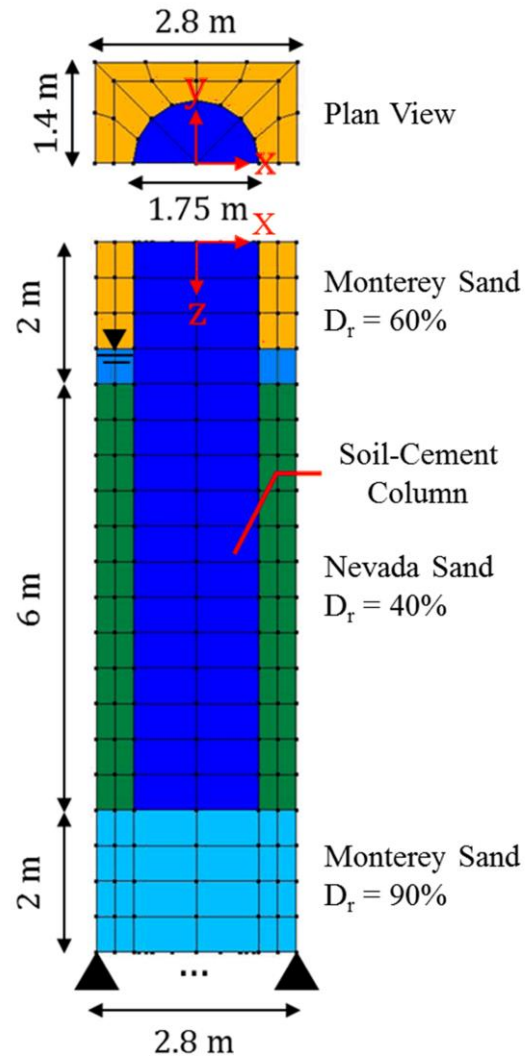


Figure 6.8: FE model used to study the reinforced centrifuge test.

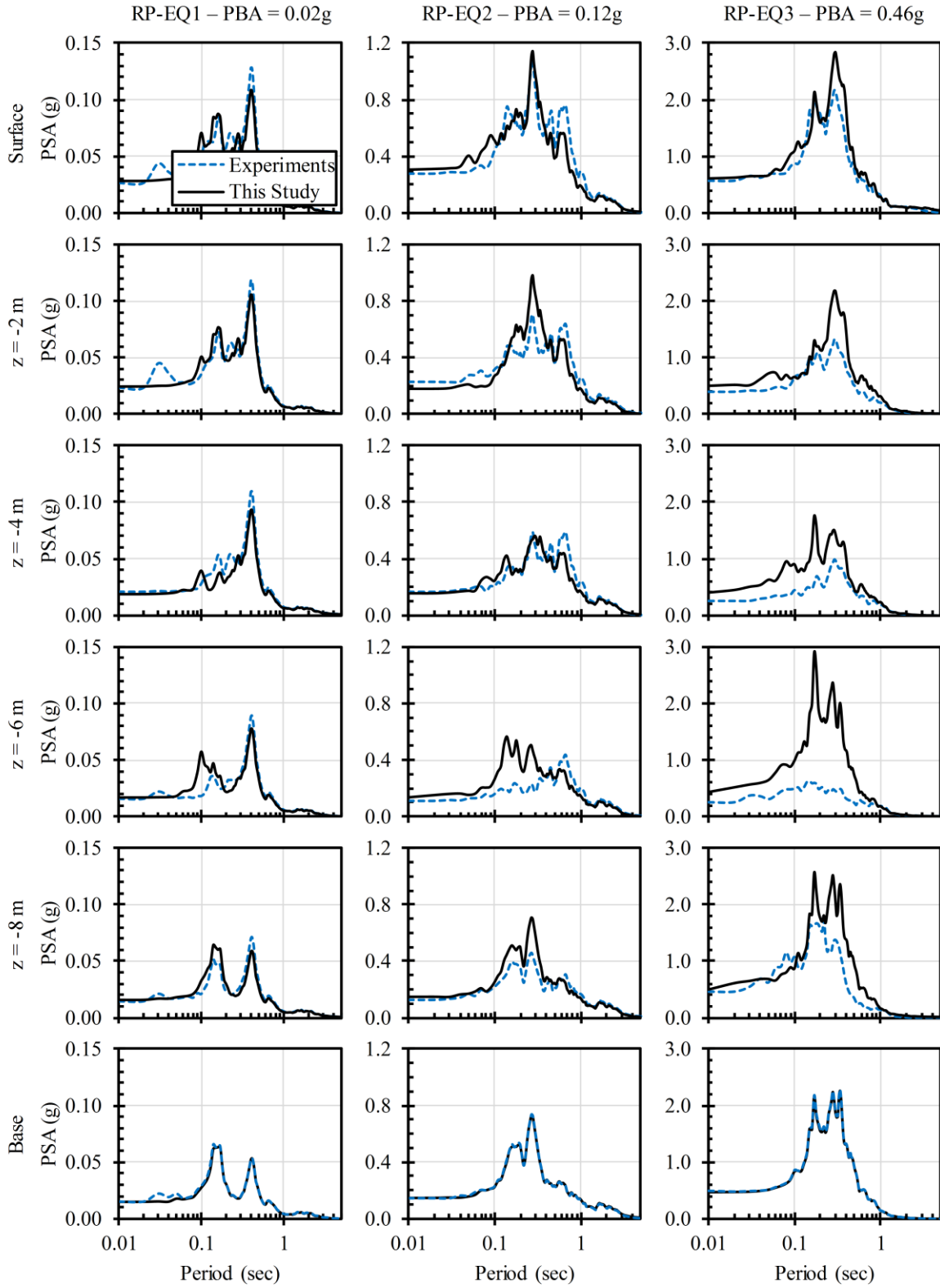


Figure 6.9: FE simulated pseudo spectral accelerations vs. centrifuge reinforced profile recordings throughout soil profile for Chi-Chi motion with different peak base accelerations.

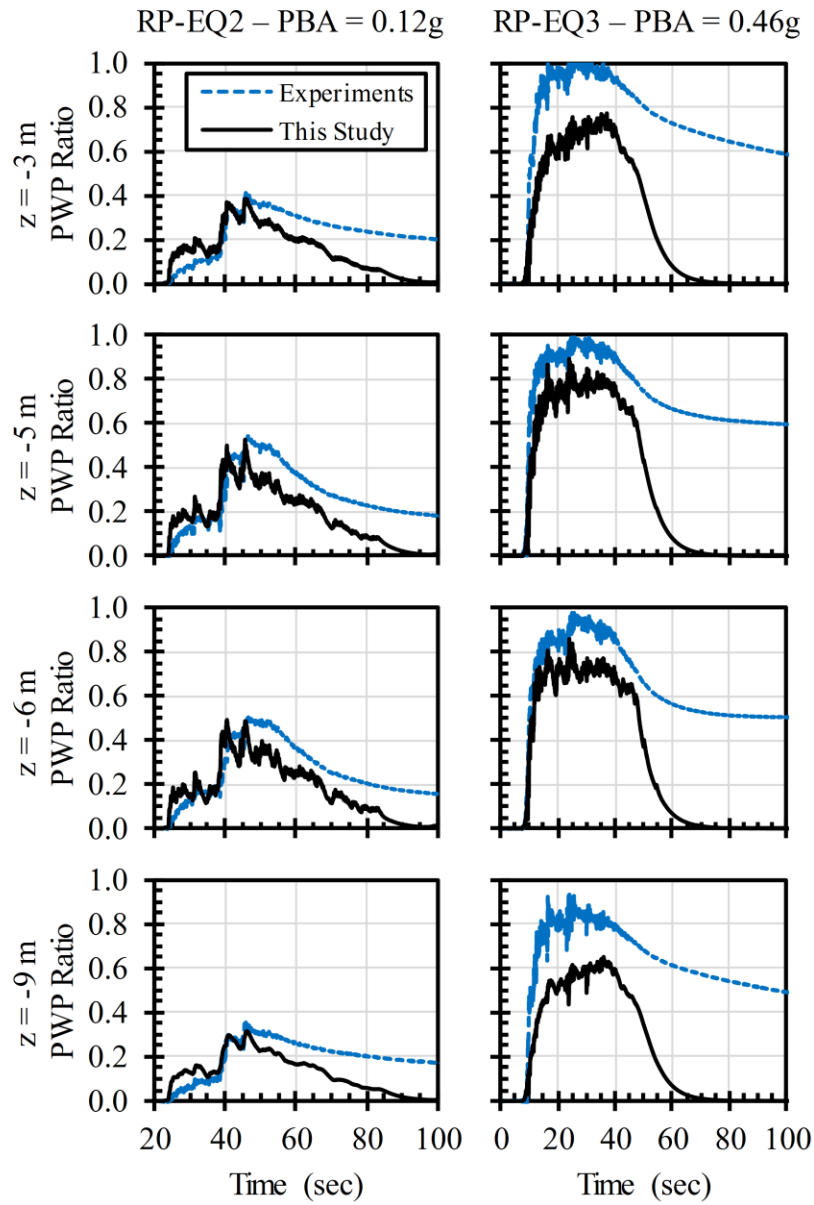


Figure 6.10: FE simulated excess PWP vs. centrifuge reinforced profile recordings in Nevada sand layer for Chi-Chi motion with peak base accelerations of 0.12g and 0.46g.

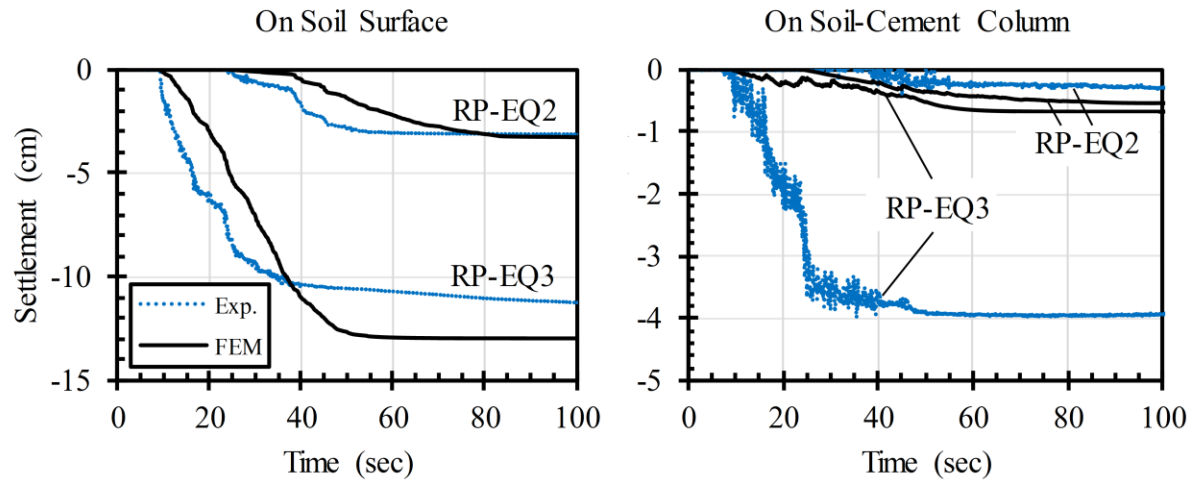


Figure 6.11: FE simulated settlement vs. centrifuge reinforced profile recordings on ground surface and on top of soil-cement column for Chi-Chi motion with peak base accelerations of 0.12g (RP-EQ2) and 0.46g (RP-EQ3).

Chapter 7

Summary

7.1 Summary of the Research

In this research, a numerical approach was adopted to study the seismic performance of columnar reinforced sandy grounds. The study builds on centrifuge tests performed by Rayamajhi et al. (2014) to further investigate effect of the reinforcements on Pore Water Pressure (PWP) generation, ground deformation and seismic intensity variation. Rayamajhi et al. (2014) performed four centrifuge tests with free-field and reinforced profiles to observe the effect of reinforcements on seismic behavior of loose grounds. The results from these tests were used, herein, to calibrate and validate nonlinear finite element models. The models were later used to evaluate efficiency of columnar reinforcements with different geometries during numerous earthquake excitations. In pursuit of these objectives several tasks and related research were performed. This chapter summarizes the entire body of this research.

7.2 Constitutive Modeling

The core constitutive model that represents soil behavior in geotechnical problems is perhaps the most influential factor affecting the accuracy of numerical simulations. In this study, to be able

to properly and realistically predict soil behavior in our model tests, we evaluated several constitutive models against experiments to distinguish which one serves our purposes best. These models included: NorSand by Jefferies and Been (2006), DM04 by Dafalias and Manzari (2004), Plasticity Model for Sands (PM4Sand) by Boulanger and Ziotopoulou (2012), and Pressure-Dependent-Multi-Yield-02 by Yang and Elgamal (2008).

7.2.1 Model Capabilities in Monotonic Loading

Presented in Chapter 2, NorSand and DM04 were evaluated in predicting soil behavior under monotonic loading. We simulated Toyoura and Erksak sands behavior in a series of 26 different triaxial tests in drained and undrained conditions. The tests were performed on soils with different relative densities and under a range of confining pressures. We investigated accuracy of the models in predicting the stress-strain and the volumetric behaviors observed in the experiments.

Both these models were able to capture general trend and curvature of the soil behavior. DM04 was able to predict slightly better values for deviatoric stress at the peak and 2%, 5% and 10% axial strains, while NorSand was able to predict better values at the critical state. DM04 was also able to predict the volumetric behavior realistically and with slightly better accuracy than NorSand. Dispersion analyses showed both models predict better values for stresses than strains. Overall, the results showed that following proper calibration, these models can provide realistic estimates of soil strength and deformability, and can be used within numerical domains to represent soil behavior during different monotonic loading conditions.

7.2.2 Model Capabilities in Cyclic Loading

Presented in Chapter 3, we evaluated capability of PM4Sand in predicting soil cyclic behavior. The model was implemented to predict soil response during several experiments reported in literature. De Alba et al. (1976) cyclic shake table tests were simulated to investigate capability of the model in different relative densities of 54%, 68% and 82%. Wijewickreme et al. (2005) Direct Simple Shear (DSS) tests were simulated to investigate capability of the model in different confining pressures of 50 kPa, 100 kPa, and 200 kPa. Norwegian Geotechnical Institute (1977) DSS tests were simulated to investigate capability of the model in different pre-existing shear stresses. In addition to these simulations, we investigated capability of the model against cyclic triaxial tests by Seed and Lee (1965) and cyclic torsional shear tests by Ishihara (1996). These

simulations collectively showed that PM4Sand with minor calibrations can provide a relatively accurate estimate of soil behavior and liquefaction resistance during uniform cyclic loading.

Presented in Chapter 4, we evaluated capability of PDMY02 in predicting soil cyclic behavior within a finite element model. Numerical simulations with PDMY02 as the core constitutive model were performed to capture behavior of a loose soil profile during seismic excitations with different intensities. The model was able to realistically represent soil behavior close to the recordings in the experiment.

7.3 Preparation of the Finite Element Model

7.3.1 Numerical Settings

In our preliminary studies, it was brought to our attention that numerical aspects of finite element approach, distinct from geotechnical aspects, can have substantial effect on our simulations. Accordingly, presented in Chapter 4, we performed a comprehensive study on the numerical aspects that can influence the targeted study herein. In particular, we studied convergence tolerance, solver time-step, constraint definitions, and, integration, material and Rayleigh damping.

Our investigations showed that a low convergence tolerance cannot necessarily increase the accuracy of simulations, while a high tolerance limit can drastically affect small strain behaviors. Solvers with time-steps smaller than the time-step of input motion can introduce high frequency noise to the solution system capable of affecting the results. While, solvers with larger time-steps can miss some of the influential frequencies. Manipulating stiffness matrix to enforce constraints can utterly disturb the results, and should be avoided if possible. Whereas, manipulating displacement matrix can de-stabilize solution system, particularly, if some nodes are tied in different constraints. Rayleigh damping can be used to complement lack of damping from material hysteresis behavior, and also to damp out very low and high frequency noises. However, improper use of Rayleigh damping can affect proportionality between dominant natural frequencies and severely affect the simulations. Numerical damping from integration round-offs can help preventing unintended model oscillations, stimulate computations and damp out high frequency noises. Yet, imprudent round-offs can damp model dynamic response detrimentally.

Results also show that model transfer function can be used as a strong tool to evaluate effect of numerical settings on finite element simulations. Numerical settings alter frequency amplifications in transfer function, and can be influential when they affect dominant natural frequencies. Improper setting can be avoided by continuous validation of transfer function and ensuring noises not reaching the dominant frequencies.

7.3.2 Model Calibration

Presented in Chapters 4 and 5, results from the free-field centrifuge tests were used to calibrate the PDMY02 constitutive model. For this purpose, a systematic approach was taken to calibrate the model for small strain (i.e. elastic parameters) and large strain (i.e. volumetric parameters) behaviors. The small strain parameters were calibrated by matching transfer functions of the centrifuge model and the numerical model. The large strain parameters were calibrated by indirectly adjusting the coefficient of consolidation in the numerical model to take into account findings in literature on volumetric behavior of the loose grounds. Favorably, the calibrations significantly enhanced the capability of the model in predicting accelerations, PWP generation and settlements throughout the profile.

7.4 Numerical Simulation of the Reinforced Profile during Seismic Excitations

A 3-D nonlinear finite element model was developed to evaluate effect of soil-cement columnar reinforcements on seismic response of loose sandy profiles. The calibrated parameters from the free-field studies were used for the reinforced profile analyses since the same soils were used in both tests. Chapter 5 presents the simulations for the centrifuge test with fixed-end columnar reinforcements, and Chapter 6 presents the simulations for the centrifuge test with free-end columnar reinforcements.

Several simulations were performed for each tested profile with earthquake base excitations of varying intensities. Overall, the calibrated model was able to predict realistic values for the accelerations throughout soil profile. PWP generation and liquefaction triggering were also captured accurately both for time and trend of generation. However, some deviation between

simulations and recordings was observed in the vicinity of liquefaction. In the experiments, soil liquefied in deeper depths and extended to higher elevations. FE analyses were also able to provide realistic results for the settlements at soil surface and on top of the columns. The only considerable difference was observed during the strong motions for the test with the sandy base. Recordings showed columns sinking down into the sandy base, while they stood steady in the numerical model.

Comparing the response from the free-field and the reinforced profiles showed that soil-cement columns increased seismic amplification at soil surface. Columns became detrimental during stronger motions by reducing system nonlinearity and preventing seismic de-amplification. PWP generation was almost the same in both models, and reinforcements were not able to prevent liquefaction triggering. The reinforced profile showed larger settlements on soil surface and smaller settlements on columns comparing to the free-field profile. Results collectively included that soil-cement columns were not efficient in liquefaction remediation nor reducing seismic intensity on ground surface. However, they stood strong with limited settlements after the earthquakes and were able to provide a firm base for possible above foundations.

7.5 Parametric Studies

Parametric studies were performed to further evaluate the efficiency of columnar reinforcements. For this purpose, the validated numerical models were employed to evaluate effect of reinforcements on seismic performance of a 20 m loose sandy profile. The reinforcements were in form of soil-cement-columns extended to depths of 5 m, 10 m and 20 m, and with area replacement ratios of approximately 6%, 9%, 13%, 20% and 35%. The performance was evaluated under 113 different earthquakes with magnitudes $M_w = 5.5$ to 9.0 and maximum accelerations of 0.05g to 0.5g. In total, 1808 simulations were performed. The results from these simulations are planned to be presented in future publications.

7.6 Recommendation for Practice

The results in this research collectively included that utilization of the stiff column reinforcements requires great attention and understanding of the reinforcing mechanism. Both experimental and numerical studies showed stronger seismic intensities on the surface of the

reinforced ground. Therefore, the possible increase in the seismic intensity below foundations should be considered within design procedures. The columns might not be efficient in reducing pore water pressure generation nor preventing liquefaction triggering. The settlements in between columns can be considerably larger than the settlement on top of the columns. Therefore, the possible differential settlements, and consequently, the load transfer from the ground to the columns should be considered.

In this research, no structure was placed on top of the reinforcements. The presence of a structure will add to the complexity of the ground response to different shakings. The shear loads from the structure, and possibly, the rocking of foundation can crack and crash the columns. More study is required to completely understand the effect from overlying structures. The reinforcements should be designed adequately strong to not experience any significant damage during earthquakes.

7.7 References

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