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Dynamic Soil-Structure-Soil-Interaction Analysis of Structures in Dense Urban Environments

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Dynamic Soil-Structure-Interaction Analysis of Structures in Dense Urban Environments

by

Katherine Carys Jones

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in

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of the

University of California, Berkeley

Committee in charge:

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Dynamic Soil-Structure-Interaction Analysis of Structures in Dense Urban Environments

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Katherine Carys Jones
Abstract

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Doctor of Philosophy in Engineering – Civil and Environmental Engineering

University of California, Berkeley

Professor Jonathan D. Bray, Chair

Urban centers are increasingly becoming the locus of enterprise, innovation, and population. This pull toward the center of cities has steadily elevated the importance of these areas. Growth has necessarily spawned new construction. Consequently, modern buildings are often constructed alongside legacy structures, new deep basements are constructed alongside existing shallow foundations, and city blocks composed of a variety of building types result. The underlying soil, foundation, and superstructure of each of these buildings can interact and combine to yield unique seismic responses.

Since the seminal work of researchers such as Luco and Contesse (1973) and Wong and Trifunac (1975), researchers have investigated the effects of soil-structure interaction (SSI). This phenomenon refers to the interaction between a single building, its foundation, and the underlying soil during a seismic event. However, as the trend toward urbanization continues, a shortcoming of this conventional SSI approach is that in reality, a structure will almost certainly be located near other structures in metropolitan areas.

In this line of research, the interaction of multiple, adjacent buildings during a seismic event, a phenomenon known as structure-soil-structure interaction (SSSI), is investigated. This topic does not yet command the level of attention given to SSI. However, SSSI has the potential to be significantly detrimental or beneficial, depending on the configuration and dynamic properties of the buildings and their foundations in dense urban environments. It is important to understand SSSI effects so that earthquake engineers can make informed decisions about the design and construction of structures in increasingly dense urban areas.

As part of a larger, multi-university National Science Foundation (NSF)-supported Network for Earthquake Engineering Simulation Research (NEESR) project, a series of centrifuge experiments were performed at the NEES-supported Center for Geotechnical Modeling (CGM) at the University of California, Davis. Each of these experiments examined aspects of SSI or SSSI through the use of nonlinear structural model buildings situated on different foundations that were supported on deep sand deposits. The centrifuge experiments created a suite of small-scale physical model “case histories” that provided “data” and insight that could be extended
through calibrated numerical simulations. The results of the first three centrifuge experiments in the test series (i.e., Test-1, Test-2, and Test-3) were utilized in this dissertation.

Numerical analyses are usually only performed for high-profile projects. The effort, expertise and resources required to calibrate and to perform detailed numerical simulations is often prohibitive for typical low- to mid-rise structures. There is a need for a more accessible numerical tool that both geotechnical and structural engineers can utilize to gain insight. In this research, the FLAC finite difference program (Itasca, 2005) with a fully nonlinear effective stress soil constitutive model was used to analyze the centrifuge test-generated “case histories.”

Test-1 and Test-2 examined SSI and SSSI effects of two moment-resisting frames (MRFs). Test-1 employed a solitary 3-story (prototype) MRF founded on shallow spread footings and a solitary 9-story (prototype) MRF founded on a deep basement (equivalent to 3-stories, prototype) to investigate SSI effects. In Test-2, the 3-story (prototype) and 9-story (prototype) MRFs were placed immediately adjacent to one another to examine SSSI effects. Kinematic interaction effects were primarily observed in these tests. Hence, Test-3 was designed to investigate inertial interaction effects. Three structures were included in Test-3: two MRFs founded on shallow spread footings and one elastic shear-wall structure on a mat foundation. Each of these structures was designed to maximize inertial interaction by: (1) matching the flexible base period of each structure to the soil column to induce resonance, and (2) optimizing structural properties to increase inertial interaction effects. One MRF was positioned alone at one end of the centrifuge model, a SSI condition, and the other MRF and the elastic shear-wall structure were positioned immediately adjacent to each other in the other end of the centrifuge model, a SSSI condition.

The rich data set developed through the centrifuge experiments formed the basis of the initial FLAC analyses. A critical aspect of any seismic analysis is the constitutive model used to capture the soil response to cyclic loading. Several soil models were examined during an initial seismic site response analysis. Free-field data from sensors located within the centrifuge soil column were used to quantify the vertical propagation of ground motions through the soil profile. The best model for the dense (Dr = 80%), dry sand used in the centrifuge for Test-1 through Test-3 was a Mohr-Coulomb based model with hysteretic damping, UBCHYST (Naesgaard, 2011). Pseudo-acceleration response spectra and acceleration time histories at the base and at the free-field surface from the centrifuge and the numerical model were compared. The numerical simulations successfully captured the key aspects of the observed seismic site-response for both near-fault pulse-type motions and ordinary motions at a variety of intensities.

After successfully capturing the free-field seismic site responses of Test-1 and Test-2, the dynamic responses of the structural models were examined. Each structure was modeled satisfactorily with a two-dimensional, plane-strain numerical model. Engineering design parameters (EDPs) were computed for key structural responses, including (1) transient peak roof drift, (2) residual roof drift, (3) transient peak displacement and (4) peak acceleration at the center of mass of the structure. Additionally, the acceleration time histories and pseudo-acceleration response spectra at the center of mass of the structure for each motion were examined. These metrics were used to compare the numerically estimated dynamic responses with those recorded in the centrifuge experiments. The dynamic response of the 3-story (prototype) MRF estimated with the numerical model was in close agreement with the observed experimental data for both the SSI (Test-1) and SSSI (Test-2) configurations. The more
complicated 9-story (prototype) model exhibited greater sensitivity to numerical system inputs, including fixed-base fundamental period and applied structural Rayleigh damping. However, the majority of its recorded dynamic responses were well-matched by the numerical model.

The resonant condition created in Test-3 proved challenging to model numerically. The two Test-3 conditions (i.e., SSI and SSSI) were analyzed separately. Significant inertial interaction, including rocking, was observed during the centrifuge test and in the post-processing of data; pseudo-acceleration responses three to five times those recorded in Test-1 and Test-2 were recorded. While the shapes of the pseudo-acceleration response spectra, periods of amplification, and time-histories were well-captured, the numerical model estimated significantly lower amplitudes of the responses for the structures than were observed during the centrifuge test. A sensitivity study was performed to evaluate the influence of several parameters, including (1) the shear wave velocity profile, (2) interface elements, (3) fixed-base fundamental period estimate, and (4) constitutive model parameters. Some of the relative lack of amplification in the numerical simulations was due to over damping in the constitutive model. This was addressed by altering the shear modulus and material damping curves for the soil directly beneath the structures' foundation elements. However, the primary reason for the lower amplitude estimated by the numerical model appeared to be due to the difficulty of capturing the seismic responses of structures in the resonant condition. Shifting the period of any component of the soil-structure system would necessarily have a significant impact on the dynamic response by shifting the system away from resonance. Despite this challenge, the numerical simulations yielded important insights. While the amplitudes of dynamic responses were underestimated for most of the ground motions, the changes in response of the 3-story (prototype) MRF between SSI and SSSI were captured. The elastic shear wall displayed similar behavior; while the spectral shapes were matched for most motions, the amplitudes estimated by the numerical simulations were consistently below those observed in the centrifuge. Comparison of overall change from low- to high-intensity motions or trends from SSI to SSSI could be captured with the model; however, the amplitudes of the responses were generally underestimated. This set of analyses highlighted the challenge of modeling a resonant condition. Additional work is needed to explore the characteristics of the centrifuge when intense input motions are used which are in resonance with the soil in the model.

Finally, two prototypical structures were examined. The first, a 3-story MRF, was the model upon which the centrifuge 3-story (prototype) model was based (Ganuza, 2006). Both solitary (SSI) and adjacent (SSSI) configurations were considered for this prototypical 3-story MRF founded on a dense sand soil column. The dynamic responses of the MRF for the solitary (SSI) condition paralleled those observed in the centrifuge experiments. For the considered configurations of adjacent low-rise structures, SSSI effects were found to be either negligible or only slightly beneficial or detrimental for the five ground motions utilized for dynamic analysis. The other prototypical MRF, a 5-story structure, was a simplified version of a typical, medium-rise structure (Ganuza, 2006). The 5-story MRF exhibited dynamic responses consistent with previous work. Amplification was observed at (1) the first and second modes of vibration of the structure, (2) the site period, and (3) the mean period of the motion. Further research is needed to study and more fully quantify SSSI effects for a wider set of structures, adjacent configurations, and ground motions.
To the three most important people in my life:
  Mom, Dad, and William

  Proverbs 3:5-6
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1 Motivation

Dense urban environments are one of the most complicated systems earthquake engineers must evaluate. With the social trend toward urbanization, new construction is often conducted alongside legacy structures as cities grow to accommodate the influx of residents and commerce. Low-rise buildings share blocks with steel skyscrapers, and shallow mats may be founded beside deep basement parking structures. Each of these structures has a unique seismic response.

Currently, the field of earthquake engineering has been focused on developing methods of quantification for soil-structure interaction (SSI), i.e., the circumstance in which a single structure interacts with its foundation and the soil supporting it during an earthquake. However, in this era of increasing urbanization, a shortcoming of this approach is clear – it examines a solitary structure interacting with a single foundation system. Realistically, structures will almost certainly exist near other structures in modern urban areas.

If the phenomenon of SSI occurs for a single building, then a collection of buildings in an urban environment will each experience individual manifestations of SSI during a seismic event. In such a scenario, the response of one building will likely affect the response of adjacent structures in the system, and vice versa. Therefore, the interactions between buildings during an earthquake are expected to be different than analysis of a single building would predict. Indeed, researchers have found this to be true (e.g., Luco and Contesse, 1973; Wong and Trifunac, 1975; Padrón, 2009).

Interaction between multiple structures during an earthquake is termed structure-soil-structure interaction (SSSI). This area of seismic response has not yet garnered the level of attention commanded by SSI. Instead, earthquake engineers have allocated the majority of research work to more fully quantifying the aspects of a solitary structure’s soil interaction (i.e., SSI).

This focus is due in part to the paucity of case history data of SSSI. Well-instrumented buildings, on well-instrumented soil profiles, in urban environments, which are subjected to strong shaking, are rare. Without sufficient data, it is difficult to evaluate the capability of numerical simulations and to calibrate models and validate predictions.

However, even if recordings were available from an urban center, the level of complexity of such data would likely be so great that isolating SSSI effects with any certainty from the recorded time histories would be difficult. Interactions between seismic waves, foundations, structures, and subsurface inhomogeneities alone would yield time histories with limited application potential to the SSSI phenomenon alone, simply due to the difficulty of isolating those effects.

The NSF-funded NEES City-Block (NCB) research project (of which this dissertation is a part) aimed to address these challenges in three steps. First, a suite of laboratory case histories of SSSI in a well-constrained testing environment were generated. Second, these case histories were used to calibrate numerical models and provide insight into the capabilities of several structural and geotechnical engineering software suites in the context of SSSI modeling. Third, these calibrated models were used to provide insight into more complicated, realistic geometries.
of structural configurations. With this multi-step approach, both physical (centrifuge-generated case histories) and numerical (computer model validation and forward predictions) tools could be used to gain insight into the SSSI phenomenon.

The research presented in this dissertation encompasses the geotechnical numerical modeling aspect of that goal. Data from three of the six planned centrifuge tests was used to calibrate a numerical model in the geotechnical engineering software, FLAC (Itasca, 2005) a finite difference program widely used in industry. The soil profile for the three tests was composed of a dense dry Nevada sand (D_r ~80%). The soil profile was consistent throughout the testing sequence to allow direct comparison of results between Tests-1, -2, and -3. The structures modeled over the course of the three spins included (1) several configurations of a one-story model (three-story prototype) steel moment-resisting frame (MRF) on founded on shallow spread footings, (2) two configurations of a three-story model (nine-story prototype) steel MRF constructed on a deep basement, and (3) a shear rocking wall founded on a shallow mat foundation. Data from the three tests examined:

- **Test-1**: Individual moment resisting frame (MRF) structures on shallow and deep foundations (SSI)
- **Test-2**: Adjacent MRF structures on shallow and deep foundations (SSSI)
- **Test-3**: Adjacent and solitary structures on shallow foundations at resonance (SSI and SSSI)

Tests 1-3 produced dynamic acceleration, yield, and displacement data for a suite of ground motions. These motions included “near-fault” pulse motions from recordings located within approximately 20 km of the fault rupture and “ordinary” ground motions, which in this study are defined as recordings that exhibited no forward-directivity and were located some distance from the rupture. Within these designations, several time histories were scaled and propagated through the centrifuge model at low and high intensities. The MRF structures were designed to exhibit elastic behavior during the initial, low intensity ground motions, and as testing progressed, transition to inelastic behavior for the final, most intense motions. This range of seismic excitations provided a rich data set to provide insight into the characteristics of SSSI modeling in FLAC.

After back-analyzing the centrifuge data, the researcher selected several representative ground motions from the NCB suite for use in a more realistic, multi-bay MRF configuration (SSI and SSSI). This portion of the research examined the response of a structural system too elaborate for the centrifuge modeling environment. The soil profile, originally designed to be representative of the Los Angeles basin, where this city-block concept was based, was used as the profile for this analysis, though extended to a slightly deeper elevation. The results from these analyses provide insight into the changes from SSI to SSSI induced by adjacent structures during seismic events.
1.1 Organization of Dissertation:

The research completed for this dissertation is presented in seven chapters. Each of the following chapters are summarized below:

**Chapter 2.0** A literature review of the current state of practice of soil-structure interaction and structure-soil-structure interaction is presented. Brief descriptions of the analysis tools available for seismic analysis are given. Centrifuge modeling is introduced, and numerical modeling techniques are defined.

**Chapter 3.0** The centrifuge soil model and the suite of ground motions are described. The software used for this analysis, Fast Lagrangian Analysis of Continua (FLAC), is introduced and its capabilities, solution scheme, and limitations are introduced; a comparison of results from FLAC and the commonly used equivalent linear program, SHAKE, is included. The constitutive model selected for the testing sequence is described. Finally, free-field results from FLAC and the centrifuge testing sequence are presented.

**Chapter 4.0** The first structural responses are presented in this section. Physical aspects of the centrifuge model for Test-1 (HBM02) and Test-2 (HBM03) are described. The procedures used to create numerical representations of the physical structures are given. Finally, the results from FLAC are compared with those recorded in the centrifuge, including spectral responses, drift, and footing accelerations.

**Chapter 5.0** This chapter discusses Test-3 (HBM04). This was a unique test, as the system was designed to maximize soil-structure interaction; the structures were in resonance with the soil, which was often near or at resonance with the applied ground motions. As such, results from FLAC are compared to centrifuge output with a discussion of the sensitivity of the numerical model to input parameters near resonance.

**Chapter 6.0** The final chapter contains a collection of more realistic structures. While still simplified, two multi-bay moment resisting frames are modeled with recorded (unaltered) ground motions. Several configurations of structures are examined and the results of structure interaction are examined.

**Chapter 7.0** A summary of the results is presented, with a discussion of the most significant beneficial and detrimental SSSI effects observed. Cases for which SSSI seemed to be negligible are also noted. Recommendations for future work are provided.

Representative plots are presented in the body of the dissertation where appropriate. However, due to the large volume of figures generated in these analyses, the appendices contain all responses, figures, and computer code generated in this research. All pseudo-acceleration response spectra for the centrifuge tests, time history comparisons, and the programs used to analyze data are included.
2 Introduction

Many cities exist in seismically hazardous regions. As relocation is neither desirable nor practical, these metropolises present the modern earthquake engineer with the challenge of creating safe, robust structures, and infrastructure capable of withstanding inevitable seismic events. Thus, the engineer must evaluate seismic performance in dense, urban environments.

2.1 Soil Structure Interaction Background

The relationship between a site and a building has been recognized to be of pivotal importance in evaluating how a structure will perform during an earthquake. The interaction between a site, a foundation, and a superstructure has been termed soil structure interaction (SSI) or soil-foundation-structure interaction (SFSI). This phenomenon is extremely important in seismically active regions. Publications providing background regarding this phenomenon range from the inceptive works of Jennings (1970), Jennings and Bielak (1973), Veletsos and Meek (1974), Seed et al. (1975) to the current works of Kramer and Stewart (2004) and Mylonakis et al. (2006). The reader is referred to the work of Mason (2011) for an excellent and extensive literature review on this topic. This dissertation, which follows from that of Mason (2011), will focus on relevant aspects of SSI, SFSI, and SSSI to this research.

2.1.1 Historical Key Factors in Soil-Structure Interaction

Earthquake engineers have long recognized that soil and structural properties both play critical roles in determining the degree of SSI, and consequently, the degree of damage, likely to occur in a system during an earthquake. These factors include (Veletsos and Meek, 1974, for points 1-3):

1) A parameter which takes into account the system properties. This often includes the speed at which shear waves travel at a site for the subsurface component of SSI, and building dimensions and/or properties for the structural components of the system.
2) Some ratio of building height to foundation dimension.
3) The natural frequency of the structure to and the underlying soil column to the ground motion (or exciting frequency).
4) The relative stiffness of the superstructure to the subsurface

SSI essentially describes the effects between the interface of the built and natural environments, examining the way soil influences a structure and vice versa. This phenomenon is often further discretized by dividing the effect into two main categories: inertial and kinematic effects.

2.1.2 Kinematic Interaction

Kinematic interaction is defined as the difference between the ground motion recorded at the foundation level of a structure and that which would have been expected if the structure had not been present (Stewart, 1999). The “building-free” condition is commonly referred to as the free-field case; in other words, it represents a location far enough removed from all structures that the recorded ground motion exhibits no man-made effects in the time history. The free-field motion includes the effects of local site conditions, earthquake characteristics (including source mechanism, location, magnitude), and subsurface effects (most significantly, wave travel paths). Significantly, kinematic interaction is not dependent on the mass (or weight) of the system.
Theoretically, if a massless foundation and superstructure were placed at a site, kinematic interaction would still occur (Fenves and Serino, 1994).

Kinematic interaction is due to three main effects. The first is caused by foundation embedment; the depth of a structure’s foundation will necessarily affect the response, as ground motion amplitudes differ with depth. The second is due to base slab averaging, essentially the recognition that when seismic waves interact with a structure’s foundation, their effects are “averaged” or smoothed across the footprint to some degree. The third is general wave incoherence; no two locations on a foundation will be subjected to identical ground motions – some variation will always be present (Stewart et al., 1999). This interaction is usually most significant for higher frequency ranges. In part, this is due to the fact that longer wavelengths (those on the order of kilometers, for example) are not affected by or do not affect the response of a relatively small foundation.

### 2.1.3 Inertial Interaction

Of the two SSI effects, inertial interaction is the more commonly addressed. Inertial interaction refers to the building’s movement and the resulting interaction between the ground, time history, and superstructure. This may include rocking, pounding, sliding of the superstructure, as well as changes in the recorded ground motion due to energy imparted from the oscillating building to the subsurface, both during and immediately after strong shaking. Unlike kinematic interaction, inertial interaction is tied directly to a structure’s mass.

The potential of a system to exhibit inertial interaction may be examined with parameters that compare the relative stiffnesses or masses of the soil and structural components of the system (Stewart et al., 1999). The soil-to-structure stiffness dimensionless parameter commonly used is:

**Equation 2-1**

\[ \sigma = \frac{V_s T}{h} \]

with \( h \) representing the effective height of the building above the foundation, \( T \) is the fixed-base natural period of the structure, and \( V_s \) is the shear wave velocity which is most representative of the subsurface (Stewart et al. 1999, Veletsos and Nair, 1975, & Bielak, 1975).

The relationship between soil and structure can also be examined from a mass standpoint. The dimensionless parameter recommended for that condition may be computed with:

**Equation 2-2**

\[ \gamma = \frac{m}{\rho ar u^2 h} \]

where \( m \) is the mass of the structure for the fundamental mode of vibration, \( \rho \) is the representative mass density of the soil profile, \( r_u \) is the radius of foundation that matches the area of an assumed circular foundation to the actual foundation area, and \( h \) is the effective height of the building above the foundation, as defined previously (Stewart et al. 1999, Veletsos and Nair, 1975, Bielak, 1975). Although the inertial interaction effects begin when the ground shakes the building due to a subsurface excitation, the effects continue as the building begins to shake. A rocking superstructure will necessarily introduce some perturbations in the subsurface, thus “shaking” the soil. This soil-to-structure, structure-to-soil transfer of energy may continue during
and for some time after shaking. In other words, a type of feedback loop may be established between the subsurface and the superstructure itself during excitation.

2.2 Soil Structure Interaction: State of Practice

The current earthquake engineering state-of-practice with regard to soil-structure-interaction is often to ignore the phenomenon, except for highly sensitive or large-scale projects. There is a historical precedent for this approach. In the past, neglecting SSI has been considered conservative, for three main reasons, which are discussed below (Mylonakis et. al, 2006).

The first reason is that a structure’s period is generally lengthened (in comparison to its fixed-base period) when soil and foundation influences are considered. By lowering the natural frequency of the structure, the building’s fundamental period is moved to the longer period region of the engineering design response spectrum. This region generally contains lower pseudo-acceleration response values, thus allowing the engineer to design the structure to a reduced response value. Therefore, if the shorter fixed-base period is used (e.g., SSI is ignored), the engineer may claim to have designed the structure to a “more conservative,” higher response value than would have been deemed necessary if SSI had been considered.

The second reason is that the soil acts as an additional damping mechanism. With its nonlinear response and relatively soft material properties (in comparison to structural components), soil is a source of energy dissipation not directly present in fixed-base analyses. Foundation impedances used to approximate this dissipation tend to over-predict the damping levels observed in reality (Mylonakis and Gazetas, 2000).

The third reason is due to response modification coefficients ($R$). These factors are used to decrease seismic loading on a structure estimated with an elastic response, yielding what are taken to be more realistic forces on the structure (ATC-2003). These reduced forces are commonly used for design.
However, ignoring SSI cannot be considered to be universally conservative. Indeed, a change toward thoughtful inclusion of the potential benefits as well as detrimental effects of SSI in industry is slowly manifesting itself, as demonstrated in the commentary from NEHRP regarding FEMA P-750:

*The dependence of the response on the structure-foundation-soil system is referred to as soil-structure interaction. Such interactions will usually, but not always, result in a reduction of base shear.* [emphasis, added]

For some cases, a stiff building may be pushed into higher spectral response regions than expected for its fixed-base period when an SSI-lengthened period is computed. Also, the period of the site should be considered. In some cases, lengthening the building period may push the structure into a resonant condition with the soil column. An example of this is what occurred in Mexico City in 1985. Founded on a deep alluvial deposit, the heavy damage sustained by buildings in the area was closely correlated to the relationship between the period of the structure and the underlying soil deposit (Aviles and Perez-Rocha, 1998). Severe shaking in excess of design standards was experienced by structures that had periods near the resonant frequency of the soil profile.

Another example of non-beneficial period-dependence is shown by the failure of the 630 m span of the Hanshin Expressway in Japan during the 1995 Kobe event. This dramatic failure was examined by Mylonakis et al. (2006) with an eye to the effect that SSI had on the bridge’s response. Five representative motions located near the structure’s site were selected for examination of this case history (as well as one synthetic motion). The “typical” fixed-base (T_{fixed}) and soil-structure interaction (T_{SSI}) periods for a single bridge pier were estimated to

![Figure 2-1: Reduction in base shear, recommended by NERHP-1997 seismic code (Gazetas and Mylonakis, 2000)](image_url)
be 0.84 s and 1.04 s, respectively. Figure 2-2 shows the pseudo-acceleration response spectra for the five recorded ground motions. The spectral responses associated with the periods $T_{\text{Fixed}}$ and $T_{\text{SSI}}$ are marked for each motion. As may be observed, the response at $T_{\text{SSI}}$ was lower than that associated with $T_{\text{Fixed}}$ for two motions (JMA and Fukiai). However, shifting from $T_{\text{Fixed}}$ to $T_{\text{SSI}}$ significantly increased the spectral response for the remaining motions (Takatori, Higashi Kobe, and Motoyama). For this scenario, ignoring SSI would likely have been unconservative.

In other cases, the SSI period may present a problem for a particular governing ground motion. If the frequencies of the ground motion and the structure align, resonance could occur, and levels of shaking in excess of those used for design could occur. Also, the role of soil extends beyond increased damping; the nonlinearity of the soil response, the foundation type and its interaction with the subsurface, and the building geometry all affect the level of damping a soil provides, and may implement other changes to the fixed-base computed response (Takewaki, 1998).

Figure 2-2: An example of the effect of SSI on bridge responses due to the 1995 Tokyo earthquake. For some ground motions recorded near the site of interest (JMA, Fukiai), a shift in period to $T_{\text{SSI}}$ proves advantageous. For others deemed most representative of actual site conditions (Takatori, Higashi Kobe, Motoyama), the shift is seriously detrimental. (Mylonakis and Gazetas, 2000)
As the emerging theory of self-righting structures develops, one can envision using SSI to one’s advantage. Consider the example of shallow foundations – specifically, footings. Currently, such foundations are designed to maintain a specified level of performance during a seismic event, remaining in the elastic realm. These very robust systems are strong and perform well as long as they remain within the bounds of the levels of seismic demand which they were designed to withstand. However, when failure occurs, usually at the interface between column and foundation due to plastic hinging, it is often catastrophic (Figure 2-3). A new concept for seismic design presents a revolutionary approach by postulating that SSI-induced moments could be used as self-righting mechanisms – instead of “overdesigning” structures to prevent hinging at the base of columns, “plastic hinging” could be encouraged in subsurface materials. As illustrated in Figure 2-3, carefully designing structures to allow failure mechanisms to occur in ways that minimize structural damage for small events and increase the life-safety during rare events seems possible, and requires SSI as a design parameter (Anastoupoulos, 2010). To make use of SSI in this manner, however, an engineer must have a thorough understanding of the effects, significance, and nuances of the phenomenon.

In recent years, numerical analyses have been undertaken to attempt to better quantify the likelihood of significant SSI effects. Findings from numerical simulations and seismic studies confirm that SSI may be beneficial or detrimental, depending on factors which include (Mayoral and Ramirez, 2011):

- Natural period of vibration of the structural system
- Relative stiffness of the surrounding soil to the structural elements
- Soil nonlinearity and ground motion variability

Therefore, more and more frequently, earthquake engineers are concluding that SSI cannot be neglected. In cases when SSI is considered, two main approaches are available.

The first approach is a substructure or multistep approach. Implementation of this procedure begins with estimation of the free field response of the soil, through careful modeling of the material properties of the subsurface and propagation of a motion through a calibrated soil profile. Next, the free-field motion is combined with transfer functions to generate a foundation input motion. The soil and structure interface is modeled with calibrated springs and dashpots to mimic the stiffness and damping of the system, respectively. Finally, the structural response is estimated with the foundation input motion applied to the calibrated interface elements (Kramer and Stewart, 2004).

The other method is the direct approach. As may be inferred from its designation, this procedure analyzes the system as a whole. The soil is represented as a continuum and the foundation and superstructure are modeled simultaneously. While significantly more time-intensive, this can provide a robust and detailed examination of a system’s dynamic behavior. Site, structure, and ground motion effects may all be examined, and sensitivity to system components can be explored as well as true soil and structure nonlinearities. The direct approach is usually implemented with finite-element (FEM), boundary element (BEM), or finite-difference (FDM) software (Rizos and Wang, 2002).
One of the challenges yet to be overcome regarding implementation of the direct method is the lack of a single, unifying program for all earthquake engineers. In general, geotechnical engineers and structural engineers use different software suites; each field has several specialized tools for examination of dynamic response. Academic development of a program palatable to both structural and geotechnical engineers is ongoing; the software package, OpenSees (http://opensees.berkeley.edu) is currently aiming to develop equally robust soil and structural components. Commercially available models are also attempting to better capture the interface between structural and geotechnical design. However, the current state-of-practice remains largely a divided numerical analysis field.

Another challenge in implementation of the direct method is the level of expertise necessary to perform meaningful analyses. The time required to calibrate materials, generate meshes, and model soil-structure interaction problems numerically is significant. The skill set and temporal requirements limit the use of this method – for medium to low-rise buildings, the
cost is usually too large to implement the direct method. As such, there is a need to examine the ability of more accessible, widely available programs to model dynamic problems.

### 2.2.1 Direct Approach: Numerical Analysis

There are three main solution schemes used when conducting numerical seismic analyses. In some situations, the engineer may elect to utilize software based on the finite element method. For other scenarios, a boundary element approach may be feasible. Alternatively, a computer model using finite difference scheme may be most appropriate. The three approaches have strengths and limitations, which should be taken into account when selecting and implementing either method.

#### 2.2.1.1 Finite Element, Boundary Element, and Finite Difference Overview

For FEM, BEM, and FDM, the soil is represented as a material based on physical parameters, including moduli, stiffness, damping ratios, as well as numerical model-specific calibration values. Each of these methods is used to solve the system’s governing differential equations. The differences refer to the method by which the differential equations are approximated.

The FEM discretizes a continuum into regions with defined elements and nodes. Quantities of interest are described with interpolation functions, which are defined along the length of the element. These interpolation functions are determined by the user, and take boundary conditions into account (Cook, 2002).

The BEM also discretizes the continuum. However, in this procedure, only the boundary is discretized into elements. This reduces the dimensionality of the problem by one; for a two-dimensional analysis, that is simply a line, for a three dimensional analysis, that is a surface. This greatly reduces the number of algebraic equations to be solved (Rizos and Wang, 2002).

The FDM is a conceptually simpler method. It discretizes a continuum by representing the exact governing differential equations as approximate algebraic differences between grid points. The method is relatively easy to understand. However, it does have limitations, including a greater sensitivity to grid size and mesh geometry (Gerya, 2010).

FDM and FEM have a significant advantage when considering the geological aspect of the SSI phenomenon, as they can rather easily account for inhomogeneity in the subsurface material and geometry. BEM, in contrast, uses significantly more complex mathematical equations for implementation and is currently restricted to linear elastic and viscoelastic soil models (Pitilakis et al., 2008).

All three approaches have the same goal: to discretize a continuum for solution. Each method introduces error; it is important to remember that the solutions computed are approximations. Ultimately, the user must decide whether the approximation is of value for the specific scenario and system under examination.

#### 2.2.1.2 Eulerian and Lagrangian Grid Points

Within a numerical solution method, there is also a dependency on the definition of grid points for deformation. Two procedures are commonly available to the engineer: Eulerian and Lagrangian solution schemes. The Eulerian approach defines a grid and allows no deformation to
occur. If the material being modeled deforms, it is free to flow past grid points; however, the geometry of the overall mesh will not change. Conversely, a Lagrangian grid will deform according to the behavior of the material associated with that grid point. Therefore, if the material flows, the grid will flow as well (Geryas, 2010).

2.2.1.3 Explicit and Implicit Solution Schemes
Finally, the numerical iteration scheme used by a numerical program to solve for a system state has two available approaches. Explicit methods determine the solutions of the governing equations for a future time based on the current system state. Conversely, implicit methods solve for the current and future states simultaneously through coupled sets of equations. Each method has advantages – for example, the explicit method is simpler to implement, but is dependent on the time-step (they are conditionally stable). The implicit method does not impose a theoretical limit the time-step, but is more computationally expensive (NAFEMS, Fundamentals of Numerical Techniques for Static, Dynamic, and Transient Analysis – Part 2).

2.2.2 Numerical Analysis of SSI: Soil Constitutive Models
Numerical modeling of soil-structure interaction requires a way to capture the mechanics of soil reaction. Different approaches are available, from comparatively simple springs and dashpots to fully-nonlinear anisotropic constitutive models. Two selected approaches are presented below.

2.2.2.1 Soil Representation: Linear Elastic Model
One common approach is to assign a series of springs and dashpots to represent the interface between soil media and structure. Winkler’s idealization is commonly used; for this approach, the soil is approximated by a series of linear elastic springs. These springs connect the foundation elements to a rigid base. Any induced deformation or deflection is limited to loaded regions, and the pressure, \( p \), induced by the weight of the superstructure, \( w \), is related with the coefficient of subgrade reaction (or subgrade modulus), \( k \), by Equation 2-3 (Dutta and Roy, 2002).

**Equation 2-3**

\[
p = kw
\]

This simplified method can provide meaningful insight into the soil-structure system with careful selection of the subgrade modulus. However, it is that – simplified. The system’s response cannot be captured fully by such a model.

2.2.2.2 Soil Representation: Continuum Approach
Soil is an inherently complex system; it exhibits nonlinearity, irrecoverable strains, stress dependency, strain-dependent behavior, and many other phenomena. The civil engineering field has soil constitutive models that can capture important aspects of static and dynamic soil response. The priorities for such models include capturing deformation, settlement, or lateral movement of a soil deposit, which involves accurately predicting stresses, strains, and yield strength. Currently, however, a complete constitutive model for soil does not exist. Instead, various specialized models are available for use (e.g., granular versus fine-grained, saturated versus unsaturated). Within soil types and conditions, some models are even more focused, designed to accurately predict a type of response (e.g., lateral deformation or pore-pressure dissipation) (Dutta and Roy, 2002).
2.2.2.1 Soil Representation: Constitutive Models

Within the soil continuum, stresses and strains are related through a series of equations collected and organized into what is termed a constitutive model. These constitutive models allow engineers to model soil conditions and gain insight into the response of soil and structural systems.

Constitutive models range from the simplest elastic-perfectly plastic to fully nonlinear, anisotropic strain-softening models. Complicated models may allow for more flexibility in use and calibration; however, use of these numerical schemes often introduces a much higher level of specialization of results. Simpler models may not be capable of modeling some phenomena, but can give good overall results to provide insight into a response. A more complicated model is not always the optimal choice (Lade, 2005).

Each constitutive model is optimized for some condition, whether a particular type of soil, a stress-state, or to best capture a response of interest (e.g., liquefaction or lateral spreading). Care must be taken in selecting the appropriate model for the scenario being analyzed. These models must also be calibrated to the soil state, with either laboratory or site-specific data.

In this project, the primary aim was to capture dynamic site-response. Accurate transmission of vertically propagating, horizontally polarized shear waves was the highest priority for this SSSI examination. Selection of a constitutive model was based on its ability to accurately capture this transmission in the free-field (discussed in more depth in Chapter 3).

2.3 Centrifuge Modeling

Earthquake engineers face a considerable limitation with regard to the amount of data available for analysis. Well-documented, technical case histories of seismic events are relatively few. Data captured during a seismic event, from heavily instrumented structures, near heavily instrumented soil profiles, are even rarer. As it is impossible for engineers or scientists to predict precisely the location or time of an earthquake, adding to the global suite of data is a slow process. This has been demonstrated again and again; perhaps one of the most famous examples is of the Parkfield event (USGS, http://earthquake.usgs.gov/research/parkfield/).

Engineers have thus turned to laboratory tests to study seismic building responses. Shaking tables, for example, are now routinely used for analysis of structural responses to ground motions. However, the laboratory approach presents challenges to geotechnical-focused problems and systems which contain an influential soil component. Geotechnical responses are inextricably linked to the stress state of a soil; therefore capturing the large confining stresses experienced at even relatively shallow depths is pivotal for accurate modeling of soil and structure systems.

This stress-dependency is a key challenge in laboratory geotechnical modeling. Structural systems may be scaled down to a manageable model level and meaningful results obtained. However, inducing prototypical stress distribution in a model-scale soil profile is not so straightforward. Nor are the relationships between soil behavior under stresses due to a smaller (model) load easily related to those expected at the actual site of interest. Therefore, one of the
most valuable laboratory tools for geotechnical use in these applications is the geotechnical centrifuge, which allows one-to-one scaling of stress distribution.

2.3.1 Centrifuge Mechanics and Scaling Laws

A centrifuge is composed of an arm, one end of which is attached to a fixed point. This provides a pivot about which the arm spins. From the other end, a bucket or similar apparatus is suspended. The centrifuge operates by applying a centrifugal force to the suspended bucket at the end of the centrifuge arm.

Applying a centrifugal force may be thought of as applying a form of gravitational force in excess of the standard 9.81 m/s² (or 1 gravitational unit, g, on Earth). By scaling gravity, both time and length are affected; they are scaled according laws to be explained presently. As a result, the stresses induced in the centrifuge soil model will scale in such a way as to match the stresses expected for a much larger model at a standard gravitational state.

If the level of acceleration is described by a constant scale factor “N”, the acceleration experienced by the model may be g x N. Thus, when spinning, gravity in the centrifuge model is N times larger than the standard static 1g (Kutter, 1994; Kutter 1995). This scaling of stresses allows a centrifuge test to model a fully-coupled soil-structure system in a uniquely realistic manner. Therefore, physical centrifuge modeling was used to generate the desired suite of case histories for numerical calibration and validation. The scaling laws for this experiment are included in Chapter 3. Further information on this topic is beyond the scope of this dissertation and may be found in Mason (2011).

2.4 Ground Motions

Structures and soil conditions represent two components of the SSI system. There is a third, however, which can be equally important if not more important – the ground motion. Critical factors that influence ground motions are near-fault directivity and site effects.

2.4.1 Directivity: Forward and Backward

Directivity is a near-fault effect, and results from the travel of seismic waves during rupture (Somerville et al. 1997). During rupture, displacement occurs along the fault plane. The speed at which the rupture propagates is approximately 80% of the shear wave velocity of the subsurface. Therefore, the shear waves generated during rupture travel at a speed very near the speed at which new seismic waves are generated.

Along the plane of rupture, two types of directivity can results from this overlap. Forward directivity occurs when the rupture and shear waves are traveling in the same direction, toward the site of interest. As shown in Figure 2-4, the shear waves create a wave front as the similar speeds of travel and fault rupture cause a buildup of seismic energy. This results in a high-intensity motion, with a short duration.
In contrast, backward directivity occurs when the rupture travels away from the site of interest (Figure 2-4). The motions resulting from this configuration are usually less intense, but have a much longer duration.

Forward directivity has been recognized as a significant factor in the level of damage likely to be sustained during an event in the near-fault region, usually identified as approximately 20 km from the plane of rupture (Bray and Rodriguez-Marek, 2004). There are several reasons for this. First, these motions usually contain a large amplitude velocity pulse; many researchers have observed that these pulse-like motions are extremely significant in determining the seismic performance of a structure (Hall, 1995; Bray and Rodriguez-Marek, 2004). Second, the energy from the earthquake is concentrated at a certain period. If the pulse-period of an event aligns with a structure’s fundamental period of vibration, a much stronger response than expected from acceleration analyses may occur (Bray and Rodriguez-Marek, 2004). Lastly, the response of a structure can be influenced by its period relative to the earthquake’s pulse-period (Alavi and Krawinkler, 2001).

Backward directivity motions generally command less attention during design. The peak accelerations and velocities are lower relative to those in forward directivity motions; as such,
these motions are usually less damaging to the built environment. However, due to the long duration and repeated cycles of loading, backward directivity ground motions can induce cyclic mobility. Thus, in areas with liquefaction potential, these motions can be an equally or more serious hazard than a more intense, but shorter ground motion.

### 2.4.2 Ground Motion Selection

The magnitude of a seismic event is not a sufficient descriptor for the response of a system. As such, it was important to the research team to include a variety of ground motions in the testing sequence. Both forward-directivity “near-fault” and “ordinary” ground motions were included. The “ordinary” designator was applied to those motions that exhibited no directivity effects. Additionally, the intensity of a motion is a significant factor in the response of a built environment. To examine the effects of amplitude, several of the ground motions were scaled to different acceleration amplitudes. Each of the ground motions was representative of what might be expected to occur in the Los Angeles, CA basin, where the city block concept was imagined to be located. The motions are presented below (Table 2-1). A complete explanation of the method of selection and treatment of the ground motion suite may be found in Mason (2011).
Table 2-1: Summary of ground motion suite

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Location of Recording (Designator)</th>
<th>Type of Motion (Intensity)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landers (1992)</td>
<td>Joshua Tree (JOSL)</td>
<td>Ordinary (Low)</td>
</tr>
<tr>
<td></td>
<td>Joshua Tree (JOSH)</td>
<td>Ordinary (High)</td>
</tr>
<tr>
<td></td>
<td>Lucerne (LCN)</td>
<td>Near Fault</td>
</tr>
<tr>
<td>Chi Chi (1999)</td>
<td>TCU078 (TCUL)</td>
<td>Ordinary (Low)</td>
</tr>
<tr>
<td></td>
<td>TCU078 (TCUH)</td>
<td>Ordinary (High)</td>
</tr>
<tr>
<td>Northridge (1994)</td>
<td>Rinaldi Recording Station (RRS)</td>
<td>Near Fault</td>
</tr>
<tr>
<td></td>
<td>Sylmar Converter Station (SCSL)</td>
<td>Near Fault (Low)</td>
</tr>
<tr>
<td></td>
<td>Sylmar Converter Station (SCSH)</td>
<td>Near Fault (High)</td>
</tr>
<tr>
<td></td>
<td>Newhall (WPI)</td>
<td>Near Fault</td>
</tr>
<tr>
<td>Superstition Hills (1987)</td>
<td>Parachute Test Site (PTS)</td>
<td>Ordinary</td>
</tr>
<tr>
<td>Loma Prieta (1989)</td>
<td>Saratoga West Valley College (WVC)</td>
<td>Near Fault</td>
</tr>
<tr>
<td>Kobe (1995)</td>
<td>Port Island Modified (PRImod)</td>
<td>Near Fault</td>
</tr>
</tbody>
</table>

2.5 Summary of Motivation

In summary, ignoring the effects of SFSI is no longer regarded as universally conservative. As the field of earthquake engineering moves toward implementation of the direct method, careful examination of the robustness of the tools for this analysis is needed. A dearth of well-documented case histories of SSI affects the speed at which this examination can move forward. By using “model case-histories” that are generated by the physical modeling component of the NEESR research team, the robustness of a widely available geotechnical engineering software analysis package can be examined. After comparison with lab-generated data, sound evaluations regarding seismic responses of prototypical structures to selected ground motions could be made.
3 Constitutive Modeling

The first step in the numerical SSI and SSSI analyses was to select and to calibrate an appropriate soil constitutive model. This procedure included extensive comparisons between results from constitutive models currently implemented in FLAC, laboratory data, and well-constrained linear-elastic model predictions. The selected constitutive model was then matched to the achieved soil profiles in the centrifuge modeling suite. Finally, pseudo-acceleration response spectra from FLAC and the centrifuge free-field data were compared.

3.1 Introduction: Why Centrifuge Case Histories Are Needed

As mentioned, SSI is a complex, multifaceted phenomenon. The uncertainty associated with precisely identifying all components of a naturally layered subsurface alone presents challenges to SSI and SSSI analysis in the field. Earthquake variability and fault rupture uncertainties present added incertitude to the study of this phenomenon in practice.

Borings, CPTs, SPTs, and laboratory testing on carefully obtained soil samples are all excellent tools for gathering information about a site’s soil conditions. However, the proximity and number of testing methods will never be able to capture perfectly the soil horizon. In reality, geotechnical engineers must exercise judgment and extrapolate likely geology between borings. This presents a challenge to SSSI case history development, as a perfect knowledge of the precise age, geometry of deposit, density, and saturation condition is not possible for any location any distance from an exploratory boring.

Additionally, subsurface instrumentation is costly. The relative infrequency of large ground motions, the variability associated with their locations, and the comparatively recent advent of instrumentation able to record seismic events combine to limit the number of well-instrumented soil columns in existence. Indeed, as the Park City, California project demonstrated, even extremely well-constrained faults can be unpredictable (USGS, http://earthquake.usgs.gov/research/parkfield/).

These facts highlight several distinct advantages of using laboratory generated “case history” data from a tool like the centrifuge at the CGM at the University of California, Davis. Researchers are sure not only of the input ground motion, but also of the material type, depositional history, age, saturation, and relative density of the soil profile to be tested. Finally, extensive instrumentation of the soil model can yield information about dynamic soil behavior for a range of ground motions (e.g., near-fault, ordinary, high and low intensities).

3.2 Centrifuge (Physical) Model Overview

For this testing sequence, the soil profile was composed of dense, dry Nevada sand. A uniform relative density of approximately 80% was targeted throughout the profile. Nevada sand is a naturally mined sand; as such, the properties of the material are known to vary from excavated batch to batch. Therefore, it is very important to identify the properties associated with the batch of sand that will be used for the duration of a testing sequence. The Nevada sand used in this project was sourced from Gordon Sand Co., in Compton, California. The properties were as follows (as reported by Cooper Labs):
<table>
<thead>
<tr>
<th>Material Property</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>---</td>
<td>2.65</td>
</tr>
<tr>
<td>Mean Grain Size</td>
<td>Mm</td>
<td>0.14 to 0.17</td>
</tr>
<tr>
<td>Maximum Void Ratio</td>
<td>---</td>
<td>0.748</td>
</tr>
<tr>
<td>Minimum Void Ratio</td>
<td>---</td>
<td>0.510</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>°</td>
<td>40</td>
</tr>
<tr>
<td>(estimated for D_r=80%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coefficient of Uniformity</td>
<td>---</td>
<td>1.67</td>
</tr>
<tr>
<td>Classification</td>
<td>Uniform, fine sand (SP)</td>
<td></td>
</tr>
<tr>
<td>Gradation</td>
<td>Poor</td>
<td></td>
</tr>
</tbody>
</table>

The sand was deposited through air pluviation. Relative density was checked regularly during specimen preparation with the following procedure. At predetermined lift thicknesses, pluviation was paused. The CGM flexible shear beam container (FSB2.2) and the soil it contained were weighed. Then, the weight of the empty box (determined prior to specimen preparation) was subtracted from that total. Finally, the total mass of the soil was divided by the volume of the box. This procedure allowed the researchers to monitor the relative density being achieved. If deviations from the target density were observed, the pluviating apparatus was adjusted to deposit sand at a faster or slower rate, as appropriate. For complete details on lab specimen preparation, the reader is referred to Mason (2011).

3.2.1 Soil Instrumentation

Soil instrumentation was placed in the specimen during the pluviation stage. At most lift elevations, some accelerometers and linear potentiometers were carefully located at predetermined areas of interest. Since soil and structure interaction was the key phenomenon of interest during this test, instrumentation was concentrated at depths within two foundation widths of the surface.

Vertically oriented lines of horizontal accelerometers were placed within the soil profile for each test. These arrays provided information regarding site-specific seismic characteristics. The vertical propagation of seismic waves, particularly horizontally polarized shear waves, is a key parameter for design in earthquake engineering. The interaction between structures, the effects of embedment and foundation type, as well as the general frequency content with travel through the profile were other key behaviors of interest captured by these instrumentation configurations.
Horizontally oriented arrays of vertical accelerometers were included as well. These were concentrated near the surface, to examine the inertial and kinematic effects of structures’ foundations. Vertical accelerometers were also placed directly on footings to compare the differential movements and transient displacements of various footings during shaking.

Both integrated circuit-piezometer (ICP) and micro electro-mechanical system (MEMS) accelerometers were utilized in this test. The soil profile for Test-1, Test-2, and Test-3 contained ICP accelerometers, exclusively. The accelerometers were in the $+/-50$ g and $+/-100$ g ranges.

3.2.1.1 Soil Instrumentation: Accelerometers

As mentioned, accelerometers were used to record the vertical propagation of ground motion through the soil model, as well as the structural responses and horizontal variation of motion. Each of these accelerometers was monitored during the testing sequence; a schematic of the locations of instrumentation for Test-1, Test-2, and Test-3 are shown in Figures 3-1 and 3-2.

![Figure 3-1: Accelerometer Locations, Test-1 (a) and Test-2 (b) (Mason, 2011)](image-url)
Figure 3-2: Accelerometer configurations, Test-3 (Mason, 2011).

3.2.1.2 Soil Instrumentation: Time History Filtering

Several sensitivities of ICP and MEMs accelerometers were used in the centrifuge model. Each of these instruments recorded some level of noise. This is a universal and effectively unavoidable side effect of using electronic equipment during any laboratory experimentation. As a result, proper filtering of the signal was a pivotal aspect of post-processing data interpretation.

The researchers used fifth-order Butterworth bandpass filters for all accelerometers. A lower corner frequency of 8Hz (0.145Hz prototype) and an upper corner frequency of 1375 Hz model scale (25Hz prototype) were specified. These filters were applied with a MathCAD data processing sheet, which is included in Appendix A.

The filtered acceleration time histories were then converted to prototype scale from the model scale recordings. The scaling factor for acceleration, which is in the units of [L]/[T^2], is 1/55 (for more information on scaling laws, see Chapter 2, and Mason 2011). Lastly, the data recorded during the centrifuge contained variable time steps. To correct this, linear interpolation was used in MATLAB to create files with uniform time intervals (See Appendix A for MATLAB processing files). Once the acceleration time histories were filtered, scaled, and interpolated, they were ready to be used as input histories to numerical models.

Table 3-2: Summary of relevant scaling laws for Test-1, Test-2, and Test-3

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model to Prototype scaling factor (Model/Prototype)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>1/55</td>
</tr>
<tr>
<td>Density</td>
<td>1</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
</tr>
<tr>
<td>Dynamic Time</td>
<td>1/55</td>
</tr>
<tr>
<td>Period</td>
<td>1/55</td>
</tr>
<tr>
<td>Frequency</td>
<td>55</td>
</tr>
</tbody>
</table>
3.3 Numerical Software Overview

Specimen preparation yielded a nearly homogeneous soil profile. While not perfectly uniform, the variation in relative density from lift to lift was within several percent of the target. As a result, a single set of soil constitutive model parameters was capable of capturing the response of this relatively uniform, essentially homogeneous soil.

Two main software suites were utilized during this phase of free-field numerical analysis. The first was the widely used equivalent-linear software, SHAKE2000 (Ordonez, 2000). This provided a well-understood seismic site response result that was compared with the output from more complex software. The primary software tool for analysis in this study was Itasca’s FLAC (Itasca, 2005), a finite difference program capable of modeling dynamically-induced soil non-linearity and structural elements. Also, FLAC has an interface capability which allows separation and slip between the grid and structural components. This was a key consideration when selecting the software for this analysis. Foundation rocking and sliding were expected during some ground motions; the ability to model this was deemed pivotal to accurately estimate inertial interaction during those events.

3.3.1 SHAKE: Equivalent Linear Method

As previously stated, one of the most commonly utilized tools for dynamic soil analysis is the software SHAKE2000 (Ordonez, 2000). The program uses an equivalent linear method to propagate vertically traveling, horizontally polarized shear waves through a one dimensional column of soil. The equivalent linear approach is a basic method that requires several simplifying assumptions; the most significant of these are listed below.

First, SHAKE2000 uses total stresses for computations; pore pressure, water table, and other phreatic conditions are not considered. Second, the hysteretic response of the soil profile is described by a single equivalent shear modulus (G) and equivalent viscous damping ratio (λ) for each layer. This means that the model is sensitive to the user’s definition of the number, thicknesses, and properties of each soil layer. The stratigraphy and shear wave velocity profile of a site provide information regarding the subsurface discretization into appropriate layers for the SHAKE2000 software.

The shear modulus reduction curves describe the response of the soil as shear strain occurs; these may come from predefined curves in SHAKE2000 or used-defined, soil specific data. With $\rho_i$ (density), $G/G_{\text{max},i}$, $\lambda_i$, and $h_i$ (layer thickness) for $N$ layers, SHAKE2000 solves the wave equation through an iterative process using the complex stiffness method. The initial estimates of damping ratio and shear modulus are used to compute shear strain histories for each layer in the soil column. These shear strains are used to find effective shear strain values, which are then used to compute new $G$ and $\lambda$ values. The new estimates of $G$ and $\lambda$ are then used for the next iteration. This process is repeated until the values of $G$ and $\lambda$ converge to those necessary to yield the effective shear strains predicted by SHAKE2000 (Stewart, 2008).

Despite the limitations and simplifications inherent in the equivalent linear approach, SHAKE2000 can provide a good approximation of the dynamic free-field site response. As a result, the program was used for comparison purposes in this research. The output from SHAKE2000 was compared with predicted responses from FLAC to help estimate the shear wave velocity profile of the physical system.
3.3.2 FLAC: Explicit Finite Difference Method

The main numerical analysis for this project was completed with the finite difference software, FLAC. This program uses an explicit finite-difference method to simulate civil engineering systems. All analyses are in two-dimensions, and assume a plane strain condition. An explicit Lagrangian solution scheme is utilized, allowing complex systems expected to undergo large deformations to be analyzed with a high degree of accuracy.

FLAC was originally designed for use in mining and geotechnical applications. It has continued to build on that foundation. Many user-defined constitutive models have been implemented in the program, a dynamic capability has been added, and structural components have been expanded.

3.4 Constitutive Model Overview

Soil is a complex combination of three main components: gas, fluid, and solid. Gas and fluid, commonly air and water, fill the void spaces between finite solid particles to form what is referred to as soil fabric. Fabric can vary greatly between locations and from soil type to type. It takes into account chemical properties, history, current stress state, and many additional pertinent details regarding the subsurface.

Capturing the non-linear, stress-dependent behavior of soil is an ongoing challenge for geotechnical engineers. Often, compromises must be made when selecting a constitutive model for an analysis. A single model capable of capturing all aspects of soil behavior has yet to be developed. However, there are many very good constitutive models which can capture aspects of soil response.

3.4.1 Constitutive Model Components

Constitutive models relate stresses to strains in a soil. The equations through which these values are related define the constitutive model, and span a wide range of parameters and forms. Soil type dictates the response, and thus the relationship, of the stresses and strains. However, the method through which a researcher captures that behavior is flexible. Since capturing every nuance of soil behavior is currently prohibitive, the collective geotechnical engineering field has developed sizable suite of constitutive models. Each of these usually focuses on a specific type of soil and a particular aspect of that soil’s behavior. Some common soil behavior specializations are volumetric strain, lateral displacement, and cyclic mobility.

One of the simplest soil models is the elastic-plastic model. As may be inferred from its name, this model attempts to describe soil behavior with a composite model of plastic and elastic components. For stress states below a key value, the soil is assumed to behave elastically. In this region, as stresses increase, the associated strains remain completely recoverable. If the soil were to return to its original state, no permanent deformations would have been sustained. When the induced stresses exceed a certain value, however, the soil is considered to have “yielded” and unrecoverable strains begin to accumulate. This is the plastic portion of the model.

It is important to make note of the point at which the soil “yields”. This is termed the “yield surface” and represents the transition from elastic to plastic behavior. The definition of the yield surface is a pivotal component of any constitutive model.

Elastic-plastic models require four main items to operate (Wood, 2007). These are:
1) Elastic properties
2) A yield function
3) Plastic potential
4) Hardening rule

3.4.1.1 Elastic properties
These parameters describe the manner in which elastic strains develop in the subsurface. These recoverable strains occur in the “elastic” region of the model. Elastic shear and bulk moduli may be used to describe the relationship between stress state and elastic strains.

3.4.1.2 Yield Function
A yield function is, fundamentally, a way to define the boundary of a soil’s zone of essentially elastic behavior. When the soil’s stress state corresponds to a point on the yield function the soil transitions to primarily plastic behavior. All achievable stress states for a given soil are defined within or on the yield function boundary. In other words, the yield function acts as a bound on the conditions possible in the soil.

A model may be further developed by including post-failure behavior. For example, strain softening or hardening may be included, both of which predict movement of the entire failure surface with strain. Such an inclusion could capture the strength increase experienced by a normally-consolidated material as its loading is increased (i.e., confining stress is increased) and its yield surface grows. Conversely, for soft soils which may exhibit a strength loss after peak, the post-failure strength loss may be captured by a properly developed yield function.

A generic yield function may be expressed as:

**Equation 3-1**

\[ F(\{\sigma\},\{k\}) = 0 \]

where \( \{\sigma\} \) is defined by the stress state and \( \{k\} \) are the state parameters.

For negative values of \( F(\{\sigma\},\{k\}) \), the soil is within the essentially elastic region and governed by that behavior. For \( F(\{\sigma\},\{k\}) \) equal to zero, the soil is on the yield function and yielding is occurring. The function \( F(\{\sigma\},\{k\}) \) can never be positive; by definition anything outside the yield surface is an impossible state.

The yield function may, however, increase or decrease in size depending on stress state. This change in overall size is how constitutive modelers capture strain softening or hardening effects. For hardening, two methods may used. Isotropic hardening defines a yield surface that may change in overall size, but whose focus remains unmoved (Stewart et al. PEER804). Alternately, kinematic hardening defines a yield surface that remains dimensionally constant but whose stress space location may vary.

3.4.1.3 Plastic Potential (Flow Rules)
In addition to a properly defined yield function, flow rules are necessary to yield a robust framework for soil model behavior. A flow rule describes the relationship between plastic strains and stresses; an increment of strain will correspond to an increment of stress change, depending on the relationship defined by the model. Generally, a flow rule is of the form:
Equation 3-2

\[ \Delta \varepsilon_p^i = C \frac{\partial P(\sigma,m)}{\partial \sigma_i} \]

For this equation, the plastic strain \( \varepsilon_p^i \) is related to some plastic potential function \( P(\sigma,m) \) and stress increment \( \delta \sigma_i \). The plastic potential function, reminiscent of the yield function, takes into account the current stress-state and a different set of state parameters (Stewart et al., EERI804_2008).

Engineers are particularly interested in the ultimate strength of a soil. A simple, widely used approach for quantifying this parameter is the Mohr-Coulomb method. It is a simple relationship that relates the shear strength \( \tau \) of a soil to its frictional resistance. This is defined by the following equation (Equation 3-3) where \( \sigma' \) is the vertical effective stress acting perpendicular to the plane of interest, \( \varphi' \) is the friction angle of the soil, and \( c' \) is the relative cohesion or intercept of the failure plane.

Equation 3-3

\[
\tau = c' + \sigma' \tan (\varphi)
\]

The Mohr-Coulomb model provides a simplified framework through which to represent soil behavior. Below the shear strength envelope, the soil has not failed. When the shear stress reaches \( \tau \), failure is assumed to occur.

3.4.1.4 Material Damping

Seismic considerations introduce yet another aspect of soil response that needs to be captured: material damping. Material damping is perhaps one of the most complex aspects of SSSI analysis. In structural analysis, energy is dissipated through countless mechanisms. Strains, cracks and physical distress, sound, frictional resistance, and even thermal energy are just a few of the potential sources of energy loss present during seismic shaking which are lumped into the term, “damping.” Methods have been developed for numerical modeling of these mechanisms in structural applications; these are discussed in more detail in Chapter 4.

Soil is no simpler. Just as in structural systems, damping includes frictional energy dissipation, strain energy losses. However, stress state, groundwater level, depositional and seismic history are several of the myriad additional factors which affect the response of the soil.

As such, damping schemes have been developed to capture the general trends associated with damping in natural systems. Hysteretic damping is a widely used scheme and may be implemented in a constitutive model. A small amount of Rayleigh damping may also be included to prevent high frequency ringing which can occur in numerical analyses.

3.4.2 Constitutive Model Examined

The soil profiles in the suite of experiments from the CGM were composed of dense, dry, granular materials. Two models were identified as potential fits for this material, UBCSAND (Byrne, 2001) and UBCHYST (Naesgaard, 2011).

The UBCSAND model is a constitutive model widely used in industry for liquefaction analyses. It is an effective stress, fully-coupled model (Byrne, et al., 2004). Originally designed for plane-strain analysis of dam and earth structure responses during seismic events, this model was primarily intended for use in:
1) Sand-like or other liquefiable soil layers, when shear-induced volume changes were expected to be significant
2) Layers of interest below the phreatic surface, or those expected to be saturated

This model is based on the Mohr-Coulomb model already existent in FLAC. It differs from that model by taking into account plastic strains that occur at all stages of loading. The Mohr-Coulomb model assumes that all strains below the yield surface are elastic. Instead of artificially requiring all strains to be elastic below the yield loci, UBCSAND allows plastic strains to develop below the strength envelope.

Unloading in UBCSAND is modeled as elastic. Reloading will cause plastic behavior. The plastic shear modulus ($G^p$) obeys a hyperbolic relationship between stress ratio and plastic shear strain (Figure 3-3). Upon reloading, $G^p$ will be stiffer than previously modeled.

![Hyperbolic relationship between the change in shear stress ratio and the change in plastic shear strain for UBCSAND (Byrne et al., 2004)](image)

The UBCHYST model is an adaptation of the Mohr-Coulomb model as well. However, it is a total stress model, designed to be used for analyses in:

1) Clay-like fine grained materials, when excess pore pressures are not expected to develop
2) Granular materials which allow rapid dissipation of pore pressures (i.e., no liquefaction expected)

One significant drawback to this model is the neglect of shear-induced settlements. Post-peak strength loss and rate effects are not included in either the UBCHYST or UCBSAND models. However, for this analysis, these were not key behaviors for capture. Instead, site response, SSSI,
and SFSI analysis were the highest priorities, and the UBCHYST model proved a good fit for that application.

As mentioned, the UBCHYST model is based on the Mohr-Coulomb model implemented in FLAC (Itasca, 2008). It improves upon that simple model by adjusting the maximum shear modulus \( G_{\text{max}} \) of the soil with stress state. To accomplish this, \( G_{\text{max}} \) is modified by a multiplicative factor, which is dependent on several soil state parameters. The equation for the tangent shear modulus, as calculated in UBCHYST, is as follows:

\[
G_t = G_{\text{max}} \left(1 - \frac{\eta}{\eta_{\text{rf}}}ight) R_f^n \text{Mod1Mod2}
\]

In this relationship, the developed stress ratio, \( \eta \), is defined as the shear stress normalized by the current effective vertical stress. The developed stress ratio at failure, \( \eta_{\text{rf}} \), is a function of the peak friction angle, \( \phi_f \), the cohesion (\( c' \)) and effective vertical stress, \( \sigma'_v \) (illustrated in Figure 3-4).

With this combination the model was able to capture the acceleration-time history and pseudo-spectral acceleration response for a wide range of motions, including near-fault, high intensity and low-intensity, ordinary motions. Based on laboratory generated shear modulus reduction curves and site response analyses (discussed in 3.6 Free-Field Overview of Results), the UBCHYST constitutive model was selected for use.

![Graphical representation of the parameters used in UBCHYST](Naesgaard, 2011)

Figure 3-4: Graphical representation of the parameters used in UBCHYST (Naesgaard, 2011)
3.5 Free Field Analysis

Achieving agreement between the centrifuge free-field response and that predicted by FLAC was pivotal to all SSI and SSSI analyses to follow. This goal was accomplished through the following steps. First, to better understand the capabilities of FLAC in the free-field, a seismic site response analysis was conducted and compared with SHAKE2000 for several intensities of the Diamond Heights ground motion, recorded during the Loma Prieta earthquake. After this exploration, a shear wave velocity profile for Tests -1 through -3 was developed and checked through two independent methods. SHAKE2000 was used to estimate an initial shear wave velocity profile for Tests-1 through -3 by matching pseudo-acceleration responses at the surface to those recorded at the centrifuge. This best-fit shear wave velocity profile was compared to independently collected Bender element data from a different centrifuge test with identical soil properties to confirm that it accurately represented the centrifuge specimen. Finally, the shear wave velocity profile estimated for Tests -1 through -3 was used in a FLAC numerical mesh. The input ground motions recorded at the base of the centrifuge model were propagated through the FLAC numerical soil profile for each test. The responses at the surface of FLAC mesh were compared with those recorded at the centrifuge.

3.5.1 SHAKE and FLAC Comparison

To better understand the seismic capabilities of FLAC, Example 1.7.3 in the Itasca manual was replicated (Dynamic Manual, Itasca, 2005). In this example, a 1-D soil profile, composed of 150 ft of horizontally layered clay and sand, was subjected to the 1989 Loma Prieta ground motion recorded at Diamond Heights. The soil profile properties are summarized below in Table 3-3.

<table>
<thead>
<tr>
<th>Soil</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Modulus (MPa)</td>
<td>186</td>
<td>150</td>
<td>168</td>
<td>186</td>
<td>225</td>
<td>327</td>
<td>379</td>
<td>435</td>
<td>495</td>
<td>627</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2000</td>
<td>2000</td>
<td>2000</td>
<td>2000</td>
<td>2000</td>
<td>2082</td>
<td>2082</td>
<td>2082</td>
<td>2082</td>
<td>2082</td>
</tr>
<tr>
<td>Dynamic Properties (set)</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Location (ft)</td>
<td>1-5</td>
<td>5-20</td>
<td>20-30</td>
<td>30-50</td>
<td>50-70</td>
<td>70-90</td>
<td>90-110</td>
<td>110-130</td>
<td>130-140</td>
<td>140-150</td>
</tr>
</tbody>
</table>
The centrifuge soil profile consisted of a uniform, dense sand deposit; therefore, a second profile was created in both SHAKE2000 and FLAC to examine the dynamic behavior for that condition. The soil profile’s geometry remained as before. However, all dynamic properties were assigned as Set (2) (sand) (Table 3-4).


An elastic model with hysteretic damping was used for comparison in FLAC. The model was defined as in the Dynamic Manual, Example 1.7.3 (Itasca 2005) to match the appropriate strain-dependent shear modulus reduction and material damping curves identified for use in SHAKE2000. Soil layers within the FLAC column were defined to directly correspond to those in the SHAKE2000 model.

Itasca presented the pseudo-acceleration response spectrum results from 0.0001 g scaled time history only. It was of interest to the researcher to directly observe the spectra from SHAKE and FLAC at larger, more realistic shaking levels. To accomplish this, results of scaled time histories were examined. Seismic accelerations of 0.001 g, 0.01 g, 0.1 g were propagated through the soil profile, and surface accelerations recorded.

Results from these exercises are included in Appendix A. At the lowest shaking level (0.0001 g), the results presented in the Itasca manual were replicated (Figure 3-5). As shaking increased, output from the two software programs began to diverge. At the highest levels of shaking, FLAC yielded a significantly lower response than that predicted by SHAKE2000 (shown below, Figure 3-6 and Figure 3-7). This was expected, as noted by Itasca (2005). The FLAC hysteretic damping model absorbs more energy during realistic shaking levels than the equivalent linear method implemented in SHAKE2000. At low levels of shaking, the response of the system is essentially linear; therefore, the results from both SHAKE2000 and FLAC should correspond (Figure 3-5). At higher levels of shaking, however, the results should diverge, with SHAKE2000 overpredicting the response relative to FLAC output.
Figure 3-5: Sand and Clay Profile FLAC vs. SHAKE Comparison (scaled to 0.0001g maximum)

Figure 3-6: Sand and Clay Profile, FLAC vs. SHAKE Comparison (scaled to 0.1 g maximum)
3.5.1.1 Shear Wave Velocity Estimation: SHAKE Calibration Overview

A maximum shear modulus estimate was necessary to begin the constitutive model calibration process. However, Test-1, Test-2, and Test-3 did not utilize instrumentation to determine the shear wave velocity of the soil profile. A cone penetration test (CPT) was intended to be completed in-flight during each testing sequence. However, the first attempts were unsuccessful due to equipment malfunction. Several additional attempts were unsuccessful due to the density of the model while spinning; the rod encountered refusal and was bent.

As an alternative, the software program, SHAKE2000, was used to generate an estimate of the subsurface shear wave velocity profile. This process began with creating an estimate of shear wave velocity with depth. Then, acceleration time-histories recorded from the centrifuge were propagated through this profile. The surface output was used to generate pseudo-acceleration response spectra to show the frequency content of the motion. The output from SHAKE2000 was compared with that which was recorded at the centrifuge during the relevant spin. Any discrepancies were noted and the soil profile in SHAKE2000 was adjusted until the match between motions output from the computer program were close to those observed at the centrifuge. A detailed approach is presented below.

Figure 3-7: Sand profile, FLAC vs. SHAKE comparison (scaled to 0.1 g maximum)
3.5.1.1 Shear Wave Velocity Estimation: SHAKE2000 Calibration: Soil Profile

The first step was to create a soil column. The depth of the SHAKE2000 column was taken to directly correspond to the depth of the centrifuge model, in prototype scale. This 29.5 m soil column was then discretized into fourteen 1.8 m layers.

The site period of the centrifuge model was estimated to be 0.6 s, from a calibration spin completed before the NCB testing sequence began. Based on the relationship between shear wave velocity of the soil and site period (Equation 3-5), an average shear wave velocity could be estimated.

**Equation 3-5**

\[ T_s = \frac{4H}{V_s} \]

With a soil height \( H \) corresponding to 29.5 m, and the site period \( T_s \) of 0.6 s, the average shear wave velocity was approximately 197 m/s. This was used to generate a soil profile that increased with depth, following general trend lines associated with generic shear wave velocity profiles.

This initial estimate of shear wave velocity was used as the basis for the soil profile in SHAKE2000. The confining stress at the midpoint of each soil layer was estimated, based on CGM Nevada sand data. Then, Darendeli (2001) strain-dependent shear modulus reduction and damping curves best approximating the stress condition were specified (ranging from 0.25atm at the surface to 4atm at the base). The impedance contrast between the infinite elastic halfspace and the soil was made very large, with a shear wave velocity of 362 m/s for the deepest soil layer and 3100 m/s for the infinite elastic halfspace. This was done to mimic the actual conditions at the centrifuge. The rigid base of the container does, in fact, present an extremely large velocity contrast to the Nevada sand.

Ground motions which had been recorded at the base of the laminar box during testing were used as input acceleration time histories for the SHAKE model. Each was applied as a “within” motion. Since the motions were all recorded at depth in the centrifuge model, the input time histories needed no adjustments to account for upward propagation, as is usually the case for recorded outcropping time histories.
Table 3-5: Calibrated soil properties from SHAKE for CGM profile

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (m)</th>
<th>Damping</th>
<th>$V_s$ (m/s$^2$)</th>
<th>$G_{max}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 1.8</td>
<td>0.05</td>
<td>115</td>
<td>22.6x10$^4$</td>
</tr>
<tr>
<td>2</td>
<td>1.8 - 3.8</td>
<td>0.05</td>
<td>145</td>
<td>35.5x10$^4$</td>
</tr>
<tr>
<td>3</td>
<td>3.8 - 5.9</td>
<td>0.05</td>
<td>172</td>
<td>50.2x10$^3$</td>
</tr>
<tr>
<td>4</td>
<td>5.9 - 8.0</td>
<td>0.05</td>
<td>197</td>
<td>65.9x10$^3$</td>
</tr>
<tr>
<td>5</td>
<td>8.0 - 10.1</td>
<td>0.05</td>
<td>222</td>
<td>83.8x10$^4$</td>
</tr>
<tr>
<td>6</td>
<td>10.1 - 12.2</td>
<td>0.05</td>
<td>244</td>
<td>101.2x10$^4$</td>
</tr>
<tr>
<td>7</td>
<td>12.2 - 14.3</td>
<td>0.05</td>
<td>265</td>
<td>119.3x10$^3$</td>
</tr>
<tr>
<td>8</td>
<td>14.3 - 16.4</td>
<td>0.05</td>
<td>284</td>
<td>136.9x10$^3$</td>
</tr>
<tr>
<td>9</td>
<td>16.4 - 18.5</td>
<td>0.05</td>
<td>301</td>
<td>153.6x10$^3$</td>
</tr>
<tr>
<td>10</td>
<td>18.5 - 20.6</td>
<td>0.05</td>
<td>317</td>
<td>170.0x10$^4$</td>
</tr>
<tr>
<td>11</td>
<td>20.6 - 22.7</td>
<td>0.05</td>
<td>330</td>
<td>185.0x10$^3$</td>
</tr>
<tr>
<td>12</td>
<td>22.7 - 24.8</td>
<td>0.05</td>
<td>343</td>
<td>199.1x10$^3$</td>
</tr>
<tr>
<td>13</td>
<td>24.8 - 26.9</td>
<td>0.05</td>
<td>353</td>
<td>211.7x10$^3$</td>
</tr>
<tr>
<td>14</td>
<td>26.9 - 29.5</td>
<td>0.05</td>
<td>362</td>
<td>221.9x10$^3$</td>
</tr>
<tr>
<td>15</td>
<td>&gt;29.5</td>
<td>0.01</td>
<td>3100</td>
<td>16.29x10$^6$</td>
</tr>
</tbody>
</table>

3.5.1.2 Shear Wave Velocity Estimation: Bender Elements: Overview

For the fourth spin in the centrifuge testing sequence, bender elements were employed. Bender elements are small, piezoceramic elements which are designed to deflect when voltage is transmitted through them. Such deflections are very small. The general concept behind bender elements is that when buried, these elements can cause an infinitesimal deformation in the surrounding soil. The disturbance induced by the element creates a small-strain shear wave in the material.

The bender elements operate as pairs. For use at the centrifuge, two elements are carefully positioned within the subsurface of a specimen some horizontal distance apart; this provides a travel distance for the induced shear wave. Then, one element is deflected while the other is used to record the arrival of the induced shear wave. Researchers identify the first arrivals of shear waves associated with the wave train. The time required to travel the distance
between bender elements is used to estimate the shear wave velocity of the soil profile, and thus the small-strain shear modulus ($G_{\text{max}}$).

3.5.1.2.1 Shear Wave Velocity Estimation: Bender Elements: NCB Spin Details

The testing sequence of the NEES City Block project included four (4) tests founded on identical dense, dry Nevada sand ($D_r \approx 80\%$). Therefore, while the structural configurations for each test differed, the subsurface was expected to remain essentially uniform between all tests. In addition to similar soil profiles, the ground motions used for Test-4 were identical to those used in Test-1, Test-2, and Test-3. Therefore, the bender element data recorded during the fourth spin was used to examine the fit of the SHAKE-created material profile for Test-1, Test-2, and Test-3.

Test-4 spanned three days of testing. Each day had a unique ground motions schedule. Motion intensity increased with time during each spin, as in previous tests. The ground motions for the three days are presented below. Figure 3-8 through Figure 3-10 show the data recorded with the bender elements. The identifier indicates that the data was taken just after applying that ground motion.

The measurements shown in these plots display a progressive densification of the soil profile, and associated increase in shear wave velocity, with time during each spin. However, this effect is relatively small. The researcher was interested in capturing the behavior of the soil and structure system with an eye to repeatability, robustness, and practical application in industry. Therefore, the alteration to the shear wave velocity that could have been applied to the soil profile for different ground motions was considered negligible and was not applied.

![Figure 3-8: Comparison of SHAKE and CGM Bender element (by motion) $V_s$ profiles (day 2, Test-4)](image-url)
Figure 3-9: Comparison of SHAKE and CGM Bender element (by motion) $V_s$ profiles (day 3, Test-4)

Figure 3-10: Comparison of SHAKE and CGM Bender element (by motion) $V_s$ profiles (day 4, Test-4)
3.5.2 Soil Profile Generation in FLAC

The next step in the soil analysis was to create a soil profile to approximate the free-field soil condition. As previously mentioned, the soil profiles for Tests -1 through -3 consisted of a uniform, dense dry Nevada sand profile (D_r~80%).

Mesh creation in FLAC was relatively straightforward for this configuration. The depth of the FLAC model was identical to that of the centrifuge model at prototype scale (i.e., 29.5 m). The width of the FLAC mesh was designed to be wide enough to prevent any reflection of seismic wave energy from the lateral boundaries. It was made 54 m wide (prototype). Maximum grid size is limited in the dynamic configuration to ensure proper seismic wave travel. Based on the work of Kuhlemeyer and Lysmer (1973), Itasca recommends that the element dimension, Δl, be no larger than:

\[
\Delta l \leq \frac{\lambda}{10}
\]

The specified wavelength, λ, is that which is associated with the highest frequency component of the input ground motion which contains significant energy (Itasca, 2005). If the specified wavelength too low, the grid size may be altered or the ground motion must be filtered to ensure accurate transmission of seismic energy for a given system.

The grid was generated according to these parameters. Lateral boundary conditions were imposed which slaved nodes across the mesh. Each grid point on the left boundary (e.g., i=1, j=n) was attached to its corresponding grid point on the right boundary (e.g., i=k, j=n), with elevation. This ensured that the lateral boundaries of the mesh moved in tandem, mimicking conditions experienced in the centrifuge.

Free-field boundaries were also examined. However, no appreciable difference was observed between the slaved and the free-field condition in either the pseudo-acceleration response spectra or acceleration-time histories. The slaved boundary condition was employed in the calculations for this study.

3.5.3 Calibration of Constitutive Model to Laboratory Data

The final step in preparation for the numerical free-field analysis was to calibrate the constitutive models of interest to laboratory data. This allowed the researcher to gain an understanding of the constitutive models’ behavior and capabilities for the dry, dense condition of interest. The insight gained during this process guided the researcher to the most appropriate constitutive model for the soil condition, UBCHYST. This process also allowed the researcher to gain an understanding of the sensitivity of the model to different input parameters and its limiting conditions (softest and stiffest achievable responses).

3.5.4 Shear Modulus Reduction and Damping Curves

Two of the most widely used descriptors of a soil’s dynamic behavior are its shear modulus and material damping values. Many dynamic analyses require engineers to estimate equivalent linear soil properties; as a result, much attention has been given to these parameters and quantifying their correlation to dynamic soil behavior (Kramer, 1996). The maximum shear modulus (G_{max}), which corresponds to very small shear strain amplitude (0.0001%), may be defined with the equation:
where $\rho$ is the representative mass density of the soil, and $V_s$ is the shear wave velocity. While empirical equations exist, material damping values for soil are usually estimated from either laboratory data or published material damping curves.

Both shear modulus and material damping vary with shear strain amplitude; applying a shear strain to soil will yield a softened response. Shear modulus variation is usually presented as a function of shear strain on what is termed a shear modulus reduction curve. Such a figure usually presents the information on a plot with shear strain (in %) as the abscissa and a ratio of shear modulus divided by the maximum shear modulus ($G_{\text{max}}$) as the ordinate. The shape of the shear modulus reduction curve imparts valuable information regarding to the behavior of a soil, while the damping curve provides a complimentary plot of the rate of damping increase with shear strain.

Shear modulus and damping values are also both stress-dependent. A soil’s confining stress has a significant effect on both the shear modulus reduction and the material damping curves, independent of other parameters. One of the most obvious ways in which this pressure dependency manifests itself is through the volumetric shear strain value ($\gamma_{VT}$). The volumetric shear strain of a soil is defined as the shear strain at which soil particle sliding can occur which results in permanent soil deformation. This essentially moves the soil from a “non-degradable” to “degradable” response. The volumetric shear strain is a function of particle friction, contact area, modulus of elasticity and other parameters at the particle level, as well as the confining stress, which influences shear strain as $\gamma_{VT} (\%) \propto \sigma^{2/3}$. In general, as shown by this relationship, an increase in confining stress induces a shift of the volumetric threshold strain to larger values. Damping associated with the subsurface is generally lower at high confining pressures.

The constitutive model selected for use, UBCHYST, does not take into account the variation of shear modulus with confining pressure; a single maximum shear modulus is assigned for the entire thickness of the specified soil layer. Therefore, to take into account the effects of increasing confining pressure with depth, the FLAC profile model was discretized into layers during mesh generation. The collection of laboratory generated, stress-dependent shear modulus reduction and material damping curves developed by Darendeli (2001) were utilized for constitutive model calibration. Curves for 0.25 atm, 1 atm, and 4 atm mean confining stress values were used to describe the stress state at depth (midpoint of each layer) as appropriate. The profiles of interest were all composed of Nevada sand, which exhibits negligible cohesion or plasticity; therefore, the PI = 0 curves were used.

### 3.5.5 Estimating Shear Modulus and Damping Values

In the laboratory, monotonic tests or cyclic simple shear (CSS) tests are used to generate data for estimation of soil shear modulus and material damping values. CSS tests yield hysteretic loops relating shear stress to strain; shear modulus and material damping can be estimated directly from these loops. Monotonic test results show the relationship between applied shear strain and associated shear stress, yielding what is termed a backbone curve. The shear modulus of the soil can be estimated from this curve for any shear strain level of interest, as shown in Figure 3-11.
Figure 3-11: Hysteresis loop from backbone curve for $\gamma_i$

In turn, damping can also be approximated from the backbone curve with the Masing criteria. The energy dissipated during a cycle of loading (represented by the area enclosed by the hysteresis loop), $\Delta E$, divided by the area under the secant modulus, $A$ (represented by the shaded region in Figure 3-11) can describe the equivalent viscous damping ratio, $\lambda$, according to the equation:

$\lambda = \frac{\Delta E}{4\pi A}$

3.5.6 Element Tests in FLAC

An element test modeled after a CSS laboratory test was conducted in FLAC to generate shear modulus and material damping values to generate $G/G_{\text{max}}$ and $\lambda$ curves. A constant x-velocity was applied along the top boundary of the element; the base was fixed in both the x- and y-directions. The total displacement due to the applied x-velocity was limited by shear strain value. The top boundary was allowed to displace laterally until it achieved a shear strain equal to the specified maximum value. Then, the x-velocity was reversed until the magnitude of the absolute value of shear strain was achieved in the opposite direction. This generated the familiar hysteresis loop associated with cyclic simple shear laboratory tests.

Shear modulus and damping curves associated with UBCHYST constitutive models were needed for comparison with the Darendeli (2001) laboratory data. UBCHYST was calibrated with a set of five parameters. For each depth, an initial estimate of values was made based on a sensitivity analysis that had been completed. The values of normalized shear modulus and material damping were plotted versus shear strain for fifteen strain values, ranging from 0.0001% to 1% shear strain. The relative difference between the target laboratory curve for the confining stress at depth and the FLAC output was computed. Model parameters were adjusted accordingly to improve the shape of the curves. This process was continued until the UBCHYST curve was within 5% of the target for all values (Appendix A).
Based on the shear wave velocity profile determined earlier, the soil FLAC profile was divided vertically into 14 layers. The confining stress at the midpoint of each layer was calculated. Parameters for the constitutive model were determined for each layer by calibrating shear modulus reduction and damping curves to the Darendeli (2001) PI=0 curve corresponding to the relative mean confining pressure associated with that depth (see Table 3-6). Damping ratio and shear modulus reduction curves for each layer are presented in the Appendix A.

Table 3-6: Calibrated values for UBCHYST soil layers

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$G_{\text{max}}$ (kPa)</th>
<th>$\Phi$ (°)</th>
<th>$h_{\text{rf}}$</th>
<th>$h_{\text{n}}$</th>
<th>$h_{\text{dfac}}$</th>
<th>$h_{\text{rm}}$</th>
<th>$h_{\text{n1}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 1.8 m</td>
<td>2.26E+04</td>
<td>40</td>
<td>0.98</td>
<td>3</td>
<td>0.8</td>
<td>1</td>
<td>0.9</td>
</tr>
<tr>
<td>1.8 m to 3.8 m</td>
<td>3.55E+04</td>
<td>40</td>
<td>0.98</td>
<td>3</td>
<td>0.8</td>
<td>1</td>
<td>0.9</td>
</tr>
<tr>
<td>3.8 m to 5.9 m</td>
<td>5.01E+04</td>
<td>40</td>
<td>0.98</td>
<td>3</td>
<td>0.8</td>
<td>1</td>
<td>0.89</td>
</tr>
<tr>
<td>5.9 m to 8.0 m</td>
<td>6.59E+04</td>
<td>40</td>
<td>0.98</td>
<td>3</td>
<td>0.8</td>
<td>1.5</td>
<td>0.89</td>
</tr>
<tr>
<td>8.0 m to 10.1 m</td>
<td>8.38E+04</td>
<td>40</td>
<td>0.98</td>
<td>3</td>
<td>0.8</td>
<td>1.5</td>
<td>0.89</td>
</tr>
<tr>
<td>10.1 m to 12.2 m</td>
<td>1.01E+05</td>
<td>40</td>
<td>0.98</td>
<td>2.8</td>
<td>0.8</td>
<td>1</td>
<td>0.89</td>
</tr>
<tr>
<td>12.2 m to 14.3 m</td>
<td>1.19E+05</td>
<td>40</td>
<td>0.98</td>
<td>2.8</td>
<td>0.8</td>
<td>1.2</td>
<td>0.89</td>
</tr>
<tr>
<td>14.3 m to 16.4 m</td>
<td>1.37E+05</td>
<td>40</td>
<td>0.98</td>
<td>2.5</td>
<td>0.8</td>
<td>1.2</td>
<td>0.89</td>
</tr>
<tr>
<td>16.4 m to 18.5 m</td>
<td>1.54E+05</td>
<td>40</td>
<td>0.98</td>
<td>2.4</td>
<td>0.8</td>
<td>1.4</td>
<td>0.87</td>
</tr>
<tr>
<td>18.5 m to 20.6 m</td>
<td>1.70E+05</td>
<td>40</td>
<td>0.98</td>
<td>2.1</td>
<td>0.8</td>
<td>1.2</td>
<td>0.88</td>
</tr>
<tr>
<td>20.6 m to 22.7 m</td>
<td>1.85E+05</td>
<td>40</td>
<td>0.98</td>
<td>2.1</td>
<td>0.8</td>
<td>1.2</td>
<td>0.88</td>
</tr>
<tr>
<td>22.7 m to 24.8 m</td>
<td>1.99E+05</td>
<td>40</td>
<td>0.98</td>
<td>2</td>
<td>0.8</td>
<td>1.3</td>
<td>0.87</td>
</tr>
<tr>
<td>24.8 m to 26.9 m</td>
<td>2.12E+05</td>
<td>40</td>
<td>0.98</td>
<td>2</td>
<td>0.8</td>
<td>1.3</td>
<td>0.87</td>
</tr>
<tr>
<td>26.9 m to 29 m</td>
<td>2.22E+05</td>
<td>40</td>
<td>0.98</td>
<td>2</td>
<td>0.8</td>
<td>1.5</td>
<td>0.86</td>
</tr>
</tbody>
</table>
3.6 Free-Field Overview of Results

An earthquake induces a seismic site response unique to both the seismic event and the location. The ground motion and its seismic profile are simultaneously affected by the site, leading to interdependence between earthquake and location.

Intensity and type of ground motion influence the level of these interactions. For example, an intense ground motion can alter a site’s characteristics. A soil site’s period may be permanently lengthened after seismic shaking due to plastic deformations. With regard to the effects of a site on the ground motion, amplification at the site’s fundamental period, material damping with shear strain level, and topographic effects can all alter a motion’s frequency content as it propagates through a soil profile. It is challenging to capture the range of effects due to these processes with a single computer program and constitutive model.

For low-intensity ordinary motions, the modifications in site and motion will likely be comparatively small. A case for which this might be true in the NEES City Block testing sequence is the Joshua Tree, low intensity (JOS_L1) motion. Its maximum PGA was 0.14 g, and its shaking level was relatively uniform.

In comparison, near-fault high-intensity motions, which are often characterized by short shaking durations and large velocity pulses, frequently result in highly nonlinear soil response. These motions can induce shear strains with magnitudes large enough to move the shear modulus past the volumetric threshold strain into the inelastic, degradable region; an example of such a motion in this project is the Sylmar Converter Station High Intensity (SCS_H) motion. Damping values also become greater, and energy is dissipated.

These interactions are known as the seismic site response. Capturing these phenomena was the first goal. SSI and SSSI analysis can only be as successful as its individual components. Selection of the appropriate constitutive model and an assessment of the model’s ability to capture a wide range of ground motion types and intensities was a key factor in this analysis. Without confidence in the soil profile and its ability to model correctly seismic response, any soil-structure analysis would be groundless.

3.6.1 Summary of Free-Field Results

The optimum response was achieved with fourteen layers, as specified in Table 3-6. The results generally captured the shapes and magnitudes of the acceleration response spectra at the surface, as well as the time histories. Some variation was observed between ground motion types and order in the shaking schedule. Below are examples of the acceleration response spectra, acceleration-time histories, and associated explanation.
3.6.1.1 Ordinary, Low Intensity Motions

First, ordinary, low-intensity ground motions were examined. As expected, the soil profile successfully captured the frequency and time histories for these motions. The soil profile remained in an elastic region, and the seismic response was relatively linear.

The low-intensity Joshua Tree (ordinary) record was used as a baseline motion for the testing sequences. For example, in Spin 1, there were three (3) separate centrifuge flights. Between each flight, the structural models were retrofitted, surface observations were made, and instrumentation was repaired or replaced when possible. During this process some disturbance of the model was unavoidable; therefore, the level and its effect on the model was an important consideration. By beginning each spin with this same JOS_L motion, a baseline representing the changes in the model was created. This allowed direct evidence of the differences in the model.

As may be seen in the following figures, the free-field JOS_L motions 1 through 3 exhibited little difference between spins (Figure 3-12, Figure 3-15, and Figure 3-16). The slight variation observed between each record is negligible; surface PGAs for JOS_L1, JOS_L2, and JOS_L3 were 0.14 g, 0.16 g, and 0.16 g, respectively. The difficulty of exactly replicating any input ground motion at the CGM alone could account for the differences between PGA and pseudo-acceleration spectral responses.

However, it is undeniable that there are differences. While not significant to the physical modeler, this variation was of interest in the numerical realm. These differences allowed examination of the ability of the FLAC model to capture nuances between “identical” motions. With this in mind, examine the JOS_L sequence with FLAC. Of the three motions, JOS_L2 was most closely matched. The response from JOS_L1 was slightly over-predicted, while JOS_L3 was slightly under-predicted at the maximum spectral response.

Two other low-intensity motions, Chi Chi (TCU_L) and Parachute Test Site (PTS) were also examined. As may be seen in Figure 3-13 and Figure 3-14, the recorded responses from both the TCU_L and PTS motions were closely matched by the FLAC.
Northridge, 1994  
(Joshua Tree: JOSL1)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>1</td>
<td>Low</td>
</tr>
</tbody>
</table>

Free-field surface

Base input motion

Figure 3-12: Joshua Tree, low intensity (JOS_L1) motion
Chi Chi, 1999 (TCU078)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>2</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 3-13: Chi Chi, low intensity (TCU_L) motion
Figure 3-14: Parachute Test Site (PTS) motion
Figure 3-15: Joshua Tree, low intensity (JOSL2) motion
<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>13</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 3-16: Joshua Tree, low intensity (JOSL3) motion
3.6.2 Ordinary, High Intensity Motions:

Next, ordinary high-intensity motions were examined. The two motions with this designation were amplified versions of the Chi Chi (TCU_H) and Joshua Tree (JOS_H) recordings. Overall, the propagation of these motions through the subsurface was well captured (Figure 3-17 and Figure 3-18).

The time histories predicted by the FLAC model closely matched those recorded at the centrifuge. The acceleration amplitudes, period, and duration of shaking were in close agreement with those observed during testing. As may be seen in the pseudo-spectral responses, the motions also captured the spectral shape. Some response was not captured in the short period (high frequency) range of approximately 0.2 to 0.3 seconds. This is likely due to container effects and/or physical imperfections in the shaking configuration that the numerical model is not able to capture.
Northridge, 1994
(Joshua Tree: JOSH)

Type of Motion | Order of Motion | Intensity
--- | --- | ---
Ordinary | 11 | High

Figure 3-17: Joshua Tree, high intensity (JOS_H) motion
<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>16</td>
<td>High</td>
</tr>
</tbody>
</table>

Chi Chi, 1999 (TCU078-E)

Figure 3-18: Chi Chi, high intensity (TCU_H) motion
3.6.3 Near-Fault, Low Intensity Motions:

Near-fault, low intensity motions were considered next, including the Sylmar Converter Station (SCS_L), Saratoga West Valley College (WVC_L) and Newhall West Pico C (WPI_L) recordings. The numerical model consistently overpredicted the maximum spectral response for all motions in this category.

Similar to the JOS_L sequences, the SCS_L motion was repeated during Test-1. The two motions’ spectral responses varied slightly; differences were exhibited in the FLAC output for these motions as well, indicating that the numerical model was capturing the real variations in the motions. The results from these two may be seen in Figure 3-19 (SCS_L1) and Figure 3-20 (SCS_L2).

The remaining motions’ pseudo acceleration spectral responses also aligned with those observed during centrifuge testing. The WVC_L recording (Figure 3-21) showed the most significant overestimation of response for any of the near-fault motions; this amplification was focused in the 0.9s to 1s range. In contrast, the WPI_L motion response was almost identical to that observed during the centrifuge test. Therefore, while there was some variation in the individual ground motions, the overall ability of the shear wave velocity profile to capture the responses of the near fault motions in the free-field was quite good.
### Northridge, 1994
(Sylmar Converter Station, SCSL1)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault</td>
<td>5</td>
<td>Low</td>
</tr>
</tbody>
</table>

![Graph showing acceleration vs. time](image1)

**Figure 3-19:** Sylmar Converter Station, low intensity (SCS_L1) motion
**Northridge, 1994**  
(Sylmar Converter Station, SCSL2)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault</td>
<td>8</td>
<td>Low</td>
</tr>
</tbody>
</table>

**Figure 3-20**: Sylmar Converter Station, low intensity (SCS_L2) motion
<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault</td>
<td>9</td>
<td>Low</td>
</tr>
</tbody>
</table>

Loma Prieta, 1989  
(Saratoga West Valley College: WVCL)

Figure 3-21: Saratoga West Valley College, low intensity (WVC_L) motion
Northridge, 1994
(Newhall West Pico: WPIL)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault</td>
<td>12</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 3-22: Newhall West Pico C, low intensity (WPI_L) motion
3.6.4 Near-Fault, High Intensity Motions

Perhaps the most interesting motions are the near-fault, high intensity motions. These include the Lucerne (LCN), Newhall West Pico (WPI_H), Sylmar Converter Station (SCS_H), Port Island Modified (PRImod), Rinaldi Recording Station (RRS), and West Valley College (WVC_H); motions with large velocity pulses strong enough to induce non-linearity in the soil profile.

For the RRS, LCN, WPI_H, and WVC_H motions, the responses were well captured (Figure 3-23, Figure 3-24, Figure 3-26, and Figure 3-28, respectively). The pseudo-acceleration response spectra for the motions followed the responses observed during the centrifuge testing sequence. Additionally, the amplitudes were in close agreement both in the time and frequency domains.

In contrast, the PRImod (Figure 3-27) and SCS_H (Figure 3-25) motions missed some of the observed pseudo-spectral acceleration responses. The amplification of the motion in the 0.3s to 0.5s range was not captured by the FLAC model. However, the responses at longer periods (greater than 0.6s) exhibited close agreement with the observed, physical responses. These periods are usually of greater interest to earthquake engineers, as most sensitive projects are usually in the longer period ranges.

The FLAC model was not expected to precisely match every ground motion. These differences could have been due to physical aspects of the centrifuge system which were not modeled in the numerical model, including the container effects and shaking variations. Therefore, the quality of the matches was considered acceptable for the free-field condition.
Northridge, 1994
(Rinaldi Receiving Station: RRS)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault</td>
<td>3</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 3-23: Rinaldi Recording Station (RRS) motion
Figure 3-24: Lucerne (LCN) motion
Northridge, 1994
(Sylmar Converter Station, SCSH)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>10</td>
<td>High</td>
</tr>
</tbody>
</table>

Figure 3-25: Sylmar Converter Station, high intensity (SCS_H) motion
Northridge, 1994  
(Newhall West Pico: WPIH)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>14</td>
<td>High</td>
</tr>
</tbody>
</table>

Free-field surface

Base input motion

Figure 3-26: Newhall West Pico, high intensity, (WPI_H) motion
### Kobe, 1995
(Port Island (modified): PRImod)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Order of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>15</td>
<td>High</td>
</tr>
</tbody>
</table>

**Figure 3-27:** Port Island, modified (PRImod) motion
Figure 3-28: Saratoga West Valley College, high intensity (WVC_H) motion
3.7 Summary of Free-Field Results

Centrifuge data was collected and pseudo-spectral acceleration response plots computed for the suite of ground motions used in the NCB City Block testing sequence. These motions ranged from low to high intensities, and included both near-fault and ordinary records. Several of the motions were repeated, during the testing sequence, to provide data to document differences in a laboratory specimen over the course of a multi-day test.

This data was compared with results obtained from the plane-strain FLAC software suite (Itasca, 2005). Several constitutive models were examined for use in this software. The centrifuge free-field acceleration time histories and pseudo-spectral acceleration responses were the metrics by which constitutive models were judged. The UBCHYST constitutive model provided the optimal fit to the observed responses in the physical model.

Every ground motion was compared with the output from FLAC. Overall, the responses in both the time and frequency domains were very good. Both the near-fault and ordinary motions at low-intensities were well-captured, with slight amplification observed for most of the near-fault motions.
4 Dynamic Analysis of Centrifuge Test-1 and Test-2

4.1 Introduction to Geotechnical Centrifuge Test-1 and Test-2

The suite of NCB city-block SSI and SSSI geotechnical centrifuge experiments began with Test-1, which was designed to serve as a “baseline” SSI reference case for the following three SSSI experiments. The goals for the Test-1 were:

1) To investigate SSI effects of two model buildings.
2) To generate well-constrained SSI data for use in calibration of numerical models.
3) To use as a basis for comparison with more complex SSSI test geometries to come.

Test-1 (HBM02) consisted of two solitary structures founded on dense (D_r~80%), dry Nevada sand. One structure represented a 3-story (prototype) building (MS1-1), founded on four shallowly embedded spread footings. The other structure was a 9-story (prototype) structure (MS3-1), built on a deep 3-story (prototype) basement. The two buildings were positioned approximately equidistant from each other and the sides of the laminar box, to best represent the desired “solitary” soil-structure-interaction (SSI) condition.

The second test in the city-block suite, Test-2 (HBM03), stemmed directly from the baseline test. It examined the effects a tall structure founded on a deep foundation might induce on an adjacent, low to mid-rise structure constructed on a shallow foundation and vice versa. This was accomplished by placing the 9-story structure and basement (MS3-2) a distance of 0.27 m (prototype) from the 3-story (prototype) frame (MS1-2) in the center of the laminar box. This test was the first examination of structure-soil-structure interaction (SSI) in the city-block sequence. The 3-story and 9-story structures from Test-1 were reused, with slight modifications detailed in the following Test-2 section (Section 4.2). Nevada sand at D_r~80% again served as the soil profile.

Test-1 and Test-2 were then examined in FLAC. Structural and soil responses from FLAC were matched to those observed in the centrifuge through soil calibration (as described in Chapter 3) and structural calibration (described in Section 4.5.2). The relative accuracy of prediction by FLAC was quantified though:

1) Comparison of time histories recorded during centrifuge testing and from FLAC at locations of interest.
2) Comparison of several engineering design parameters between the data recorded during the centrifuge test and FLAC results. These parameters are discussed in more detail in the pages to follow (Section 0).

4.2 Test-1 and Test-2: Structural Selection Background

The structures used in the Test-1 and Test-2 sequence were special moment-resisting frames (SMRF) designed to capture the response of typical medium- to low-rise structures. The city-block used in this study was developed to be representative one located in the Los Angeles, California area; therefore, buildings likely to be found in that metropolis were targeted. The structural engineering researchers on the project worked closely with several members of industry (Trombetta et al., 2012) to select representative structures for that region.
Simplification of the prototypical structures was necessary to create models for use in the centrifuge. Three parameters were identified as pivotal to capture:

1) a flexible base period, $T_{n,SSI}$,
2) a yield strength ratio, $V_y/W$ where $V_y$ is the yield strength for base shear and $W$ is the weight of the superstructure, and
3) a yield drift ratio, $\partial_y = \Delta_y/H$, with $\Delta_y$ describing the structure’s displacement and $H$ representing the structure’s height, measured from the soil elevation.

Structural engineers used these targets and the finite element analysis computational software, OpenSees, to create structural models which satisfied the design goals.

Hollow A513 steel tubes were used for the beam and column components of both frames. Potential inelastic response of the structures was identified during the numerical modeling design phase of the physical model development. To force localization of this response at a desired location, the researchers included several variations of fuses (areas of reduced cross-section) in all beams and columns. The fuses ensured that if yielding were to occur during a ground motion, the inelastic deformation would be isolated at these intentionally weakened locations. Three types of fuses were designed, based on the desired strength of the system. These are presented in Figure 4-1.

Another significant advantage of the centrifuge testing device is repeatability. If the soil properties permit (e.g., the soil is neither liquefiable nor extremely loose), there is the opportunity to run multiple ground motions over the course of several days of testing for a single laboratory specimen. This capability was capitalized upon during the city block testing sequence in two ways.

First, as mentioned, a suite of ground motions was developed and applied to the model over the course of several days for each test. This allowed ordinary and near-fault motions, at high and low intensities, to be applied to the same system. The myriad motions provided insight into the response of a particular structural and soil system to a wide variety of seismic excitations.

Second, the structures themselves were specifically designed to take advantage of this capability. The fundamental period of a structure and the type of fuse used in construction dominated each frame’s response during testing. A different flexible-base period could significantly change the dynamic response of the system, in both amplitude and frequency content. The fuses dictated at what point the structure began to yield, introducing nonlinearity, additional damping, and irrecoverable deformation – all of which influenced the response of the structure.

The opportunity to test several frames during a spin, rather than being limited to a single configuration, was recognized during the design phase of the experiment. Therefore, beams, columns, and the centered masses for each structure were designed with a modular approach. All components had bolted connections and were thus able to be changed during testing. This allowed retrofitting of the structure if excessive nonlinearity was induced during a spin, as well
as the ability to exchange masses or (through use of various beam fuses) modify the stiffness to alter the fundamental period of a structure during a testing sequence.

In Test-1, three configurations were used for the MS1 frame; Test-2 contained two configurations. The MS3 superstructure was uniform throughout Test-1 and Test-2; identical masses and beams were used for construction. The deep basement was altered slightly between the experiments, but this change was minor. The only effect observed from this change was a slight change in the period-lengthening ratio \( \frac{T_{SSI}}{T_{fixed}} \) between ground motions from 1.05 in Test-1 to 1.0 in Test-2.)

Figure 4-1: Summary of configurations for Test-1 and Test-2, as well as the fuse configurations (Trombetta et al., 2012). Small structure (left) approximates a three-story (prototype) SMRF (MS1-1 and MS1-2), large structure (right) approximates a nine-story (prototype) SMRF (MS3-1 and MS3-2)
4.3 FLAC Structural Background

The software suite selected for numerical analysis of the tests, FLAC, imposes a plane strain condition on all analyses. A plane strain condition assumes that the dimension in the z-direction (defined as into the page) is infinitely long. With such an assumption, strains in the z-direction can be assumed to be negligible. This plane strain assumption is reasonable and realistic for soil profiles, excavations, and other applications in which the length of the system is very large relative to the width. However, this condition imposes some limitations on applications in which that ratio is not large.

For example, the three dimensional renderings of the structures used during the Test-1 and Test-2 centrifuge experiments are presented below (Figure 4-2). As is immediately clear, each frame was founded on a square footprint, and unquestionably three-dimensional. Therefore, accurately approximating the three dimensional dynamic response with the two dimensional tool, FLAC, required creation of a 2-D equivalent to the 3-D system. To capture the effects of soil-structure interaction, key aspects of the frame structure were identified and matched within the FLAC model.

![Figure 4-2: Structural configurations for Test-1 (left) and Test-2 (right). Note that the soil profile is not included in this rendering, to allow observation of the foundations (Mason, 2011).](image-url)
4.3.1 Beam Elements in FLAC

FLAC provides several options for structural element modeling. For the moment-resisting frames in this testing sequence, beam elements were selected. These components are two-dimensional, plane strain elements. They have three rotational degrees of freedom at the end node of each element – one in the x-direction (horizontal), y-direction (vertical), and one rotational (θ). All beam elements are modeled as elastic unless a yielding moment is specified by the user. If a yielding moment is input, the elements behave as an elastic-perfectly plastic material. Figure 4-3 below shows the degrees of freedom \( (u_1, u_2, \theta) \) for the beam, at both end nodes (a, b).

![Beam element in FLAC (Itasca, 2008)](image)

Beam elements require several physical parameters for generation. An elastic modulus, E, must be specified. A cross-sectional area (A) and moment of inertia (I) are used to specify the dimensions of the elements. A material density (ρ) is not required, but is necessary to compute gravity or dynamic loads. Lastly, plastic moments (default is infinite), axial peak yield strength (default is infinite), axial residual strength (default is zero), and axial compressive yield strength (default is infinite) may be specified, as appropriate.

4.4 Structural Numerical Model Priorities

All aspects of three-dimensional response of the SMRFs in this testing sequence could not be captured simultaneously by a FLAC model. Therefore, careful attention was given to selection of key parameters of interest for the numerical analyses. The following structural properties were identified as most important for this SSI and SSSI analysis:

1) Mass distribution  
2) Stiffness of the superstructure  
3) Fixed-base fundamental period of vibration
Consideration was given to the two additional parameters used by the structural engineering team members during design of the physical centrifuge models, the base shear normalized by superstructure weight \( (V_y/W) \) and the drift ratio. However, these parameters are primarily indicative of the expected level and type of nonlinear response of a structural system. Capturing the details of structural nonlinearity was not a principal goal for the geotechnical modeling; therefore, these parameters were not considered during generation of the plane strain equivalent FLAC models.

4.4.1 Mass and distribution

The dynamic response of any system is influenced by myriad factors, one of which is the mass distribution. This physical aspect of the system influences inertial response, including rocking and pounding, as well as the fundamental period of vibration and the associated amplification of the excitation. The total mass of the physical structure was important to capture, but the relative masses of beams, columns, and footings were equally pivotal to ensure accurate representation of dynamic response.

As such, the distributed mass of each component of the structure was estimated and matched in FLAC. The beams, columns, footings, and centered masses of the centrifuge structures were all distributed over the physical (centrifuge) model’s z-dimension (into the page) to determine a mass per unit length. This was used to find a plane-strain equivalent material density and component dimensions (I, A).

4.4.2 Fundamental Period

Mass is an important system parameter, but it is not the only component which influences a system’s response. Both mass and stiffness of a system combine to describe its fundamental period. The natural (or fundamental) period of vibration \( (T_n) \) is a pivotal descriptor of the dynamic response of a structure. The angular natural frequency \( (\omega_n) \) of a structure can be found with the stiffness of the system \( (k) \) and moving mass \( (m) \) of the system and Equation 4-1.

\[
\omega_n = \frac{k}{\sqrt{m}}
\]

Equation 4-1

\[
T_n = \frac{2\pi}{\omega_n}
\]

This parameter \( (T_n) \) describes “the time required for the undamped system to complete one cycle of free vibration” (Chopra, 2007). As such, it describes the resonance of the system. This is the frequency at which the largest response may be expected due to cyclic loading. Therefore, lateral stiffness (parallel to the axis of shaking) was a parameter of interest as well.

4.5 Model Construction Procedure

FLAC’s plane strain assumption, while reasonable and natural for most geotechnical applications, required that the user implement several approximations when modeling the physical, finite, three-dimensional frame structure. The conversion from three-dimensional reality to the two-dimensional plane strain condition required several steps.

The structural model construction was accomplished through the following procedure. An initial estimate was made of the density and geometry of the structural components based on the
physical models. Then, a fixed-base fundamental period ($T_n$) of vibration was estimated with FLAC. This process was repeated until the $T_n$ period estimate from FLAC matched that provided by the structural engineering researchers, estimated from OpenSees (Trombetta, pers. com) to the one-hundredths decimal place.

4.5.1 Fixed-Base Iterations

A set of equations were written to relate the physical centrifuge model properties to plane-strain estimates. All of the equations were manipulated until they were dependent on a single parameter – material density. This parameter was then adjusted to tune the structure. Columns and beams were assumed to have uniform densities, cross-sectional areas, and moments of inertia, as in the physical model (with the exception of element fuses).

Therefore, to begin the numerical model tuning process, an estimate of material density for the SMRF columns and beams was made. Then, the plane-strain geometry needed to match the distributed mass per unit length from the physical model was estimated. Both the total width and height of the numerical model SMRF were identical to the dimensions of the physical SMRF; only the thickness of the structural members was varied. This established the parameters:

1) the cross-sectional area ($A$) of both the columns and beams
2) the moment of inertia ($I$) of both the columns and beams
3) the material density ($\rho_{col}$) of both the columns and beams

Determination of the spanning beam’s total material density required one additional step – treatment of the structure’s floor mass. To accomplish this, the centrifuge SMRF’s floor mass was distributed over the length of the structure ($z$-dimension, into the page), perpendicular to the plane of shaking. As for the beams and columns, this yielded a total mass-per-unit-length equivalent for the centered mass. The plane-strain dimensions of the beam ($I, A$) determined earlier were held constant. An equivalent material density based on those dimensions was calculated by matching the value of the distributed centered mass (mass/length). This equivalent material density was then lumped into the previously determined FLAC beam’s material density.

Next, masses for the footings were determined. This was accomplished by combining the masses of four footings and distributing them over the length (into the page dimension) of the 3D (physical) structure. The height and width of the footings in the FLAC model were equal to those of the spread footings in the centrifuge. This meant that the material density was adjusted until the strip footing (2-D) mass was equivalent to the spread footing (3-D) mass. A copy of the MathCAD processing sheets for these iterations is included in Appendix A.

Lastly, fuses were included as appropriate. Physical yielding was observed in the MS1 structure’s beams during testing, so it was important to include these in the FLAC model. Beams in FLAC are represented by an elastic, perfectly-plastic constitutive model. If no yielding moment value is specified then the structure is assumed to be infinitely strong. As for the other structural material properties, yielding strengths were distributed over the length ($z$-dimension, into the page) of the structure for use in FLAC.

4.5.2 FLAC $T_{\text{fixed}}$ Estimation

Once a set of equivalent structural properties had been calculated, the beam element values were input to FLAC to create a plane-strain representation of the three-dimensional frame
structure. The final step was to verify that the structure created through the aforementioned method accurately captured the period of interest and contact stresses.

To confirm that the fundamental period was matched, a transfer function of the structural model was used. Fundamentally, a transfer function relates an applied excitation to the dynamic response of a system (Chopra, 2007). Consider the equation of motion for an undamped system:

Equation 4-3  \[ m \ddot{u} + ku = p(t) \]

For free vibration, initial conditions specifying some initial velocity and displacement are needed, \( \dot{u}(0) \) and \( u(0) \), respectively.

The response of the system to a unit impulse may be used to initiate free vibration in such a system. The Dirac delta function, \( \delta(t - \tau) \), describes such a force; it consists of a large impulse at \( t = \tau \) and zero at all other times, \( t \). Considering Newton’s second law of motion:

Equation 4-4  \[ p = m \ddot{u} \text{, which, when integrated, yields } \int_{t_1}^{t_2} p \, dt = m(\dot{u}(t_2) - \dot{u}(t_1)) \]

In other words, the change in momentum of the system (the product of mass, \( m \), and velocity, \( \Delta \dot{u} \)), is equal to the magnitude of the impulse (integrand). Therefore, applying a unit impulse at \( t = \tau \) yields a velocity of \( \dot{u}(\tau) = 1/m \) and a displacement of \( u(\tau) = 0 \).

Taking the solution of Equation 4-3 and substituting the appropriate equations associated with a unit impulse yields:

Equation 4-5  \[ (t - \tau) = \frac{1}{m\omega_n} \sin(\omega_n(t - \tau)) \quad t \geq \tau \]

Therefore, an initial unit velocity was applied to the footings of the fixed-base FLAC frame model. This initial condition was consistent with loading due to an impulse motion centered around time \( t = 0 \) (\( \dot{u}(0) = 1 \)). Then, the initial condition was removed and the system was allowed to oscillate for approximately 20 cycles of free vibration. The frame’s response to the excitation was recorded at the center of the top span, the footings, and at the frame’s connections. To find the period of vibration of MS1, the resulting acceleration time history at the top center of the FLAC structural model was recorded. The fast Fourier transform (FFT) of this output was then calculated for this nodal history.

A Fourier transform is a mathematical transformation that takes data from the time domain and converts it to a frequency domain representation. This operation is possible because periodic signals may be represented as a summation of sine and cosine waves, with the waveforms allowed to have varying amplitudes and periods. There are four types of transforms that a researcher may utilize (Smith, 1997):

1) **Fourier Transform**, for aperiodic-continuous signals: appropriate when the data is an infinite, non-repeating, continuous signal (e.g., decaying signal, pulse-type input)
2) **Fourier Series**, for periodic-continuous signals: appropriate when the data is an infinite, repeating, continuous sequence (e.g., waveforms)
3) *Discrete Time Fourier Transform*, for aperiodic-discrete signals: for use when the data is sampled at discrete points, and is an infinite, non-repeating sequence (identical to aperiodic-continuous, but with individual data points).

4) *Discrete Fourier Transform*, for periodic discrete signals: for use when the signal is sampled at discrete points, and is an infinite, repeating sequence (again, identical to periodic-continuous, but composed of individual data points).

For seismic applications, the discrete Fourier transform (DFT) is usually used. This is due to two main reasons. First, the accelerometers used to record ground motions yield discrete values of acceleration with time. Second, theory requires that an infinite number of waveforms be used to fully describe aperiodic signals. In reality, that task is impossible from a computational (or computer-based) platform.

However, any signal can be represented as periodic by repeating a portion of data. By defining a length of data, the researcher can effectively create a repeating series of recorded data. Use of a DFT allows the researcher to utilize much more efficient algorithms to calculate the transform, one of the most common of which is the fast Fourier transform (FFT). The FFT used for these analyses was generated in MATLAB (MATLAB, 2009a).

Loading a structure at its base with a unit (1 m/s) velocity, removing that initial condition, and then allowing the system to freely vibrate caused the structure to display excitation at its fundamental mode of vibration, no response at some frequencies, and a minimal response at the remaining frequencies. With some manipulation in the frequency domain, the output acceleration time history was processed to show the frequencies at which the structure had a significant response. The natural frequency of the structure was directly identified from the plot (Figure 4-4).

This figure clearly illustrated the points at which frequencies were amplified, corresponding to the fixed-base natural period of vibration of the structure. Additionally, the fixed-base fundamental period of the structure was estimated by observing the number of zero crossings for approximately 20 cycles of free vibration (e.g., cycles/time). This free vibration of the undamped numerical model corroborated the frame’s fixed-base fundamental frequency estimate garnered from the frequency content.

Some iteration was required to tune the structure to the desired natural frequency; this simply involved repeating the procedure described above with an updated estimate of the initial beam and column material density. Plots for each of the tuned structures are presented in Appendix A; an example of one collection of plots is shown below for Test-1, Configuration 1 (Figure 4-4 and Figure 4-5).
Figure 4-4: FFT of acceleration time history from application of a unit velocity at time $t = 0$ and approximately 20 cycles of free vibration ($T = 1/f$, $T_{\text{FLAC}}$, fixed $= 0.91$ seconds, $T_{\text{CGM}}$, fixed $= 0.90$ seconds).

Figure 4-5: Free vibration from application of initial x-velocity, $\dot{u}(0) = 1$ ($T_{\text{FLAC}}$, fixed $= 0.907$ seconds, $T_{\text{CGM}}$, fixed $= 0.90$ seconds)
4.5.3 FLAC Model Dimensions

The cross-sectional area and fourth moment of inertia associated with the density that achieved a fundamental period within one one-hundredth of the target $T_{\text{fixed}}$ were used as the FLAC structural properties. By optimizing the thickness of the beam and column, the stiffness/unit length of the real frame was matched with the equivalent plane-strain value. Table 4-1 through Table 4-3 summarize the actual and plane strain values associated with the three configurations of the MS1 structure for Test-1 [i.e., $MS1-I(l)$, $MS1-I(2)$, $MS1-I(3)$].

Table 4-1: Summary of structural properties for MS1 (Test-1) configuration 1 [$MS1-I(l)$].

<table>
<thead>
<tr>
<th>Configuration 1</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
<td>Units</td>
<td>Value (Plane Strain)</td>
<td>Value (3-D, Prototype)</td>
</tr>
<tr>
<td>$I_b$</td>
<td>m$^4$</td>
<td>0.0114</td>
<td>0.014</td>
</tr>
<tr>
<td>$I_c$</td>
<td>m$^4$</td>
<td>0.0114</td>
<td>0.014</td>
</tr>
<tr>
<td>$A_b$</td>
<td>m$^2$</td>
<td>0.5152</td>
<td>0.2135</td>
</tr>
<tr>
<td>$A_c$</td>
<td>m$^2$</td>
<td>0.5152</td>
<td>0.2135</td>
</tr>
<tr>
<td>$E_{\text{footing}}$</td>
<td>Pa</td>
<td>193.2 x 10$^9$ (plane strain condition)</td>
<td></td>
</tr>
<tr>
<td>$E_{b,c}$</td>
<td>Pa</td>
<td>35.1 x 10$^7$ (plane strain condition)</td>
<td></td>
</tr>
<tr>
<td>$\rho_{\text{steel}}$</td>
<td>kg/m$^3$</td>
<td>--</td>
<td>7,850</td>
</tr>
<tr>
<td>$\rho_{\text{column}}$</td>
<td>kg/m$^3$</td>
<td>1,072</td>
<td>7,850</td>
</tr>
<tr>
<td>$\rho_{\text{beam and mass}}$</td>
<td>kg/m$^3$</td>
<td>13,108</td>
<td>--</td>
</tr>
<tr>
<td>$\rho_{\text{footings}}$</td>
<td>kg/m$^3$</td>
<td>6,542</td>
<td>7,850</td>
</tr>
</tbody>
</table>
Table 4-2: Summary of structural properties for MS1 (Test-1) configuration 2 [MS1-I(2)].

<table>
<thead>
<tr>
<th></th>
<th>Configuration 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>I_b</td>
<td>m⁴</td>
</tr>
<tr>
<td></td>
<td>0.0128</td>
</tr>
<tr>
<td></td>
<td>0.014</td>
</tr>
<tr>
<td>I_c</td>
<td>m⁴</td>
</tr>
<tr>
<td></td>
<td>0.0128</td>
</tr>
<tr>
<td></td>
<td>0.014</td>
</tr>
<tr>
<td>A_b</td>
<td>m²</td>
</tr>
<tr>
<td></td>
<td>0.5356</td>
</tr>
<tr>
<td></td>
<td>0.2135</td>
</tr>
<tr>
<td>A_c</td>
<td>m²</td>
</tr>
<tr>
<td></td>
<td>0.5356</td>
</tr>
<tr>
<td></td>
<td>0.2135</td>
</tr>
<tr>
<td>E_{footing}</td>
<td>Pa</td>
</tr>
<tr>
<td></td>
<td>193.2 x 10⁹ (plane strain condition)</td>
</tr>
<tr>
<td>E_{b,c}</td>
<td>Pa</td>
</tr>
<tr>
<td></td>
<td>35.1 x 10⁹ (plane strain condition)</td>
</tr>
<tr>
<td>ρ_{steel}</td>
<td>kg/m³</td>
</tr>
<tr>
<td></td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>7,850</td>
</tr>
<tr>
<td>ρ_{column}</td>
<td>kg/m³</td>
</tr>
<tr>
<td></td>
<td>1,745</td>
</tr>
<tr>
<td></td>
<td>7,850</td>
</tr>
<tr>
<td>ρ_{beam and mass}</td>
<td>kg/m³</td>
</tr>
<tr>
<td></td>
<td>10,781</td>
</tr>
<tr>
<td></td>
<td>--</td>
</tr>
<tr>
<td>ρ_{footings}</td>
<td>kg/m³</td>
</tr>
<tr>
<td></td>
<td>6,542</td>
</tr>
<tr>
<td></td>
<td>7,850</td>
</tr>
</tbody>
</table>
Table 4-3: Summary of structural properties for MS1 (Test-1) configuration 3 [MS1-I(3)].

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>m³</th>
<th>0.01420</th>
<th>0.014</th>
</tr>
</thead>
<tbody>
<tr>
<td>I_b</td>
<td></td>
<td>m³</td>
<td>0.01420</td>
<td>0.014</td>
</tr>
<tr>
<td>I_c</td>
<td></td>
<td>m³</td>
<td>0.01420</td>
<td>0.014</td>
</tr>
<tr>
<td>A_b</td>
<td></td>
<td>m²</td>
<td>0.5543</td>
<td>0.2135</td>
</tr>
<tr>
<td>A_c</td>
<td></td>
<td>m²</td>
<td>0.5543</td>
<td>0.2135</td>
</tr>
<tr>
<td>E_footing</td>
<td>Pa</td>
<td></td>
<td>193.2 x 10⁹ (plane strain condition)</td>
<td></td>
</tr>
<tr>
<td>E_b,c</td>
<td>Pa</td>
<td></td>
<td>35.1 x 10⁹ (plane strain condition)</td>
<td></td>
</tr>
<tr>
<td>ρ_steel</td>
<td>kg/m³</td>
<td>--</td>
<td>7,850</td>
<td></td>
</tr>
<tr>
<td>ρ_column</td>
<td>kg/m³</td>
<td>1,410</td>
<td>7,850</td>
<td></td>
</tr>
<tr>
<td>ρ_beam and mass</td>
<td>kg/m³</td>
<td>10,140</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>ρ_footings</td>
<td>kg/m³</td>
<td>6,542</td>
<td>7,850</td>
<td></td>
</tr>
</tbody>
</table>

The numerical generation and tuning process for MS3-1 was almost identical to that of MS1-1. However, instead of only a single degree of freedom (DOF), three DOFs were necessary to capture the vibrational characteristics of the MS3-1 structure. Again, the floor masses located at each story were assumed to be lumped into the appropriate spanning beam. The structural properties for the numerical model are shown below in Table 4-4.
### Table 4-4: Summary of structural properties for MS3 (Test-1) (MS3-1).

<table>
<thead>
<tr>
<th>Configuration 1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parameter</strong></td>
</tr>
<tr>
<td>$I_b$</td>
</tr>
<tr>
<td>$I_c$</td>
</tr>
<tr>
<td>$I_{basement}$</td>
</tr>
<tr>
<td>$A_b$</td>
</tr>
<tr>
<td>$A_c$</td>
</tr>
<tr>
<td>$A_{basement}$</td>
</tr>
<tr>
<td>$E_{basement}$</td>
</tr>
<tr>
<td>$E_{b,c}$</td>
</tr>
<tr>
<td>$\rho_{steel}$</td>
</tr>
<tr>
<td>$\rho_{column}$</td>
</tr>
<tr>
<td>$\rho_{beam and mass, floors 1 and 2}$</td>
</tr>
<tr>
<td>$\rho_{beam and mass, floor 3}$</td>
</tr>
<tr>
<td>$\rho_{basement}$</td>
</tr>
</tbody>
</table>

#### 4.5.4 Structural Model Damping

Rayleigh damping was applied to the numerical mesh for all computations to account for energy dissipation at very small shear strain levels and to prevent low-level oscillation (or numerical instability) (Itasca, 2008). Rayleigh damping within FLAC is expressed in matrix form, as per tradition:

**Equation 4-6**

$$ C = \alpha M + \beta K $$

For the above equation, $C$ is the damping matrix, $M$ is the mass matrix, $K$ is the stiffness matrix, $\beta$ is the stiffness-proportional constant, and $\alpha$ is the mass-proportional constant. The critical damping ratio, $\xi$, may be described at every frequency for a multi-degree-of-freedom system with the following equation (Bathe and Wilson, 1976):
This describes the critical damping ratio (for mode $i$), for every angular frequency $\omega_i$. FLAC allows the user to input the values of minimum damping ratio ($\xi_{min}$) and center frequency ($f_{min}$), which together define $\alpha$ and $\beta$:

**Equation 4-8**
\[
\xi_{min} = \sqrt{\alpha \beta}
\]

**Equation 4-9**
\[
f_{min} = \frac{\omega_{min}}{2\pi} = \sqrt{\frac{\alpha}{\beta}}
\]

Shown in Figure 4-6 is an examination of the effects of center frequency and minimum damping value on Rayleigh damping within FLAC. Additional plots to present the relationships between center frequency and damping ratio are included in Appendix A.

![Rayleigh Damping: Effects of Center Frequency](image)

**Figure 4-6**: This plot shows the effect of changing the center frequency value ($f_{min}$) in the FLAC formulation. All lines are for the same minimum damping ratio ($\xi_{min} = 0.2\%$).

For these analyses, the applied Rayleigh damping was centered at the mean period of each ground motion for the mesh (computed from SeismoSignal (SeismoSoft, 2011) and SHAKE2000; a table of these values is presented in Appendix A).

In FLAC, specified Rayleigh damping is applied strictly to the grid and any structural elements that are directly attached to the mesh. Structural damping, if desired, must be applied to those elements separately. For MS1-1, a sensitivity analysis of damping was conducted for ground motions from Test-1. All ground motions were run with no structural Rayleigh damping
and with 3% structural Rayleigh damping. For MS3-1, Test-1 ground motions were examined for two scenarios as well – no structural Rayleigh damping (0%) and a small amount of Rayleigh damping (0.5%). It was expected that the MS3-1 structure would require significantly less applied damping than MS1-1 since MS3-1 exhibited no visible physical yielding or distress during the testing sequence. Also, the soil-structure damping ratio ($\beta_{SSI}$) identified for the first mode by the structural engineering team varied between the structures, providing insight into the level of system damping and nonlinearity (Chen, et.al 2012). The percent differences between the $\beta_{SSI}$ estimated for MS1-1 and for MS3-1 ranged from 200% to 40% (with an average percent difference of 80%) for every ground motion. The MS3-1 frame damping ratio was less than that of MS1-1 for every motion in the testing sequence.

When possible, structural Rayleigh damping was not used in the analyses. It is extremely computationally expensive to implement mass- and stiffness-proportional Rayleigh damping in the FLAC environment. When damping which contains a stiffness-proportional component is applied, there is a significant decrease in the dynamic timestep. The timestep reduction is explained by Equation 4-10 (Belytschko, 1983).

Equation 4-10

$$\Delta t_{\beta} = \left[\frac{2}{\omega_{max}}\right] \left[\sqrt{1 + \lambda^2} - \lambda\right]$$

"where $\omega_{max}$ is the highest eigenfrequency of the system and $\lambda$ is the fraction of critical damping at this frequency" (Itasca, 2008). FLAC does not solve eigenvalue problems, therefore these parameters $\omega_{max}$ and $\lambda$ are approximated in the software as functions of the dynamic timestep (without damping) and the damping and angular frequency selected for the damping application. This decrease can easily necessitate a reduction in timestep of more than an order of magnitude for Rayleigh damping values greater than 1%.

4.6 Numerical Analyses for Test-1 and Test-2

4.6.1 Test-1(HBM02)

The final step in the numerical mesh generation was to merge the plane-strain equivalent structure, which had captured the natural period and mass distribution associated with the prototypical SMRF model, with the free-field soil profile. Since the models were sufficiently distant during Test-1 to minimize interactions and maintain a “solitary” condition, MS3-1 was analyzed separately from MS1-1. The FLAC free-field soil properties were as described in the Calibration section (Chapter 3). The mesh was 29.5 m deep, identical to the centrifuge model dimensions (in prototype). The width extended 60.4 m, to match the width of the centrifuge box (in prototype).

Two dynamic boundary conditions are available in the FLAC environment. One option is “quiet” (or absorbing) boundaries, which absorb outward propagating seismic energy through unlinked dashpots oriented in the normal and shear directions. The other is the “free-field” boundary condition. This condition is available within the numerical environment to approximate the energy absorption of soil, in-situ, as well. Most subsurface geometries have soil profiles which extend infinitely in the lateral direction (x-dimension). This allows seismic energy to be absorbed both through material damping (particular to the subsurface), and through seismologic effects, including wave scattering, dispersion, attenuation, and geometrical spreading. The “free-field” boundary condition enforces absorption of outwardly propagating waves. This non-
reflection of seismic waves is accomplished by attaching a free-field soil column to the boundaries of the main grid through a series of viscous dashpots (Itasca, 2008). Any unbalanced forces present in the free-field column boundary are transmitted to the main grid. In this manner, any perturbations due to a structure in the main grid are absorbed by the dashpots. Upwardly propagating vertical plane waves are unmodified, as the free-field columns move in tandem with the main grid.

However, the condition in the centrifuge is not “free-field” nor is it a “quiet” environment. Instead, the laminar box introduces a significant impedance contrast to the soil, both at the base and along the lateral boundaries. A key aspect of this contrast is the reflection and refraction of seismic waves back into the model; this is a very different phenomenon than that which is modeled in either of the dynamic boundary conditions in FLAC. Therefore, a boundary condition better able to approximate the refraction and reflection effects was desired. A more representative condition was achieved by attaching gridpoints on the left boundary to the corresponding gridpoints on the right boundary of the mesh. This condition meant that the lateral boundaries of the numerical model would move in tandem (i.e., the gridpoints were slaved), mimicking the displacement of the centrifuge’s laminar box.

Time histories were recorded at key points within the numerical soil profile and along the footings, as well as at the center of mass of each SMRF floor. Each of these positions corresponded to an accelerometer location in the centrifuge model, allowing direct comparison of acceleration time histories and pseudo-spectral acceleration responses between the physical and numerical models.

4.6.2 Test-2 (HBM03)

The soil profile in Test-2 targeted the same relative density as Test-1. Dense, ~80% relative density Nevada sand was dry pluviated into the same laminar box used in Test-1. Therefore, the same soil properties used in the Test-1 were applied to Test-2. Again, the mesh was 29.5 m deep and 60.4 m wide, corresponding to the centrifuge box, in prototype.

Unlike Test-1, the structures in Test-2 were adjacent and thus were modeled simultaneously in FLAC. The more complex mesh geometry required for the Test-2 analysis resulted in a reduced timestep from that used for Test-1 analyses. This decrease occurred prior to any applied structural damping. Therefore, as the run-time of each analysis was already increased for Test-2, the sensitivity of the numerical model to structural Rayleigh damping was of great interest. The mesh was examined for two structural Rayleigh damping values: none (0%) and a small amount (0.5%). The MS1-2 structure exhibited negligible change between the two values, as expected. The overall response of the MS3-2 structure, however, did show a lowered response, which improved the overall fit. The majority of the ground motions were run without applied structural Rayleigh damping due to the considerable computational requirements of structurally damped analyses. However, with the knowledge garnered during the structural damping sensitivity analyses, the dynamic response of the system can be extrapolated with confidence for the damped condition.

4.7 Results from Test-1 and Test-2 Analyses

All ground motions from Test-1 and Test-2 were inspected with FLAC. Trends observed in the centrifuge data were noted, and the relative success of the FLAC model at capturing these
trends was examined. Additionally, several engineering design parameters of interest were directly compared between the FLAC output and the centrifuge data.

4.7.1 Test-2 (HBM03) Engineering Design Parameters

Several engineering design parameters (EDPs) were selected to provide metrics by which the relative success of the numerical model at capturing the responses observed in the centrifuge could be measured. These were determined with the assistance of structural engineering team members to ensure that the system response was adequately captured from both the geotechnical and structural standpoints. Each of these parameters is described below.

First was the transient peak roof drift. The modifier “transient” simply refers to the fact that this was a measure of the response of the system during strong shaking; it was not a permanent effect. It is described by Equation 4-11. The numerator represents the maximum value of the absolute difference between the total lateral displacement (in the shaking direction) recorded at the center of mass of the structure, \( \Delta x_f \), and the free-field total lateral displacement, \( \Delta x_g \). The peak roof drift obtained by normalizing this difference by the height of the structure, \( h \).

\[
\text{Equation 4-11} \quad \frac{\max |(\Delta x_f - \Delta x_g)|}{h}
\]

Next was the “residual” peak roof drift (Equation 4-12). This refers to the difference in the position of the superstructure roof before and after shaking. The modifier “residual” is included to indicate that it is a measure of permanent offset in the superstructure which remained after strong shaking had ceased. In this equation, \( \Delta x_f \) is the total lateral displacement (in the shaking direction) recorded at the center of mass of the structure and \( \Delta x_g \) is the total lateral displacement recorded at the free-field. The difference between the two yields the roof displacement. Dividing this by the height of the structure above the soil elevation, \( h \), yields the residual roof drift.

\[
\text{Equation 4-12} \quad \frac{(\Delta x_f - \Delta x_g) - (\Delta x_f - \Delta x_g)_i}{h}
\]

The maximum displacements and accelerations were also examined at:

1) The displacement at the center of mass of the superstructure’s roof, \( \Delta x_f \)
2) The displacement at the surface of the soil in the free-field, \( \Delta x_g \)
3) The acceleration at the center of mass of the superstructure’s roof, \( \ddot{u}_f \)
4) The acceleration at the surface of the soil in the free-field, \( \ddot{u}_g \)

The values for displacement from the centrifuge were obtained by double-integrating time histories recorded by accelerometers. The displacement time histories from FLAC were obtained directly from the program. Both centrifuge and FLAC time histories were filtered by Butterworth bandpass filters with lower and upper cutoff frequencies of 0.1 and 25 Hz (prototype), respectively.

Finally, pseudo-acceleration response spectra plots were generated for each motion at:

1) The base, to compare input motions between the centrifuge and FLAC;
2) The free-field surface, to check the relative success of the FLAC model in capturing the soil site response;
3) The center of mass of the structure, to show the relative success of the FLAC system at matching the observed experimental response.

For some motions, pseudo-acceleration response spectra were also examined at the footing level. This was to ensure that the footing input motion achieved in FLAC was representative of that observed in the centrifuge.

4.7.2 Test-1, 3-Story (prototype) Structure Results (MS1-1)
The Test-1 MS1-1 structure was examined for three configurations. The fixed-base natural periods for each are $T_{N,1} = 0.90 \text{ s (MS1-1(1))}$, $T_{N,2} = 0.82 \text{ s (MS1-1(2))}$, $T_{N,3} = 0.76 \text{ s (MS1-1(3))}$. The fuses used for Configurations 1 and 2 are identical; they exhibited the softest response, with permanent deformations visible post-spin. Configuration 3 had significantly stiffer fuses that did not exhibit obvious physical yielding or permanent offsets.

In the centrifuge, the footings were covered by 16 cm (model scale) of sand which corresponded to 0.9 m of cover, prototype. This embedment was expected to have a negligible effect on the structures’ dynamic response. To check the accuracy of this assumption, the effect of embedment was examined in the FLAC experimental suite. One set of analyses was conducted with spread footings founded at the correct elevation, but with no soil cover. Complementary analyses with identical structural configurations and embedment depth, but an overlying sand layer (0.9m, prototype), were run simultaneously. Additionally, the effect of applied structural Rayleigh damping was considered. The identified soil-structure damping ratio, $\beta_{SSI}$, for each motion was provided by the structural engineering research team (calculated post-spin, Chen, et.al 2012). Examining the trends in the data, an average value of Rayleigh damping, $\beta = 3\%$, was found to be a representative estimate, and was applied to the MS1-1 superstructure at its fixed-base fundamental frequency. Therefore, two scenarios were examined in FLAC (Figure 4-7(a), (b)). One scenario consisted of an undamped structure with uncovered footings; the other consisted of a damped ($\beta = 3\%$) structure with embedded footings.
### Test-1, MS1-1: Example of Mesh and Geometry

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Figure 4-7(a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No cover of footings</td>
<td></td>
</tr>
<tr>
<td>No structural Rayleigh damping</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Figure 4-7(b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedded footings</td>
<td></td>
</tr>
<tr>
<td>Structural Rayleigh damping, 3%</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4-7:(a) FLAC mesh used to model MS1-1 with elastic surface layer and UBCHYST for the remainder of the soil profile, no footing cover, and no structural Rayleigh damping. (b) FLAC mesh used to model MS1-1 with UBCHYST, footing cover, and structural Rayleigh damping of $\beta = 3\%$ applied at $f_{fixed}$.

The dynamic responses of MS1-1 to representative ground motions are presented for each of the four categories: ordinary low- and high-intensities and near-fault low- and high-intensities. For each of the selected motions, acceleration-time histories and acceleration response spectra are shown for both FLAC meshes. The soil-structure identified damping ratio ($\beta_{SSI}$) for each
motion, as calculated by Chen, et.al (2012) is included for reference, as well as the ground motion’s intensity and type. These plots are followed by a table of the engineering design parameters of interest for all ground motions (Table 4-5 and Table 4-6). Note that the accelerometers on the roof mass (at the top of the superstructure) went out of range for two motions; EDPs and spectra were not calculated for these motions (i.e., the PRImod and TCU_H motions). The dynamic responses from the complete ground motion suite, including all spectra and acceleration time histories, are presented in Appendix A.

4.7.2.1 Ordinary High and Low Intensity Motions

Ordinary motions are presented first. The Joshua Tree (JOS) recording presents a good overview of the responses observed for this type of dynamic excitation. The acceleration time histories of this type of record are characterized by a long duration motion, as might be observed at distances greater than 20 km from the fault plane and exhibited no directivity. The dynamic responses of MS1-1 to this type of motion were well captured by the FLAC model.

As may be observed, intensity levels of the JOS motion were bounded by the FLAC models (Figure 4-8 and Figure 4-9). The differences observed were due to primarily to the applied Rayleigh damping. The acceleration time histories show little change in any aspect other than amplitude.
Northridge, 1994 (Joshua Tree: JOSL2, Motion 2)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>2</td>
<td>3.14%</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 4-8: Joshua Tree, low intensity (JOS_L2), center-of-mass response. FLAC results at roof level for undamped structure on surface (blue) and damped structure (3%) embedded (green).
Northridge, 1994 (Joshua Tree, JOSH)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>2</td>
<td>2.58%</td>
<td>High</td>
</tr>
</tbody>
</table>

Figure 4-9: Joshua Tree, high intensity (JOS_H), center-of-mass response of MS1-1(2). FLAC results at roof level for undamped structure on surface (blue) and damped structure (3%) embedded (green).
4.7.2.2 Near-Fault High and Low Intensity Motions

The responses to the near-fault motions are presented next. These recordings were pulse-type motions, with relatively short durations. As for the ordinary motions, the dynamic response of the SMRF was well captured. Several ground motions are included for both high and low intensity levels to show the range of the numerical model.

The shape of the pseudo-acceleration response spectra was well captured by both the damped and undamped FLAC meshes. However, the numerical model consistently overpredicted the pseudo-acceleration spectral responses at the MS1 center of roof mass. The configuration with applied structural Rayleigh damping (3%) yielded the best fit; for the low intensity motions, including Sylmar Converter Station (SCS_L, Figure 4-10), Saratoga West Valley College (WVC_L, Figure 4-11) this response was only slightly higher than the observed values.

This trend continued in some of the high intensity motions. The Lucerne (LCN, Figure 4-12) motion showed a similar amplification to that observed in the lower-intensity motions. The Sylmar Converter Station (SCS_H) motion, conversely, was slightly underpredicted in the 1.2 s to 1.3 s period range. These motions induced significant nonlinearity, both in the soil and in the superstructure.
Figure 4-10: Sylmar Converter Station, low intensity (SCS L2), center-of-mass response. FLAC results at roof level for undamped structure on surface (blue) and damped structure (3%) embedded (green).
Figure 4-11: Saratoga West Valley College, low intensity (WVC_L), center-of-mass, response. FLAC results at roof level for undamped structure on surface (blue) and damped structure (3%) embedded (green).
Figure 4-12: Landers (LCN), high-intensity motion, center-of-mass response. FLAC results at roof level for undamped structure on surface (blue) and damped structure (3%) embedded (green).
Loma Prieta, 1989 (Sylmar Converter Station, SCSH)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>2</td>
<td>6.93%</td>
<td>High</td>
</tr>
</tbody>
</table>

Figure 4-13: Sylmar Converter Station, high intensity motion (SCSH) center-of-mass response. FLAC results at roof level for undamped structure on surface (blue) and damped structure (3%) embedded (green).
Based on examination of pseudo-acceleration response spectra and acceleration-time history data for MS1, the damped configuration with covered spread footings was selected as the most representative mesh for Test-1. The displacement time histories from FLAC were used to generate EDPs for all ground motions in the Test-1 suite, except for those motions that caused accelerometers in the centrifuge to go out of range (i.e., PRImod and WVC_H). Shown below are summaries of the transient drift and maximum transient displacement of the MS1-1 COM in figure form. These parameters are presented in Table 4-5 and Table 4-6 (this data is also presented graphically in (Figure 4-14 and Figure 4-15).

The transient drifts were very well captured for the ground motions in Test-1 (Figure 4-14). The transient drifts observed for each of the three structural configurations of MS1-1 were equally well described. While the numerical model did not perfectly capture the values, particularly for drifts greater than 1.2%, the trending patterns were matched. Transient displacements were similar (Figure 4-15). Though the exact values of displacement were not perfectly estimated by FLAC, the progression of the transient displacement induced by each motion was followed.
<table>
<thead>
<tr>
<th>Motion ID</th>
<th>CGM</th>
<th>FLAC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Transient Roof Drift</td>
<td>Residual Roof Drift</td>
</tr>
<tr>
<td></td>
<td>((\Delta_{\text{COM}} - \Delta_{\text{HA14}})_{\text{m/12.75m}})</td>
<td>((\Delta_{\text{COM}} - \Delta_{\text{HA14}})_{\text{final - initial m/12.75m}})</td>
</tr>
<tr>
<td>JOSL1</td>
<td>0.58%</td>
<td>0.0%</td>
</tr>
<tr>
<td>TCU</td>
<td>0.31%</td>
<td>0.0%</td>
</tr>
<tr>
<td>RRS</td>
<td>1.53%</td>
<td>0.0%</td>
</tr>
<tr>
<td>PTS</td>
<td>0.53%</td>
<td>0.0%</td>
</tr>
<tr>
<td>SCSL1</td>
<td>0.93%</td>
<td>0.7%</td>
</tr>
<tr>
<td>LCN</td>
<td>1.99%</td>
<td>0.0%</td>
</tr>
<tr>
<td><strong>Configuration 1</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>JOSL2</td>
<td>0.44%</td>
<td>0.0%</td>
</tr>
<tr>
<td>SCSL2</td>
<td>0.94%</td>
<td>0.0%</td>
</tr>
<tr>
<td>WVCL</td>
<td>1.45%</td>
<td>0.0%</td>
</tr>
<tr>
<td>SCSH</td>
<td>2.42%</td>
<td>0.0%</td>
</tr>
<tr>
<td>WPIIL</td>
<td>1.94%</td>
<td>0.0%</td>
</tr>
<tr>
<td>JOSH</td>
<td>1.71%</td>
<td>0.0%</td>
</tr>
<tr>
<td><strong>Configuration 2</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>JOSL3</td>
<td>0.53%</td>
<td>0.0%</td>
</tr>
<tr>
<td>WPIIH</td>
<td>2.32%</td>
<td>0.0%</td>
</tr>
<tr>
<td>TCUH</td>
<td>0.58%</td>
<td>0.0%</td>
</tr>
<tr>
<td>PRImod</td>
<td>Roof COM accelerometer out of range</td>
<td></td>
</tr>
<tr>
<td>WVCH</td>
<td>Roof COM accelerometer out of range</td>
<td></td>
</tr>
</tbody>
</table>
Table 4-6: MS1-1 maximum displacement, center of mass and free-field (Configurations 1-3)

<table>
<thead>
<tr>
<th>Motion ID</th>
<th>CGM</th>
<th>FLAC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Acceleration</td>
<td>Maximum Displacement</td>
</tr>
<tr>
<td></td>
<td>((ü, \text{COM})) (g)</td>
<td>((Δ\text{COM,}_x)) (m)</td>
</tr>
<tr>
<td><strong>Configuration 1</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>JOSL1</td>
<td>0.18</td>
<td>0.11</td>
</tr>
<tr>
<td>TCU</td>
<td>0.18</td>
<td>0.04</td>
</tr>
<tr>
<td>RRS</td>
<td>0.49</td>
<td>0.19</td>
</tr>
<tr>
<td>PTS</td>
<td>0.30</td>
<td>0.09</td>
</tr>
<tr>
<td>SCSL1</td>
<td>0.46</td>
<td>0.10</td>
</tr>
<tr>
<td>LCN</td>
<td>0.52</td>
<td>0.38</td>
</tr>
<tr>
<td><strong>Configuration 2</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>JOSL2</td>
<td>0.25</td>
<td>0.06</td>
</tr>
<tr>
<td>SCSL2</td>
<td>0.48</td>
<td>0.12</td>
</tr>
<tr>
<td>WVCL</td>
<td>0.57</td>
<td>0.27</td>
</tr>
<tr>
<td>SCSH</td>
<td>0.69</td>
<td>0.38</td>
</tr>
<tr>
<td>WPIH</td>
<td>0.54</td>
<td>0.28</td>
</tr>
<tr>
<td>JOSH</td>
<td>0.61</td>
<td>0.21</td>
</tr>
<tr>
<td><strong>Configuration 3</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>JOSL3</td>
<td>0.29</td>
<td>0.08</td>
</tr>
<tr>
<td>WPIH</td>
<td>0.72</td>
<td>0.33</td>
</tr>
<tr>
<td>TCUH</td>
<td>0.52</td>
<td>0.11</td>
</tr>
<tr>
<td>PRImod</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof COM accelerometer out of range</td>
<td>0.71</td>
<td>Roof COM accelerometer out of range</td>
</tr>
<tr>
<td>WVCH</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof COM accelerometer out of range</td>
<td>0.44</td>
<td>Roof COM accelerometer out of range</td>
</tr>
</tbody>
</table>
Figure 4-14: Peak transient drift of roof (COM) for MS1-1, 3% damped structure, embedded in UBCHYST.

Figure 4-15: Peak transient displacement of roof (COM) for MS1-1, 3% damped structure, embedded in UBCHYST.
4.7.3 Test-1, 9-story (prototype) Structure Results (MS3-1)

The MS3-1 structure was a SMRF with three degrees of freedom, designed to capture the response of a nine-story (prototype) structure founded on a three-story (prototype) basement. The basement was designed to be essentially rigid; as a result, this deep foundation was numerically modeled with beam elements. The cross sectional area and moment of inertia of the beam elements were increased by the researcher to ensure that the basement would exhibit a rigid response. The material density for the beams was correspondingly adjusted to achieve the appropriate total mass. Each of the beam elements was rigidly connected to the next, and all structural nodes were directly attached an adjacent gridpoint. Additionally, two beams were attached from the left upper to the right lower corners and the right upper to left lower corners of the basement to act as cross-bracing. These beams had identical dimensions to those used for the basement walls but no material density. Thus, they offered no contribution to the inertial response of the system, only additional stiffness. All spectra and acceleration time histories for the Test-1 ground motions are presented in Appendix A.

The fixed-base period of vibration of the structure was evaluated for each ground motion by the structural engineering researchers in the NEESR team, post-test. The fixed-based fundamental period varied by several tenths of a second over the course of the testing sequence ($T_{\text{fixed}} = 2.25$ s to $T_{\text{fixed}} = 2.86$ s). Averaging the fundamental periods identified from the various ground motions led to an average fixed-base estimate of $T_N = 2.7$ s. This value was used as the target for the MS3-1 structure during the FLAC analyses. The structure was not retuned to match the identified $T_{\text{fixed}}$ for each motion.

As mentioned when noting the progressive densification of the soil profile during shaking in Chapter 3, the goal of this research was to examine the robustness, the sensitivity, and the capabilities of FLAC for practical SSSI analyses of mid- to low-rise structures. The aim of this numerical analysis was to capture the trends and overall dynamic responses from a range of types (near fault, ordinary) and intensities (low and high) of ground motions. Therefore, the MS3-1 structure was not retuned for each motion; rather, the overall fit and the sensitivity of the model to changes in the system period were examined.

4.7.3.1 Test-1, MS3-1: Ordinary Low Intensity Motions

First, ordinary low-intensity motions were examined. These included three variations of the Joshua Tree motion (JOS_L1, JOS_L2, and JOS_L3), as well as the Parachute Test Site (PTS) and Chi Chi (TCU_L) recordings. MS3-1 responses to all motions were examined for several fixed-base fundamental periods to examine the sensitivity of the superstructure to this parameter in FLAC (plots for these configurations are included in the Appendix A).

With the exception of PTS, the spectra generated from the FLAC model were in close agreement with the centrifuge results. MS3-1 remained in the linear-elastic range for almost all motions during the centrifuge testing; therefore, damping in the superstructure due to strains or yielding was expected to be very small. In an undamped FLAC configuration, the JOS_L motions slightly overpredicted the dynamic response; the first of the three motions is shown in Figure 4-16. To correct this, a very small amount of Rayleigh damping was applied to the frame. The response was satisfactorily lowered with a Rayleigh damping value of 0.5%. Since this minimal level of damping satisfactorily captured the response of the MS3-1 steel SMRF superstructure, the most significant component of system damping was likely contributed by the
soil. Similarly, the response from the TCU_L motion was significantly overpredicted when no damping was applied. Applying 0.5% Rayleigh damping to the superstructure corrected this, resulting in a good fit in both the frequency and time domains (Figure 4-17).

However, for the PTS recording, neither the response nor the spectral shape from the centrifuge were well matched by the FLAC model at a MS3-1 of $T_{\text{fixed}} = 2.69$ s. As mentioned, the $T_{\text{fixed}} = 2.7$ s was an average of the fixed-base fundamental periods identified for MS3-1 from the test data; the period ranged from $T_{\text{fixed}} = 2.25$ s to $T_{\text{fixed}} = 2.86$ s during the testing sequence. Therefore, several other fundamental periods of vibration were examined for this specific motion.

This examination demonstrated that the MS3-1 structure exhibited significant sensitivity to fixed-base fundamental period when shaken with the PTS motion. Shifting the fixed-base fundamental period from $T_{\text{fixed}} = 2.69$ s to a longer estimate, $T_{\text{fixed}} = 2.73$ s, caused a negligible change in the MS3-1 response. Shifting the fixed-base period to a shorter estimate, however, induced a significant transformation. Two shorter configurations, $T_{\text{fixed}} = 2.67$ s and $T_{\text{fixed}} = 2.58$ s were examined. These exhibited progressively amplified superstructure responses with decreasing fixed-base fundamental period. The response spectra for $T_{\text{fixed}} = 2.67$ s was very consistent with that observed in the centrifuge.

The PTS motion exhibited marked sensitivity to small variations in structural properties (Figure 4-19). Neither the mean period ($T_m = 0.84$ s) of the PTS ground motion nor the predominant period ($T_p = 0.34$ s) were close to the superstructure’s fixed-base fundamental period. In conclusion, while the effect of structural properties is undeniable, it is not immediately obvious why the PTS motion induced such pronounced sensitivity to small changes in the fixed-base fundamental period of the MS3-1 structure.
Figure 4-16: Joshua Tree, low intensity (JOS_L1), MS3-1 (T_{fixed} = 2.69 s) with no structural Rayleigh damping (dashed) and with β=0.5% structural Rayleigh damping (solid).
Figure 4-17: Chi Chi, low intensity (TCU_L), MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Figure 4-18: Parachute Test Site (PTS) motion ($T_{\text{fixed}} = 2.69$ s) for MS3-1 with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Parachute Test Site (PTS): MS3-1, $T_{\text{fixed}} = 2.58, 2.67, 2.73$ s

**Figure 4-19(a)**
Fixed base fundamental period MS3-1.

$T_{\text{fixed}} = 2.58$ s

**Figure 4-19(b)**
Fixed base fundamental period of MS3-1.

$T_{\text{fixed}} = 2.67$ s

**Figure 4-19(c)**
Fixed base fundamental period MS3-1.

$T_{\text{fixed}} = 2.73$ s

Figure 4-19: Comparison of fixed-base period estimates (2.58, 2.67, and 2.73 seconds) for MS3-1 during Test-1 for PTS ground motion.
4.7.3.2 Test-1, MS3-1: Ordinary High Intensity Motions

Second, ordinary high-intensity motions were examined. These included two only – the Joshua Tree (JOS_H) and the Chi Chi (TCU_H) motions. The response to the JOS_H motion estimated from FLAC matched that experienced by the physical model (Figure 4-20). Application of a small amount of structural Rayleigh damping \((\beta = 0.5\% \text{ at } T_{\text{fixed}})\) caused little overall change in the response. In contrast, the TCU_H motion significantly overpredicted the response without some amount of Rayleigh damping applied to the superstructure (Figure 4-21). This response was consistent with that observed for the low-intensity version of the motion (TCU_L). Interestingly, the structural response of MS3-1 did not exhibit a clear first mode response for this motion (Chen et al., 2012).
Figure 4-20: Joshua Tree, high intensity motion (JOSH) MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Figure 4-21: Chi Chi, high intensity motion (TCU H) MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
4.7.3.3 Test-1, MS3-1: Near-Fault Low Intensity Motions

Third, near-fault low-intensity motions were examined. These included two applications of the Sylmar Converter Station (SCS_L1 and SCS_L2) motion, the West Valley College (WVC_L) recording, and the West Pico (WPI_L) motion. The SCS_L1 (Figure 4-25 and Figure 4-22) and WVC_L (Figure 4-23) motions spectral amplitudes were captured without applied structural damping. When structural Rayleigh damping ($\beta = 0.5\%$) was applied at $T_{\text{fixed}}$, the amplitude of the response was very slightly lowered – the overall fit remained very good. Conversely, the WPI_L (Figure 4-24) motion required Rayleigh damping ($\beta = 0.5\%$) to achieve the amplitude recorded in the centrifuge; the effect of damping on this motion was considerable.
Figure 4-22: Sylmar Converter Station, low intensity motion (SCS_L1) MS3-1 (T_{fixed} = 2.69 s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Figure 4-23 West Valley College, low intensity motion (WVC_L) MS3-1 (T_{fixed} = 2.69 s) with no structural Rayleigh damping (dashed) and with \( \beta = 0.5\% \) structural Rayleigh damping (solid).
Figure 4-24: West Pico, low intensity motion (WPI L) MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
4.7.3.4 Test-1, MS3-1: Near-Fault High Intensity Motions

Fourth, near-fault high-intensity motions were examined. These included Sylmar Converter Station (SCS_H), Port Island, Modified (PRImod), West Pico (WPI_H), Lucerne (LCN) and Rinaldi Recording Station (RRS) motions. The numerical model captured the shape and amplitude of the responses of SCS_H (Figure 4-25), PRImod (Figure 4-26), and LCN (Figure 4-28), and exhibited little sensitivity to applied structural Rayleigh damping. Consistent with the observed response at low-intensity, the WPI_H (Figure 4-27) motion overpredicted the amplitude of the response without slight structural Rayleigh damping ($\beta = 0.5\%$).
Figure 4-25: Sylmar Converter Station, high intensity motion (SCS_H) MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Figure 4-26: Port Island Modified, high intensity motion (PRImod) MS3-1 (T_f = 2.69 s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Figure 4-27: West Pico, high intensity motion (WPI_H) MS3-1 (T_{fixed} = 2.69 s) with no structural Rayleigh damping (dashed) and with \( \beta = 0.5\% \) structural Rayleigh damping (solid).
Figure 4-28: Lucerne, high intensity motion (LCN) MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Figure 4-29: Rinaldi Recording Station, high intensity motion (RRS) MS3-1 ($T_{\text{fixed}} = 2.69$ s) with no structural Rayleigh damping (dashed) and with $\beta=0.5\%$ structural Rayleigh damping (solid).
Engineering design parameters were calculated for the MS3-1 structure in the damped (0.5%) configuration. The values for peak transient drift and peak transient displacement estimate from the FLAC model are presented below (Table 4-7 and Table 4-8), with the centrifuge recorded values. Tabular representation of all the parameters follows Figure 4-30 and Figure 4-31.

The EDPs calculated from the MS3-1 superstructure were well matched overall. For transient displacements exceeding 0.4 m, some divergence between the numerical and physical modeling output was observed (Figure 4-30). However, the progression of values was captured by the numerical model. For transient drift values in excess of 1.1%, the precise values estimated from FLAC differed from those computed from the centrifuge data (Figure 4-31). The global trends of the values were described the numerical model. The fits to particular motions could likely be improved by tuning the MS3-1 structure to a more accurate value for that motion, as mentioned in Section 4.7.3: Test-1, MS3-1 Structure Results.
Table 4-7: Test-1 EDPs MS3-1 peak transient and residual roof drift

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<th>FLAC</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Peak Roof Drift</td>
<td>Residual Roof Drift</td>
</tr>
<tr>
<td></td>
<td>((\Delta_{\text{COM}} - \Delta_{\text{HA14}})_{\text{final}}) m</td>
<td>((\Delta_{\text{COM}} - \Delta_{\text{HA14}})_{\text{initial}}) m</td>
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<tr>
<td>JOSL1</td>
<td>0.3%</td>
<td>0.0%</td>
</tr>
<tr>
<td>TCU</td>
<td>0.1%</td>
<td>0.0%</td>
</tr>
<tr>
<td>RRS</td>
<td>0.6%</td>
<td>0.0%</td>
</tr>
<tr>
<td>PTS</td>
<td>0.5%</td>
<td>0.0%</td>
</tr>
<tr>
<td>SCSL1</td>
<td>0.5%</td>
<td>0.1%</td>
</tr>
<tr>
<td>LCN</td>
<td>1.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>JOSL2</td>
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<td>0.0%</td>
</tr>
<tr>
<td>SCSL2</td>
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<td>0.0%</td>
</tr>
<tr>
<td>WVCL</td>
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<td>0.0%</td>
</tr>
<tr>
<td>SCSH</td>
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<td>0.0%</td>
</tr>
<tr>
<td>WPIIL</td>
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<td>0.0%</td>
</tr>
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<td>JOSH</td>
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</tr>
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<td>JOSL3</td>
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</tr>
<tr>
<td>WPIH</td>
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</tr>
<tr>
<td>TCUH</td>
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<td>0.0%</td>
</tr>
<tr>
<td>PRImod</td>
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<td>0.0%</td>
</tr>
<tr>
<td>WVCH</td>
<td>No data from CGM</td>
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</tr>
</tbody>
</table>

MS3-1, \(T_{\text{fixed}} = 2.69\) s
Table 4-8: Test-1 EDPs, MS3-1 maximum displacement at center of mass and free-field.

<table>
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<th>Motion ID</th>
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<th>FLAC</th>
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<td></td>
<td>Maximum Acceleration</td>
<td>Maximum Displacement</td>
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<td>$(\ddot{u}, \text{FF})$ (g)</td>
</tr>
<tr>
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</tr>
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</tr>
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<td>PTS</td>
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<td>0.19</td>
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<td>LCN</td>
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<td>WVCH</td>
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</table>

MS3-1, $T_{\text{fixed}} = 2.69$ s

No data from CGM
Figure 4-30 Maximum transient displacement of roof (COM) for MS3-1, 0.5% damped structure.

Figure 4-31: Maximum transient drift of roof (COM) for MS3-1, 0.5% damped structure.
4.8 Test-2 (HBM03)

After the Test-1 baseline experiment had been successfully completed, the first experimental configuration examining SSSI, Test-2, was conducted. This test placed the same three-story (prototype) moment resisting frame (MS1-2) directly adjacent to the nine-story (prototype) superstructure, with deep basement (MS3-2). The two buildings were laterally separated by 0.27 m (prototype). This distance was chosen to prevent pounding or any other physical interaction that might occur between the two superstructures, while still positioning the buildings close enough to affect one another through the subsurface during strong shaking (i.e., through SSSI). The Nevada sand soil profile was deposited to target a relative density of 80%, as in Test-1.

4.8.1 Test-2: MS1-2 Configurations

Two configurations were used for the MS1-2 structure in Test-2. The first was identical to the initial design (Configuration 1) used in the Test-1 sequence. The heavy roof mass (4.11 kg, model scale) was reused, and beams with the long fuses (Fuse 3) were employed. Yielding was expected to occur, as in Test-1. The fixed-base fundamental period remained $T_{\text{fixed}} = 0.9$ s for this configuration. MSF1_SF80 Configuration 1 was used for ground motions 1-6 of Test-2.

For the remaining motions 7-21 of Test-2, the original floor mass (4.11 kg, model scale) remained, but stronger Fuse 2 beams replaced the original (Fuse 3) beams. With these stronger fuses in the beams, the model structure had an estimated fixed-base fundamental period of $T_{\text{fixed}} = 0.84$ s (Configuration 4). This value was first estimated from a fixed-base analysis in FLAC. The fixed-base fundamental period was then confirmed by the structural engineering research team members using the program OpenSees.

4.8.2 Test-2: MS3-2 Configurations

The MS3-2 structure in Test-2 was intended to be identical to the model used in Test-1, with the exception of one slight alteration to the basement to improve constructability of the model. In Test-1, the basement walls were directly below the columns of the superstructure. However, this required a slight overhang of the lid of the basement box to allow for the bolt connections between columns and basement (these connections straddled the basement walls). For Test-2, the basement was reconfigured so that the lid was flush with the walls. Connections between the basement and the supporting columns were made inside the basement walls.

This change was expected to exhibit negligible influence on the dynamic response of the system. One parameter which can provide insight into SSI is the period lengthening ratio; this value was used to examine the dynamic response of the MS3-2 structure for both Test-1 and Test-2, for comparison. The period-lengthening ratio is obtained by normalizing the soil-structure flexible-base period ($\tilde{T}$) by the first mode period of a fixed-base structure ($T_{\text{fixed}}$) yielding a parameter which provides insight into inertial interaction (Equation 4-13). The greater the period-lengthening ratio, the greater the inertial interaction of a system (Stewart, 1999). The alteration to the structural design slightly decreased the period-lengthening ratio from Test-1 to Test-2, from approximately 1.05 to 1.0 for most motions.

\[ \text{Equation 4-13} \quad \text{Period Lengthening Ratio} = \frac{T}{T_{\text{fixed}}} \]
The fixed-base natural period of the MS3-2 model structure unfortunately changed from Test-1 to Test-2. The period was stiffened from an average $T_{\text{fixed}} = 2.7$ s in Test-1 to $T_{\text{fixed}} = 2.4$ s for Test-2. After some discussion with the structural engineering team members, the most likely causes identified were tighter connections between beams and columns and the foundation during construction. The columns were not perfectly fabricated, so some twisting was necessary to adequately secure the nut-and-bolt connections. These connections may have been insufficiently tightened during Test-1.

### 4.8.3 Test-2: FLAC Mesh and Soil

The soil constitutive model parameters remained unchanged from Test-1 to Test-2. The mesh was regenerated with a grid configuration of 102 x 30 elements. The overall dimensions of the mesh were 75.4 m x 29.5 m. A schematic of the finished grid is presented below (Figure 4-32).

![Figure 4-32: Configuration of Test-2 mesh in FLAC](image)

Also, structural Rayleigh damping was examined to ascertain the model’s sensitivity to changes in its value. The more complicated numerical model required for Test-2 had a smaller timestep than the mesh used for Test-1, resulting in a significantly longer runtime per ground motion. A small value of 0.5% structural Rayleigh damping (identified as optimal in Test-1 MS3-2 analyses) either (1) brought the amplitudes of the response of the MS3-2 structure to levels comparable to the centrifuge data or (1) had a minimal effect (dependent on the motion). This small level of damping had a negligible effect on the MS1-2 structure. An example of this is
shown for the Lucerne (LCN) motion in Figure 4-33. Therefore, the ground motions were all run with no applied structural Rayleigh damping, due to time constraints. The amplitudes of the dynamic responses could be improved (i.e., lowered) by application of a small, realistic amount of structural damping (e.g., 0.5%) if analysis time were not a constraint.
Figure 4-34: Lucerne (LCN) near-fault motion, with no applied structural Rayleigh damping and 0.5% structural Rayleigh damping for MS3-2.
4.8.4 Results: Comparison of Motions 1-12 (MS1-2 Configuration 1 and Configuration 4)

Six ground motions were run with MS1-2 Configuration 1: Joshua Tree (JOS_L), Rinaldi Recording Station (RRS), Chi Chi (TCU_L), Parachute Test Site (PTS), Lucerne (LCN), and Sylmar Converter Station (SCS_L). After successful completion of that testing sequence, the arm was spun-down and the yielded beams of MS1-2 were replaced. This exchange converted the structure from Configuration 1 ($T_{\text{fixed}} = 0.9$ s) to Configuration 4 ($T_{\text{fixed}} = 0.84$ s). The same six ground motions (motions 7-12 in the global Test-2 sequence) were rerun in the centrifuge to observe the effect of altering the structural characteristics of MS1-2 on its dynamic response.

The pseudo-acceleration response spectra and acceleration time histories from motions applied to the two SMRFs were compared for both MS1-2 (Configuration 1, $T_{\text{fixed}} = 0.9$ s) and MS1-2 (Configuration 4, $T_{\text{fixed}} = 0.84$ s). Of the six motions, half were ordinary ground motions (JOS_L, PTS, and TCU_L) and half were near-fault, forward directivity motions (RRS, LCN, and SCSL). Note also that the RRS and LCN motions were both high-intensity motions.

For four of the motions, LCN (Figure 4-35), RRS (Figure 4-37), JOS_L (Figure 4-38), and PTS (Figure 4-40), little difference in the spectral shape was observed in the centrifuge. Some amplification occurred at the center of the roof mass of each MRFs, with JOS_L exhibiting nominal amplification and PTS the greatest amplification. For the remaining two motions, SCS_L (Figure 4-36) and TCU_L (Figure 4-39), a considerable shift in dynamic response from MS1-2 (Configuration 1, $T_{\text{fixed}} = 0.9$ s) to MS1-2 (Configuration 4, $T_{\text{fixed}} = 0.84$ s) was visible, altering the shape of the pseudo-acceleration response spectra and the amplitude of response. The FLAC model accurately captured all of these nuances.
Figure 4-35: Lucerne (LCN), high intensity motion; center of roof mass responses
MS1-2: Configuration 1: $T_{\text{Fixed}} = 0.90$ s

Sylmar Converter Station (SCSL_1)

Figure 4-36: Sylmar Converter Station (SCS_L), high intensity motion; center of roof mass responses
Figure 4-37: Rinaldi Recording Station (RRS), high intensity motion; center of roof mass responses
MS1-2: Configuration 1: $T_{\text{Fixed}} = 0.90$ s

Joshua Tree (JOS_L1)

Figure 4-38: Joshua Tree (JOS_L), low-intensity motion; center of roof mass responses
MS1_SF80: Configuration 1: $T_{\text{Fixed}} = 0.90$ s

Chi Chi (TCUL_1)

Figure 4-39: Chi Chi (TCU_L), low-intensity motion; center of roof mass responses
Figure 4-40: Parachute Test Site (PTS), low-intensity motion; center of roof mass responses
4.8.4.1 Test-2, MS1-2 and MS3-2: Ordinary High Intensity Motions

Two of the ordinary motions were applied to the centrifuge model at high intensity: Joshua Tree (JOS_H) and Chi Chi (TCU_H). As may be seen, the amplitudes of the MS3-2 pseudo-acceleration response spectrum for the JOS_H record were better matched for this high-intensity motion than for the low-intensity motion (Figure 4-41). The MS1-2 dynamic response from FLAC exhibited a slight deviation from the observed spectral shape for the TCU_H motion (Figure 4-42(a)). This difference in dynamic response was not observed for any other motions in the sequence. No obvious explanation was identified during a check of the properties of the ground motion and the data from the centrifuge. Since the motion was well-captured during its two low intensity applications, and the effect was not seen in any other motions, this was likely a physical effect present in the centrifuge environment that was not captured in the numerical model. In contrast, the MS3-2 response to this recording was well-captured by FLAC.
**Figure 4-41:** Joshua Tree (JOS_H), high-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-2

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<th>Period (sec)</th>
<th>$S_a$ (g)</th>
<th>$\lambda = 5%$</th>
</tr>
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<td>MS1-2, Roof, CGM</td>
<td></td>
<td></td>
</tr>
<tr>
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<td></td>
<td></td>
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<tr>
<td>MS1-2, Roof, FLAC</td>
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**Figure 4-42:** Chi Chi (TCU_H), high-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-2

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>$S_a$ (g)</th>
<th>$\lambda = 5%$</th>
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<tbody>
<tr>
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<td>Base, FLAC</td>
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</table>
4.8.4.2 Test-2: Near-Fault High and Low Intensity Motions

Fourth, near-fault high-intensity motions were examined. The West Valley College (WVC) motion was applied to the centrifuge model MS1-2 Configuration 4 only, for both high and low intensities. Therefore, the low-intensity spectral response is presented above the high-intensity response to show the differences in dynamic response of MS1-2 and MS3-2 due to shaking amplitude (Figure 4-43 and Figure 4-44). The spectral shapes of the WVC motions were well captured by both models for both intensities. Changes in overall frequency content were reflected by the FLAC models of both MS1-2 and MS3-2. The shaking amplitude associated with the higher intensity motion was not achieved by the MS1-2 numerical model. While some amplification was reflected in the FLAC output, with $S_a$ maximums shifting from 2.75 g for WVC_L to just over 3 g for WVC_H, the 3.75 g achieved in the centrifuge were not matched. Meanwhile, the acceleration amplitudes recorded at roof masses of the MS3-2 centrifuge structure were closely matched by the FLAC model.

Also included in this group were the Sylmar Converter Station (SCS_H), Port Island Modified (PRImod), West Pico (WPI_H), and Lucerne (LCN) motions. Both PRImod (Figure 4-47) and WPI_H (Figure 4-45) induced responses in the numerical model which exhibited excessive amplification in the MS3-2 response spectra. This could be corrected with application of a small amount (0.5%) structural Rayleigh damping. The Lucerne motion was discussed earlier, while comparing the responses of the MS1-1 Configurations 1 and 2 (Figure 4-35). The responses induced by the SCS_H motion in both MS3-2 and MS1-2 were very representative of those recorded in the centrifuge (Figure 4-46).
Figure 4-43: West Valley College (WVC_L), low-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-2

Figure 4-44: West Valley College (WVC_H), high-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-1
Figure 4-45: West Pico (WPI_H), high-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-2.

Figure 4-46: Sylmar Converter Station (SCS_H), high-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-2.
Figure 4-47: Port Island Modified (PRImod), high-intensity motion; center of roof mass responses at (a) MS1-2 and (b) MS3-2
4.8.5 Test-2 Engineering Design Parameters (EDPs)

As for Test-1, the EDPs of interest were computed for MS1-2 and MS3-2 in Test-2. Overall, the dynamic responses estimated from the FLAC model were closely correlated with those recorded in the centrifuge. The transient drifts of the roof mass of MS1-2 computed from FLAC displacement time histories closely paralleled the values calculated from the centrifuge data. Importantly, the dynamic response for every motion was correctly mapped as an increase or decrease by the FLAC model (Figure 4-49). While the progressions of transient drift were well-captured, the numerical estimates of the parameter under-predicted the responses from MS1-2 for all ground motions, particularly those which induced transient drifts over 1.9%. Another key parameter, transient displacement of the roof mass, was also closely matched by the FLAC output. All trends observed in the centrifuge were captured, with increasing and decreasing values from motion to motion clearly matched by the numerical model output (Figure 4-48). For transient displacement values under 0.2 m (prototype), the FLAC output matched the recorded values almost exactly. When transient displacement values exceeded 0.2 m, the numerical model slightly underpredicted the dynamic response of MS1-2. While the exact values of transient drift and transient displacement were not precisely matched for all 20 ground motions, the general trends were captured for every motion in the suite.
<table>
<thead>
<tr>
<th>Motion ID</th>
<th>CGM</th>
<th>FLAC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Roof Drift</td>
<td>Residual Roof Drift</td>
</tr>
<tr>
<td></td>
<td>((\Delta_{\text{COM}} - \Delta_{HA14})/12.75\text{m})</td>
<td>((\Delta_{\text{COM}} - \Delta_{HA14})_{\text{final}} - \text{m/12.75m})</td>
</tr>
<tr>
<td>Configuration 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>JOSL1</td>
<td>0.4%</td>
<td>0.0%</td>
</tr>
<tr>
<td>TCU</td>
<td>0.3%</td>
<td>0.0%</td>
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<td>RRS</td>
<td>1.6%</td>
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</tr>
<tr>
<td>PTS</td>
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<td>0.1%</td>
</tr>
<tr>
<td>LCN</td>
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<td>0.0%</td>
</tr>
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<td>Configuration 4</td>
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<td>TCU2</td>
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<td>JOSH</td>
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<tr>
<td>WPI</td>
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<tr>
<td>PR1mod</td>
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</tr>
<tr>
<td>TCUH</td>
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</tr>
<tr>
<td>WVCH</td>
<td>1.5%</td>
<td>2.4%</td>
</tr>
</tbody>
</table>
Table 4-10: Summary of displacements (COM and FF) and maximum accelerations for MS1-2

| Motion ID | CGM | | | | | | FLAC | | | |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| | Maximum Acceleration | Maximum Displacement | Maximum Displacement | | | | | | | | | | | | |
| | (ü, COM) (g) | (ü, FF) (g) | (ΔCOM, x) (m) | (ΔFF, x) (m) | (ΔCOM, x) (m) | (ΔFF, x) (m) |
| Configuration 1 | | | | | | | | | | | | | | | | |
| JOSL1 | 0.16 | 0.11 | 0.06 | 0.03 | 0.05 | 0.03 |
| TCU | 0.14 | 0.19 | 0.04 | 0.03 | 0.03 | 0.03 |
| RRS | 0.46 | 0.37 | 0.20 | 0.09 | 0.15 | 0.09 |
| PTS | 0.32 | 0.26 | 0.12 | 0.06 | 0.12 | 0.05 |
| SCSL1 | 0.36 | 0.33 | 0.11 | 0.05 | 0.09 | 0.05 |
| LCN | 0.47 | 0.32 | 0.38 | 0.21 | 0.32 | 0.19 |
| Configuration 4 | | | | | | | | | | | | | | | | |
| JOSL2 | 0.18 | 0.12 | 0.06 | 0.03 | 0.06 | 0.03 |
| TCU2 | 0.19 | 0.22 | 0.05 | 0.03 | 0.04 | 0.03 |
| RRS2 | 0.58 | 0.41 | 0.22 | 0.10 | 0.17 | 0.10 |
| PTS2 | 0.37 | 0.26 | 0.13 | 0.06 | 0.13 | 0.05 |
| SCSL2 | 0.43 | 0.34 | 0.13 | 0.05 | 0.10 | 0.04 |
| LCN2 | 0.65 | 0.35 | 0.39 | 0.21 | 0.33 | 0.20 |
| WVCL | 0.59 | 0.42 | 0.31 | 0.16 | 0.29 | 0.17 |
| SCSH | 0.75 | 0.60 | 0.41 | 0.24 | 0.38 | 0.22 |
| JOSH | 0.72 | 0.54 | 0.29 | 0.13 | 0.25 | 0.14 |
| WPI | 0.65 | 0.44 | 0.36 | 0.22 | 0.33 | 0.19 |
| PRImod | 0.80 | 0.68 | 0.34 | 0.16 | 0.26 | 0.13 |
| TCUH | 0.38 | 0.49 | 0.09 | 0.05 | 0.08 | 0.07 |
| WVCH | 0.70 | 0.53 | 0.46 | 0.27 | 0.41 | 0.28 |
Figure 4-48: Maximum displacement ($\Delta_{COM,x}$) at roof (COM) of MS1-2, FLAC compared with centrifuge (CGM)

Figure 4-49: Peak transient drift at the roof (COM) of MS1-2, FLAC compared with centrifuge (CGM)

MS3-2 produced similar results. When the transient displacements exceeded 0.2 m, the FLAC results underestimated the dynamic response of MS3-2 for all ground motions, as for MS1-2 (Figure 4-50). However, as before, the numerical model did accurately capture the overall trends. The transient displacement values from the centrifuge which fell under 0.2 m were almost perfectly matched by the FLAC output, with the exception of the JOS_L motions (these records were consistently underpredicted). The transient drifts were also well captured (Figure 4-51). Again, the output from FLAC underpredicted the responses of MS3-2 as the calculated drifts increased. The trends were consistent between the centrifuge data and FLAC output, providing confidence in the ability of the FLAC model to accurately estimate and provide insight into the dynamic response of the SMRFs.
Table 4-11: Summary of peak transient and residual roof drifts (COM) for MS3-2

<table>
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<th>Motion ID</th>
<th>CGM</th>
<th>FLAC</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Peak Roof Drift</td>
<td>Residual Roof Drift</td>
</tr>
<tr>
<td></td>
<td>$(\Delta_{\text{COM}} - \Delta_{\text{HA14}})/34.75$ m</td>
<td>$(\Delta_{\text{COM}} - \Delta_{\text{HA14}})<em>{\text{final}} - (\Delta</em>{\text{COM}} - \Delta_{\text{HA14}})_{\text{init}} /34.75$ m</td>
</tr>
<tr>
<td>JOSL1</td>
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<td>0.01%</td>
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<tr>
<td>TCU</td>
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<td>0.13%</td>
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<td>RRS</td>
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<td>PTS</td>
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<td>SCSL1</td>
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<tr>
<td>LCN</td>
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<td>0.00%</td>
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<tr>
<td>TCUUL2</td>
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<tr>
<td>RRS2</td>
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<td>PTS2</td>
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<tr>
<td>WVCH</td>
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Table 4-12: Summary of maximum displacements (COM and FF) and maximum accelerations for MS3-2

<table>
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<th>Motion ID</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Acceleration</td>
<td>Maximum Displacement</td>
</tr>
<tr>
<td></td>
<td>(\ddot{u}, COM) (g)</td>
<td>(\ddot{u}, FF) (g)</td>
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<tr>
<td>JOSL1</td>
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<tr>
<td>RRS</td>
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<td>0.37</td>
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<td>PTS</td>
<td>0.34</td>
<td>0.26</td>
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<td>SCSL1</td>
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<td>0.33</td>
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<td>LCN</td>
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<td>0.32</td>
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<tr>
<td>JOSL2</td>
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<td>0.12</td>
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<td>TCUL2</td>
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<td>RRS2</td>
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<td>0.41</td>
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<tr>
<td>WVCH</td>
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</tr>
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</table>
4.9 Test-2: Altered Structural Configuration, MS1-2 without MS3-2

After extensive work with the FLAC models of MS1-2 and MS3-2 from Test-2 the effect of removal of the nine-story structure on the remaining SMRF was of interest. Therefore, the numerical mesh created for Test-2 was rerun, with a slight alteration. The well-calibrated MS1-2 frame remained unchanged, in the structural Configuration 4 \( (T_{\text{fixed}} = 0.84 \text{ s}) \), while the MS3-2 structure was removed from the input file. The mesh and soil properties remained unchanged – the only difference was the removal of the MS3-2 superstructure and basement (Figure 4-52).
4.9.1 Results from Test-2: Altered Structural Configuration, MS1-2 without MS3-2

Four ground representative ground motions from the Test-2 suite were selected for use in this analysis, one from each type of dynamic excitation. The Joshua Tree motion was used to provide insight into both the high- and low-intensity, ordinary ground motions (JOS_L1 and JOS_H). The Sylmar Converter Station (SCS_L1) record served as the near-fault, low-intensity motion, while Lucerne (LCN) functioned as the high-intensity, near-fault recording.

The dynamic response at the center of the roof mass of the now solitary MS1-2 frame was compared with that previously observed for the SSSI adjacent case (Test-2). Both the centrifuge results and the numerical output from that configuration are plotted for reference. Notably, the MS3-2 structure did have a discernible effect on the dynamic response of the MS1-2 frame.

First, the flexible-base period ($T_{SSI}$) was slightly lengthened when the nine-story structure was removed. This is visible in the spectra as a shift to a longer period range. It is also seen in the acceleration time histories after strong shaking has ceased; this effect is clearest in the short-duration near-fault motions (after ~37 s the SCS_L1 record, and after ~40 s in the LCN history). This could be related to the loss of confinement that the deep basement provided. When MS3-2 was adjacent, the soil was constrained in that (lateral) direction. Once that basement wall was removed, the soil was free to displace in either direction, exhibiting more nonlinearity which was transmitted to the structure through the footings.
Second, the amplitude of the overall acceleration response decreased. This could be due to increased damping in the subsurface as a result of increased nonlinearity due to the loss of confinement from the adjacent basement. Alternatively, some of the dynamic energy from the MS3-2 structure could have been transferred to the adjacent MS1-2 frame through SSSI. This effect is most noticeable in the two ordinary ground motions (JOS_L1 and JOS_H).
Figure 4-53: Sylmar Converter Station (SCS_L1) near-fault motion, MS1-2 with no MS3-2 structure, no applied structural Rayleigh damping (0%).
Figure 4-54: Lucerne (LCN) near-fault motion, MS1-2 with no MS3-2 structure, no applied structural Rayleigh damping (0%).
Figure 4-55: Joshua Tree (JOS_L1) ordinary motion, MS1-2 with no MS3-2 structure, no applied structural Rayleigh damping (0%).
Figure 4-56: Joshua Tree (JOS_H) ordinary motion, MS1-2 with no MS3-2 structure, no applied structural Rayleigh damping (0%).
4.10 Summary of Test-1 and Test-2 Results

In summary, dynamic responses from the special moment resisting frames, MS1-1 and MS1-2, MS3-1 and MS3-2, for both Tests-1 and -2 were well-captured by the FLAC numerical model. The structures from Test-1 were modeled separately, to establish a baseline SSI case for comparison with future tests. Three MS1-1 structural configurations (Configurations 1 – 3) and a single MS3-1 configuration (T_{\text{fixed}} = 2.69 s) were modeled in separate analyses to best approximate the solitary, SSI condition. Pseudo-acceleration response spectra, acceleration time histories, and EDPs of interest were all well matched and trends with motion sequence were captured for both MRFs in the Test-1 geometry.

Test-2, which positioned the two SMRFs adjacent to examine the effects of SSSI, was also well captured by the numerical model. Trends in EDPs, including transient drift, maximum displacement and acceleration, were matched. Some under-prediction of transient drifts and displacements at the roof mass of the SMRFs occurred.

Finally, an additional examination of the effect of removal of the MS3-2 structure from the Test-2 configuration was conducted in FLAC. The effect of MS3-2 on its neighbor was explored by rerunning the numerical mesh and MS1-2 structure from the Test-2 analysis with MS3-2 removed. The output from this SSI analysis showed that the soil-structure flexible-base period of the MS1-2 frame was altered, and the amplitude of the acceleration time history was lowered relative to the SSSI case. The flexible-base period lengthening and the decreased dynamic response of MS1-2 are both indicative of effects of SSSI between the MS3-2 and MS1-2 structures.
5 Structural Analysis, Test-3

5.1 Introduction to Test-3

The first two tests in the NCB centrifuge sequence (Test-1 and Test-2) successfully examined the responses of simplified prototypical structures in both (1) ‘solitary’ configurations to establish baseline SSI responses of two MRF structures and (2) an ‘adjacent’ layout, which used neighboring MRFs to examine the effects of SSSI. The structures from both tests exhibited effects of SSI and SSSI, including period lengthening and some kinematic interaction. However, neither of the tests manifested quantifiable inertial interaction.

Therefore, the next experiment, Test-3 (HBM04), was designed to maximize inertial interaction. Three structures were used for this centrifuge test. Two of these were single-degree-of-freedom MRFs, which were variations of the MS1-1 and MS1-2 frames from Test-1 and Test-2 (). The third was a two-story (model) elastic shear wall structure (designated MS2F_M). These structures were designed to maximize interaction with both the soil profile and each other through careful optimization of SSI parameters. The soil profile remained consistent with Test-1 and Test-2; it was composed of dense (Dr = 80%), dry Nevada sand.

The three structures of Test-3 were oriented in the CGM flexible shear beam container (FSB2.2) to achieve the two goals defined for this test. The first was to examine the behavior of the newly designed three-story (prototype) MRF in a solitary, baseline SSI case; therefore, one of the models was positioned in the North half of the box (herein referred to as MS1-SSI). Establishing a baseline for this structure was necessary, as the frame’s dimensions were significantly altered from the previous experiments, making direct comparison between the three-story frame of Test-3 and the three-story structure of Test-1 and Test-2 unreasonable. In the opposite half of the container (i.e., the South section), the second three-story (prototype) frame MS1-SSSI and the MS2F_M structure were placed immediately adjacent. These two models were configured to provide data on SSSI effects. A schematic of the geometry is shown below (Figure 5-1).

The baseline (e.g., the single three-story frame control structure ) and adjacent (MS2F_M next to MS1-SSSI) configurations were each examined in FLAC. As described in Chapter 4, the structural and soil responses from FLAC were compared with those recorded in the centrifuge. Again, the relative accuracy of the models created in FLAC was quantified though:

1) Comparison of time histories recorded during centrifuge testing and those calculated using FLAC.
2) Comparison of several engineering design parameters from the data recorded during the centrifuge test and the results of the FLAC analyses.
Figure 5-1: Schematic of Test-3, single (SSI) case on left, adjacent (SSSI) case on right; dimensions are in model scale and should be multiplied by 55 for prototype scale dimensions. (Mason, et al., Data Report, 2010)

5.2 Test-3: Structural Model Background

The physical structural models for this test were designed using a different approach than that of Test-1 and Test-2. Recall that the first two tests had focused on capturing the response of structures with realistic prototypical design parameters. As outlined in Chapter 4, the structures in Test-1 and Test-2 were specifically selected to best model some structural configurations that are commonly found in the greater Los Angeles area. This goal was achieved for those experiments. As a result, however, the observed SSI and SSSI effects were almost exclusively due to kinematic interaction.

Therefore, the next test in the NEES city-block sequence, Test-3, aimed to examine a different aspect of SSSI – inertial interaction. This goal allowed the researchers to pursue a methodology focused entirely on the laboratory testing environment while designing structural models for the centrifuge (e.g., prototypical drift ratios and normalized base shears were no longer constraints). An extensive literature review of the current SSI and SSSI knowledge base led to several design criteria for the physical structural models.

First, it was recognized that inertial interaction is maximized when a resonant condition is achieved. Resonance occurs when a soil column’s fundamental period aligns with a structure’s fundamental period. In extreme cases, a ‘maximum’ resonant condition may be attained when a ground motion’s mean period also aligns with the fundamental period of the structure and site. Since the experimental soil profile was constant throughout the NEES city-block testing sequence (Test-1 through Test-4), an estimate of the fundamental period of the soil ($T_{soil}$) was
made based on data from Test-1 and Test-2, yielding $T_{\text{soil}} \sim 0.6 \text{ s}$. Therefore, one of the identified design parameters for the structural models was a soil-structure interaction (flexible-base) fundamental period ($T_{\text{SSI}}$) of 0.6 s.

Second, the relative masses of structures were observed to be of importance. Therefore, the weights of the three structures used for Test-3 were different; the MS1-SSI and MS1-SSSI frames were considerably lighter than the MS2F_M structure. The MS2F_M structure, an elastic shear wall design, was deliberately engineered to respond to dynamic excitation at the site period without exhibiting yielding. This strictly elastic response was intended to force energy back into the soil (through the “feedback” loop described in the Inertial Interaction section of Chapter 2) with negligible energy dissipation in the superstructure itself. Ultimately, this would cause more total energy to be transmitted to the much lighter adjacent MS1-SSSI structure through SSSI than if no adjacent structure were present. In other words, the MS2F_M structure was intended to act as a “transmitting” structure while the MS1-SSSI frame was intended to serve as a “receiving” structure during seismic shaking.

Third, several dimensionless design parameters which indicate the likelihood of significant SSI effects were identified from the literature review (e.g., Veletsos and Meek, 1974; Veletsos and Nair, 1975; Stewart, et al. 1999). These dimensionless parameters were used to optimize the expected inertial response of the MS2F_M structure. The most significant predictor of SSI is the wave parameter ($1/\sigma$), given by Equation 5-1.

\textbf{Equation 5-1} \quad \frac{1}{\sigma} = \frac{h_{\text{eff}}}{T_{\text{fixed}}V_s}

In this relationship, $h_{\text{eff}}$ is the effective height of the structure, $T_{\text{fixed}}$ is the fixed-base estimate of the building, and $V_s$ is the average shear wave velocity. To maximize SSI, this ratio should be greater than 0.2.

The next important parameter to be identified was the equivalent foundation radius ratio, defined as the ratio of effective height to the effective foundation radius (in the rocking direction, $r_\theta$). This parameter was optimized when between 4 and 0.1 (Equation 5-2).

\textbf{Equation 5-2} \quad 4 > \frac{h_{\text{eff}}}{r_\theta} > 0.1

The third important parameter to be considered, $\delta$, relates the mass of the superstructure to the mass of the soil (Equation 5-3). This equation normalizes the mass of the superstructure ($m_{\text{struc}}$) to the average density of the soil ($\rho_{\text{soil}}$), the effective foundation radius dimension in translation ($r_{\text{eff}}$), and the effective height of the structure ($h_{\text{eff}}$). To ensure inertial interaction, this parameter should be greater than 0.15.

\textbf{Equation 5-3} \quad \delta = \frac{m_{\text{struc}}}{r_{\text{eff}}h_{\text{eff}}\pi\rho_{\text{soil}}} > 0.15

Finally, three additional relationships shown to have an effect on SSI were also examined. While not as significant predictors as the previous set (e.g., Equation 5-1 through Equation 5-3), they have been found to accurately describe the inertial interaction potential of a system.
\[
\frac{D_f}{r_{eff}} > 0.25
\]
\[
\frac{m_{\text{foundation}}}{m_{\text{struct}}} \sim \text{minimize}
\]
\[
\frac{L}{2h_{eff}} \sim \text{minimize}
\]

In Equation 5-4, the depth term, \(D_f\), refers to the depth of embedment of the foundation. The term \(L\) is the length of the foundation measured in the plane of translation. Finally, in Equation 5-5, the term \(m_{\text{foundation}}\) is the total mass of the foundation.

Within the centrifuge container, the adjacent models (MS2F_M and MS1-SSSI) were placed 0.275 m (prototype scale) away from each other. As in Test-2, this distance was chosen to avoid pounding while ensuring sufficient proximity to maximize SSSI effects.

### 5.3 Centrifuge Structural Model Construction

#### 5.3.1 Physical Model: MS2F_M

The MS2F_M model was the first non-MRF structure used. The model was composed of two shear walls, welded to a shallow mat foundation (footprint of 13.8 m by 13.8 m, prototype). Masses were located at both the roof and the single floor level, forming a two-story (model) elastic rocking structure. The structure itself was composed exclusively of Aluminum 6061 plate stock.

Since the goal of this analysis was to achieve maximum SSSI through resonance with the soil column, the flexible-base fundamental period of the structure (\(T_{\text{MS2F}_{\text{SSI}}}\)) was targeted at \(T_{\text{MS2F}_{\text{SSI}}} = T_{\text{soil}} = 0.6\) s. As for Test-1 and Test-2, the model dimensions and properties necessary to achieve this condition were determined through iteration with the program, OpenSees (http://opensees.berkeley.edu). A design fixed-base fundamental period (\(T_{\text{MS2F}_{\text{FB}}}\)) of 0.13 s was determined to meet the flexible-base natural period target.

#### 5.3.2 Numerical Model: MS2F_M

The development of a plane-strain equivalent model of MS2F_M was slightly different than that of the previous MRFs. To begin, the distributed masses of the mat, walls, and centered floor masses were obtained by dividing the actual mass by the actual dimensions (i.e., \(z\)-dimension, into the page) of the physical model. In this configuration, the floor and roof extended laterally into the page \((z\)-dimension\), while the walls were oriented parallel to the direction of shaking. This yielded a distributed mass per unit length for use in plane strain analyses. Since the MS2F_M rocking wall was configured with its walls parallel to the direction of shaking (i.e., perpendicular to the \(z\)-direction, into the page) beam elements in FLAC were not the best choice to model the superstructure. Instead, solid elements were selected.

A mesh composed of 35 by 38 zones was filled with an elastic constitutive model to provide a realistic representation of the MS2F_M aluminum shear wall. Young’s modulus of aluminum was used to determine the bulk and shear moduli of the structure, while material densities for the walls, floor, and roof were obtained from distributed area calculations (see Appendix B for MathCAD calculations). A small amount of Rayleigh damping (0.5%) was
applied to the mesh to prevent numerical oscillations. The centered masses at the floor and the roof were modeled in two ways: solid elements or beam elements. No differences in dynamic structural response were observed between these two modeling approaches, so the simpler solid element approach was used for all analyses.

The fixed-base fundamental period of MS2F_M was matched using the method described in Chapter 4 (as implemented for MRFs in Test-1 and Test-2). A unit velocity was applied to the base of the structure and then removed, and the wall was allowed to vibrate freely for approximately 20 cycles. The resulting acceleration-time history at the center of mass of the wall was examined, both in the frequency domain with an FFT and in the time domain with a simple check of the number of cycles of free vibration per length of time. The final structural numerical model exhibited a fixed-base period of 0.14 s.

5.3.3 Physical Model: MRFs MS1-SSI and MS1-SSSI

The general design of the MS1-SSI and MS1-SSSI structures was similar to that of the MRFs used in Test-1 and Test-2. The model was a single degree-of-freedom system, with a lumped mass at the roof level of the frame, founded on shallowly embedded spread footings (4.4 m by 4.4 m, prototype). However, the dynamic properties of the frame structure differed significantly from the previous tests.

The flexible-base fundamental period \( T_{MS1\_SSI} \) targeted for MS1-SSI and MS1-SSSI was equal to the site period, \( T_{MS1\_SSI} = T_{soil} = 0.6 \) s. As before, structural engineering team members targeted this value with the computer program, OpenSees. Design specifications for this configuration differed from Test-1 and Test-2 in two primary ways. First, the structure was 2.75 m (prototype) shorter than MS1-1 and MS1-2 from Test-1 and Test-2, with vertical heights of 9.95 m for Test-3 and 12.75 m for Test-1 and Test-2. Additionally, the centered masses for MS1-SSI and MS1-SSSI were significantly lighter in Test-3 than in Test-1 and Test-2 (1.75 kg versus 4.11 kg, model scale). The strongest fuse configuration (e.g., Fuse 3, from Test-1 and Test-2) was employed for the beams; column properties remained consistent throughout the city-block testing sequence. These changes shifted the fixed-base fundamental period to 0.47 s, well outside the previous range of 0.76 s to 0.90 s from Test-1 and Test-2.

5.3.4 Numerical Model: MS1-SSI and MS1-SSSI

Returning to the global orientation of the structures in Test-3, it is important to recall that each of the three-story (prototype) MRFs used in this testing sequence captured different conditions. One represented a “solitary” SSI case in the North half of the container (MS1-SSI). The other acted as a “receiving” structure (MS1-SSSI) adjacent to MS2F_M and provided insight into SSSI. While the locations yielded different dynamic configurations and responses, the solitary and adjacent frames both had identical structural properties. Therefore, a single numerical model was generated for the MRFs.

Beam elements were chosen to represent the MS1-SSI and MS1-SSI frames’ structural members. The calibration technique was as described in Chapter 4 for Test-1 and Test-2. The targeted fixed-base fundamental period was 0.47 s for both structures, and the numerical model achieved a fixed-base fundamental period of 0.48 s.
5.4 Comparison of MS1-1 for Test-1 and MS1-SSI Test-3

To illustrate the significant differences between the 3-story (prototype) MRF of Test-1 (non-resonant case) and Test-3 (resonant case), this section provides an overview of the acceleration responses for the free-field and the solitary frames: MS1-1 for Test-1 and MS1-SSI for Test-3. The structural details are outlined below. Table 5-1 summarizes the key differences between the frame configurations.

As outlined in Chapter 4, three physical model configurations were used in Test-1. The fixed-base natural periods for each were $T_{N,1} = 0.9$ s, $T_{N,2} = 0.82$ s, $T_{N,3} = 0.76$ s. The fuses used for Configurations 1 and 2 were identical; they exhibited the softest response, with permanent deformations visible post-spin. Configuration 3 had significantly stiffer fuses, which did not exhibit obvious physical yielding. A single structural configuration was used in Test-3. The fixed-base natural period for the north, single story (model) moment-resisting frame structure was $T_{N,4} = 0.47$ s.

Table 5-1: Summary of structural configurations (Test-1 and Test-3)

<table>
<thead>
<tr>
<th>Spin</th>
<th>Configuration</th>
<th>Fixed- base period (sec)</th>
<th>Column height, prototype (m)</th>
<th>Centered Mass, prototype (kg)</th>
<th>Beam span, prototype (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.9</td>
<td>12.76</td>
<td>683,800</td>
<td>9.955</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.82</td>
<td>12.76</td>
<td>532,400</td>
<td>9.955</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.76</td>
<td>12.76</td>
<td>532,400</td>
<td>9.955</td>
</tr>
<tr>
<td>3</td>
<td>--</td>
<td>0.47</td>
<td>9.955</td>
<td>291,160</td>
<td>9.955</td>
</tr>
</tbody>
</table>
5.4.1 Test-1 versus Test-3: Horizontal Accelerations in Free-field and of MS1-1 (Test-1) and MS1-SSI (Test-3)

First, horizontal acceleration-time histories and S_a response spectra are shown for 5 selected motions (Figure 5-2 through Figure 5-11). These motions are summarized in Table 5-2. Please note that a single Joshua Tree record was included from each test; JOS_L1 from Test-1, and JOS_L4 Test-3.

Table 5-2: Summary of examined ground motions (Test-1 and Test-3)

<table>
<thead>
<tr>
<th>Motion ID, Test-1</th>
<th>Achieved CGM PGA, g (Intensity)</th>
<th>Motion ID, Test-3</th>
<th>Achieved CGM PGA, g (Intensity)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MS1-1: Configuration 1</strong></td>
<td></td>
<td><strong>MS1N: Configuration 4</strong></td>
<td></td>
</tr>
<tr>
<td>JOS_L1</td>
<td>0.14</td>
<td>JOS_L4</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>MS1-1: Configuration 2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCS_H</td>
<td>0.59</td>
<td>SCS_H1</td>
<td>0.61</td>
</tr>
<tr>
<td>WPI_L</td>
<td>0.40</td>
<td>WPI_L</td>
<td>0.40</td>
</tr>
<tr>
<td>JOS_H</td>
<td>0.47</td>
<td>JOS_H</td>
<td>0.46</td>
</tr>
<tr>
<td><strong>MS1-1: Configuration 3</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TCU_H</td>
<td>0.45</td>
<td>TCU_H</td>
<td>0.47</td>
</tr>
</tbody>
</table>

The horizontal acceleration time histories highlight several key aspects of the centrifuge tests (Figure 5-2 through Figure 5-11). The base input motions did not vary between experiments, which confirmed the consistency and repeatability of the tests. Significantly, the free field responses from both tests were almost identical for all ground motions. This further confirms the results of the shear wave velocity sensitivity analysis (described in Section 5.5.1.1) which indicated that the soil profile was not the source of the differences in dynamic response between the tests. Plots of the pseudo-acceleration responses of the footings, however, show the beginning of the divergence of data between the two experiments. The dynamic response at the roof of the MRF structure shows the fully developed amplification.
Figure 5-2: Horizontal acceleration time histories of JOS_L for Test-1 (black) and Test-3 (blue) at base, free-field, and roof locations.
Figure 5-3: Joshua Tree, low intensity (JOS_L), comparison of pseudo-acceleration time history response spectra at free-field, foundation, and roof locations.
Northridge, 1994 (Joshua Tree, high intensity)
Horizontal acceleration time histories

<table>
<thead>
<tr>
<th>Test</th>
<th>Type of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>MS1-1 Configuration 2</td>
<td>MS1-SSI</td>
<td>Ordinary</td>
</tr>
</tbody>
</table>

Figure 5–4: Horizontal acceleration time histories of JOS_H for Test-1 (black) and Test-3 (blue) at base, free-field, and roof locations.
Figure 5-5: Joshua Tree, high intensity (JOS_H), comparison of pseudo-acceleration time history response spectra at free-field, foundation, and roof locations.
Figure 5-6: West Pico, low intensity (WPI_L), horizontal acceleration time histories for Test-1 (black) and Test-3 (blue) at base, free-field, and roof locations.
Figure 5-7: West Pico, low intensity (WPI_L), comparison of pseudo-acceleration time history response spectra at free-field, foundation, and roof locations.
Chi-Chi, 1999 (TCU recording, high intensity)

<table>
<thead>
<tr>
<th>Test</th>
<th>Type of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>MS1-1</td>
<td>MS1-SSI</td>
<td>Ordinary</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Test-1</th>
<th>Test-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof MRF</td>
<td><img src="image1.png" alt="Graph" /></td>
<td><img src="image2.png" alt="Graph" /></td>
</tr>
<tr>
<td>Free-field</td>
<td><img src="image3.png" alt="Graph" /></td>
<td><img src="image4.png" alt="Graph" /></td>
</tr>
<tr>
<td>Base Input</td>
<td><img src="image5.png" alt="Graph" /></td>
<td><img src="image6.png" alt="Graph" /></td>
</tr>
</tbody>
</table>

Figure 5-8: Horizontal acceleration time histories of TCU_H for Test-1 (black) and Test-3 (blue) at base, free-field, and roof locations.
Figure 5-9: Chi Chi, high intensity, (TCU_H), comparison of pseudo-acceleration time history response spectra for free-field, foundation, and roof locations.
Figure 5-10: Sylmar Converter Station, high intensity (SCS_H), horizontal acceleration time histories for Test-1 (black) and Test-3 (blue) at base, free-field, and roof locations.
Figure 5-11: Sylmar Converter Station, high intensity (SCS_H), comparison of pseudo-acceleration time history response spectra for free-field, foundation, and roof locations.
5.4.2 Test-1 versus Test-3: Accelerations of Footings

The vertical and horizontal acceleration-time histories of the MRF spread footings from the centrifuge are similarly enlightening. In the first two experiments, the vertical accelerations of the footings of MS1-1 are negligible; for most of the motions, there is essentially no movement in the vertical direction. In stark contrast, vertical accelerations recorded at the MRF (MS1-SSI) footings in Test-3 were consistently triple or quadruple the horizontal accelerations (Figure 5-12 through Figure 5-15). This was not due to differences in the centrifuge itself (e.g., the arm, the shakers, etc.). As illustrated in Figure 5-12 and Figure 5-13, the vertical accelerations measured at the top of the centrifuge box (i.e., ring) at both the north and south boundaries were identical from test to test. These findings have several ramifications regarding the accuracy of the responses modeled in FLAC.

The first is the interdependence of “horizontal” (i.e., x-direction) and “vertical” (i.e., y-direction) acceleration time histories in the centrifuge. These are orthogonal by definition, and were recorded by appropriately oriented ICP or MEMS accelerometers. Due to CGM instrumentation limitations (i.e., the number of available DAQ channels), vertical accelerometers were not included at every horizontally instrumented location – horizontal accelerations were deemed a higher priority, and thus more were used during testing. Additionally, due to geometric limitations in the confined centrifuge testing environment, any “corresponding” vertical and horizontal measurements were separated by some distance. For example, horizontal accelerometers were glued to the edges of footings, while vertical accelerometers were secured to the center of a footing, and laterally offset.

During testing, when the footings separated from the soil surface during rocking induced by inertial interaction, the “vertical” direction was no longer perpendicular to the soil surface. Instead, the vector normal to the rocking footing (previously oriented in “vertical” direction) now contained both x- and y-components. By default, during this time the “horizontal” accelerometers must also have had x- and y-components, since they are always orthogonal to the “vertical” accelerometers. However, since the instruments were not placed directly adjacent to each other during testing, it is not possible to accurately back out the components of each accelerometer. Therefore, the “vertical” and “horizontal” acceleration data cannot be treated as purely x- or y-values.

Additionally, the vertical accelerometers exhibited pounding behavior for almost all of the motions. The majority of the footings had at least one vertical accelerometer go out of range during a giving motion due to this phenomenon. The sensors would become operational after a period of time. However, interference between the sensors almost certainly occurred due to feedback from the out of range accelerometers.
<table>
<thead>
<tr>
<th>Test</th>
<th>Vertical Acceleration of Footings (g)</th>
<th>Horizontal Acceleration of Footings (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test-1</td>
<td><img src="image1.png" alt="Graph" /></td>
<td><img src="image2.png" alt="Graph" /></td>
</tr>
<tr>
<td>Test-3</td>
<td><img src="image3.png" alt="Graph" /></td>
<td><img src="image4.png" alt="Graph" /></td>
</tr>
</tbody>
</table>

Figure 5-12: Test-1 (PTS) versus Test-3 (PTS) acceleration time histories (Footing 1 [black], Footing 2 [gray]). For vertical acceleration of ring (bottom plot), red is Test-1, black is Test-3.
<table>
<thead>
<tr>
<th>Test</th>
<th>Vertical Acceleration of Footings (g)</th>
<th>Horizontal Acceleration of Footings (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
</tr>
<tr>
<td>Test 3</td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
</tr>
</tbody>
</table>

Figure 5-13: Test-1 (JOS_L2) versus Test-3 (JOS_L4) acceleration time histories (Footing 1 [black], Footing 2 [gray]). For vertical acceleration of ring (bottom plot), red is Test-1, black is Test-3.
<table>
<thead>
<tr>
<th>Test</th>
<th>Vertical Acceleration of Footings (g)</th>
<th>Horizontal Acceleration of Footings (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test -1</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
</tr>
<tr>
<td>Test -3</td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
</tr>
</tbody>
</table>

Figure 5-14: Test-1 (LCN) versus Test-3 (LCN) footing acceleration time history comparison (Footing 1 [black], Footing 2 [gray])
<table>
<thead>
<tr>
<th>Test</th>
<th>Vertical Acceleration of Footings (g)</th>
<th>Horizontal Acceleration of Footings (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test -1</td>
<td><img src="image1.png" alt="Graph" /></td>
<td><img src="image2.png" alt="Graph" /></td>
</tr>
<tr>
<td>Test -3</td>
<td><img src="image3.png" alt="Graph" /></td>
<td><img src="image4.png" alt="Graph" /></td>
</tr>
</tbody>
</table>

Figure 5-15: Test-1 (SCS_H1) versus Test-3 (SCS_H) footing acceleration time history comparison (Footing 1 [black], Footing 2 [gray]).
5.5 Numerical Analysis Results of Test-3

The Test-3 analyses consisted of two distinct numerical meshes. The “solitary” MS1-SSI frame was examined with a FLAC mesh of 59 by 30 zones. The “adjacent” MS1-SSSI frame was analyzed with the MS2F_M frame in a 101 by 69 mesh. The results from these analyses are presented in the following sections.

5.5.1 Solitary (SSI) MS1-SSI

To begin, the single MS1-SSI structure was examined. This relatively simple soil-structure system consisting of a single MRF was expected to exhibit conventional SSI responses, and thus establish a baseline structural dynamic response for comparison with the anticipated SSSI between the two adjacent structures. Since the Nevada sand profile was consistent with the previous tests (i.e., dry, $D_r = 80\%$), no changes were expected to be present in the Test-3 soil profile. Therefore, the UBCHYST constitutive model in FLAC for Test-3 was unchanged from previous analyses; the centrifuge soil profile was modeled with the calibrated layers from Test-1 and Test-2 work. The first six ground motions were applied to the FLAC mesh and responses compared with the centrifuge data. Immediately, a significant difference between the recorded data and the predicted response was observed.

The maximum amplitudes of pseudo-acceleration responses from the numerical model MS1-SSI footings and roof were approximately 50% lower than those recorded during the centrifuge testing sequence. This surprising result triggered an examination of its possible cause through careful review of the assumptions and numerical modeling techniques used in the analyses for Test-3, as well as Test-1 and Test-2.

The low intensity Joshua Tree motion (JOS_L) provides one of the clearest examples of the differences between the centrifuge Test-1 and Test-3 and the numerical model data. A comparison of the dynamic response induced by this recording in Test-1 and Test-3 is presented in Figure 5-16. The flexible base system period ($T_{SSI}$) of the frame shifted from approximately 1.05 s in Test-1 to 0.61 s in Test-3, as expected based on the structural properties of the frame in each spin. The FLAC model accurately captured this shift.

The amplitude of the response observed in the centrifuge during Test-3, however, was not matched by the numerical output. The frame achieved $S_a = 5$ g in Test-3 during the low intensity motion. In contrast, the responses of the MS1-1 and MS1-2 frames to any application of the JOS_L motion in Test-1 or Test-2 never exceeded $S_a = 1.3$ g, meaning that the new MRF design caused a five-fold amplification in the frame’s maximum response. The numerical model predicted a doubling of the amplitude from Test-1 to Test-3 (e.g., $S_a = 2.3$ g in Test-3 from $S_a = 1.18$ g in Test-1), a considerable underprediction of the dynamic response.

Therefore, a sensitivity study was conducted to ascertain the probable cause of the differences between the FLAC and centrifuge models; this is recorded in Section 5.5.1.1 through Section 5.5.1.4. First, the soil’s $V_s$ profile and interface elements were examined. Then, the fixed-base fundamental period of the structure was inspected. Finally, the constitutive model parameters were considered. Details of each of these are included and recommendations presented in Section 5.5.2.
5.5.1.1 Shear Wave Velocity Profile

Some variation in the soil profile from centrifuge test to test is unavoidable. Physical conditions are different during the pluviation stage of each test, from the rate of sand deposition to airflow drafts in the laboratory. As mentioned in Chapter 3, Nevada sand is a naturally mined sand and thus varies both between and within individual batches. Test-3 utilized the last of the batch of sand; there may have been differences in gradation between Test-1, Test-2, and Test-3. Additionally, as shown in the Bender element data (Chapter 3) the soil profile always experienced progressive densification during shaking sequences. The order of the ground motion sequence used for Test-3 diverged from the schedule used in Test-1 and Test-2.

While these variations were assumed to have a minimal effect on the tests, a sensitivity analysis was conducted to ensure that variation in the shear wave velocity profile was not the cause of the difference between the numerical and physical data. Fifteen $V_s$ profiles were examined in FLAC for two representative ground motions, a near-fault, low-intensity motion (Parachute Test Site, [PTS_L]) and an ordinary, low-intensity motion (Joshua Tree, [JOS_L]). By systematically changing the $V_s$ profile within FLAC, the sensitivity of the soil-structure system to that parameter was examined. The following aspects of the shear wave velocity profile were modified:

1) The slope of the $V_s$ profile
2) The initial and final shear moduli of the profile (specified surface and 29.5 m depth estimates)
3) The overall period of the site (ranged from 0.3 s to 0.7 s).

Three of the fifteen examined shear wave velocity profiles are presented below, in Figure 5-18(a), to illustrate the range of profiles included. The center $V_s$ profile, (i.e., estimate 3, red trace) is equal to that used in Test-1 and Test-2 analyses. The footing responses from the MRF in FLAC are presented in Figure 5-18(b) for each $V_s$ profile, as well as the centrifuge MRF footing data (black trace).

These analyses showed that while the amplitude of either the high-frequency or low-frequency responses could be increased through manipulation of the shear wave velocity profile to match the recorded HBM04 footing responses, both high- and low-frequencies could not be improved simultaneously (Figure 5-18(b)). The optimal spectral shapes and acceleration-time histories remained those obtained from the original shear wave velocity profile estimate.

![Figure 5-17: Location of footing response ($S_a$) in FLAC for three story (prototype) MRF](image)

Figure 5-17: Location of footing response ($S_a$) in FLAC for three story (prototype) MRF
5.5.1.2 Interfaces

The connections between the MS1-SSI spread footings and the numerical mesh were also examined. Numerical sensitivity analyses of the MRF spread footings in Test-1 and Test-2 indicated that interfaces were not necessary to accurately capture the dynamic response of the structures for those configurations. Therefore, initial modeling of the Test-3 buildings was conducted with an “attached” foundation condition, consistent with the previous numerical work.

However, the SSI and SSSI responses during the first two centrifuge tests were almost exclusively kinematic. In contrast, significant inertial interaction, including foundation rocking, was observed during Test-3, both in high-speed camera footage and in post-processing of vertical accelerometer data. This meant that separation of the spread footings from the soil surface and sliding occurred during Test-3. Hence, these relative soil-foundation movements could not be captured with a rigidly attached foundation condition. Therefore, it was postulated that the inertial SSSI associated with Test-3 was not captured adequately in FLAC due to an unrealistic foundation interface condition. Interfaces elements were added to the bases and lateral sides of the MS1-SSI footings to see whether the dynamic response could be improved by their inclusion.

Interface elements in FLAC are composed of a series of vertical (normal) and lateral (shear) springs. Interfaces are specified along adjacent sides of structural elements and the numerical grid, or simply between two meshes (i.e., grid to grid). Each grid point or node on one face (e.g., Side A) is matched to the nearest contacting grid point or node on the opposing face (e.g., Side B), as shown in Figure 5-19. As an example, if node \( N \) is considered to be the active node for formulation in Figure 5-19, the contact length for that node, \( L_N \), is one-half the distance to Node \( M \) and one half the distance to Node \( P \) (the two nearest nodes in either direction). Each interface’s normal and shear springs are assigned stiffness values (\( k_s \) and \( k_n \), respectively) when
interfaces are specified and the interfaces themselves are governed by either “Coulomb sliding and/or tensile separation” (Itasca, 2005).

It is recommended that interface stiffnesses be within one order of magnitude of the stiffest neighboring zone; if the stiffness values differ dramatically, convergence will be significantly slowed. An initial estimate of interface stiffnesses may calculated by normalizing the bulk modulus, $K$, and the shear modulus, $G$, of the adjacent soil constitutive model by the “smallest width of an adjoining zone in the normal direction”, $\Delta z_{\text{min}}$ (Itasca, 2007). Itasca provides the following equation to estimate a value of $k_n$ and $k_s$:

**Equation 5-7**

$$k_{s,n} = \max \left[ \frac{K + \frac{4G}{3}}{\Delta z_{\text{min}}} \right]$$

The normal and shear stiffnesses were first estimated using Equation 5-7. Based on a shear modulus at the surface of 22.6 MPa interface friction was set equal to 40 degrees. This configuration was examined for the JOS_L4 input motion.

![Figure 5-19: Interface formulation between two numerical meshes, Side A and Side B (Itasca 2005)](image)

The observed response for this configuration was significantly different than the rigidly attached condition; the change was toward a less realistic response. While the amplitude of spectral response remained unchanged, the peak response shifted 0.2 seconds to the right, indicating period lengthening. This is reasonable, as forcing the connection at the soil-structure horizon to be softer would necessarily create a more flexible condition and thus a longer $T_{\text{SSI}}$. An example of the response in shown below in Figure 5-20.
Figure 5-20: Joshua Tree, low-intensity (JOS_L4), MS1-SSI (solitary) with interfaces

5.5.1.3 Fixed Base Period of Structure ($T_{FB}$)

The estimate of fundamental period of MS1-SSI was another component of the system examined in the sensitivity study of the Test-3 numerical model. A collection of MRF models were generated with fixed-base fundamental periods ranging from 0.24 s to 0.63 s to bracket the originally estimated $T_{FB} = 0.47$ s. These frames were subjected to two motions, one was the high-intensity near-fault motion Lucerne (LCN) that occurred in the middle of the shaking sequence, and the other was a low-intensity ordinary record Joshua Tree (JOS_L) that was applied near the beginning of the sequence. The results from the Lucerne record are presented in Figure 5-21.

Shifting the $T_{FB}$ of the structure did alter the dynamic response of the system, as expected. Shorter fixed-base fundamental periods in the 0.2 s to 0.35 s range shifted the pseudo-acceleration response spectra to higher frequencies, yielding a spectral shape more closely aligned with the centrifuge data. However, the amplitudes were forced even lower than the original $T_{FB} = 0.47$ s estimates. Longer MS1-SSI fixed-base fundamental periods (ranging from approximately 0.5 s to 0.63 s) amplified the $S_a$ response slightly, but shifted the maximum structural response to longer period regions, which was inconsistent with the laboratory data. While it remained to be determined whether the total system’s soil-structure ($T_{SSI}$) period was being accurately captured by the numerical model, these results indicated that the building’s $T_{FB}$ alone was not responsible for the deamplified acceleration response.
5.5.1.4 Soil Profile Parameters

The last stage of the numerical sensitivity analysis was examination of the constitutive model parameters. It is widely recognized that as shear strain in a soil increases, hysteretic damping values estimated with the Masing criteria tend to overpredict the expected damping (when compared laboratory data) (Stewart and Kwok, 2008). Since Test-3 induced significant inertial interaction, the shear strains developed in the soil profile were likely considerably higher than in Test-1 and Test-2. The selection of constitutive model parameters during the free-field calibration stage was weighted toward capture of the modulus degradation curves with depth. While the damping curves were monitored during the calibration process, the best fit achieved with the UBCHYST model showed considerable overestimation of damping for shear strain values greater than approximately 0.1%. For the free-field regions, this was not significant, nor was it a factor for Test-1 or Test-2. For the structures in Test-3, however, this could have contributed to the lowered dynamic response in the numerical model.

Therefore, the shear modulus reduction and damping curves were revisited with new calibration goals. Instead of matching the Darendeli et al. (2001) curves with depth, the shear modulus values were held constant from previous work and the modification parameters in UBCHYST were shifted. The UBCHYST parameters have a recommended range over which they can vary (Byrne, pers. comm.). During the calibration phase of this work (see Chapter 3), the UBCHYST model was exercised over each parameter’s range to give the researcher familiarity with the behavior of the constitutive model. Selected results from that are shown below in Figure 5-22 and Figure 5-23.
Figure 5-22: Modulus reduction curve, at elevation 16.4m (parameter h_n range: 1.0 to 3.0)

Figure 5-23: Damping curve, at elevation 16.4m (parameter h_n range: from 1.0 to 3.0)
Each factor that contributed to the stiffening of the shear modulus reduction path and the lowering of the damping curve with increasing shear strain was shifted to its maximum value (within the allowable range). The two parameters found to have the greatest effect on this were $h_{rf}$ and $h_{n1}$. These were set to 0.7 and 3 respectively, which were the maximum values allowed for these parameters. At shallow depths (i.e., less than 4.9 m), the responses were within one standard deviation of the average response curve. At depths from 4.9 m to 10.1 m, the UBCHYST responses were stiffer than the 84% prediction from Darendeli (2001). Two examples of this are shown below (Figure 5-24 and Figure 5-25).

**Figure 5-24:** Modulus reduction curve for stiffened and original soil parameters (0.9 m depth) plotted against Darendeli (2001) mean and plus one standard deviation (e.g., 84%) curves.
The constitutive model directly below the mat foundation and spread footings was assigned the $h_{rf}$ and $h_{n1}$ ‘stiffened’ parameters. Two shear wave velocity profiles were examined for this configuration. One was equal to that used in Test-1 and Test-2 (Profile C). The other was slightly softer, 80% of the shear wave velocity profile (Profile D), which had shown some improvement in select ground motions through amplification.

This change led to a significantly improved MS1-SSI dynamic response. The pseudo-spectral responses and acceleration time histories recorded at the roof of the MRF are presented for four ground motions: Joshua Tree (low intensity, Figure 5-26), Lucerne (Figure 5-27), Sylmar Converter Station (low intensity, Figure 5-28), and Parachute Test Site (Figure 5-29), are presented below. The dynamic responses obtained with the original parameters are included for the motions for reference.
Figure 5-26: Joshua Tree (JOS_L4), low intensity, comparison of soil profiles and stiffnesses, MS1-SSI (Profile C [red] and Profile D [blue]).
Figure 5-27: Lucerne (LCN) motion, high intensity, comparison of soil profiles and stiffnesses, MS1-SSI (Profile C [red] and Profile D [blue])
Figure 5-28: Sylmar Converter Station (SCS_L1), low intensity, comparison of soil profiles and stiﬀnesses, MS1-SSI (Profile C [red] and Profile D [blue]).
Figure 5-29: Parachute Test Site (PTS), high intensity, comparison of soil profiles and stiffnesses, MS1-SSI (Profile C [red] and Profile D [blue]).
5.5.2 Test-3: Resonance

In summary, the resonant condition achieved in the physical model presented a challenge in the numerical modeling realm. The four key components of the numerical model investigated in Section 5.5.1.1 through 5.5.1.4 indicated that the constitutive model was likely contributing to the deamplification through over-damping. This was addressed by adjusting the shear modulus and damping curves within the UBCHYST constitutive model for the region directly beneath the structures. However, this did not completely eradicate the differences – the centrifuge acceleration time histories and pseudo-acceleration response spectra continued to show values considerably higher than those estimated from the numerical model. This was likely due to the difficulty of capturing the resonant condition in the numerical model.

Resonance is exceedingly sensitive to slight changes in fundamental period. Shifting the first mode period of a system just one tenth of a second higher or lower than the resonant value can lower the achieved amplitude by an order of magnitude. The variables in centrifuge testing, including cumulative damage to the structures due to consecutive ground motions, progressive soil profile densification during an experiment, and the lack of precise shear wave velocity data for each motion in the Test-3 suite introduced enough variables to prevent the capture of the resonant condition.

Due to limits on the number of available instruments, every aspect of a centrifuge test cannot be recorded. For non-resonant cases (e.g., Test-1 and Test-2) this is surmountable in FLAC. However, for the extremely sensitive cases of resonance (e.g., Test-3), this is a significant challenge. The sensitivity of the numerical model to this is most clearly illustrated by the Joshua Tree (low intensity) motion (Figure 5-16). This motion had a mean period, $T_m$, of 0.66 s. As a result, almost perfect resonance of the system was achieved: $T_m \sim T_{soil} \sim T_{SSI} \sim 0.6$ s. This event yielded the most dramatic amplification recorded in the entire testing sequence and was consistently underpredicted by the numerical model.

5.5.3 SSSI: Adjacent MS2F_M and MS1-SSI

After careful examination of the solitary MS1-SSI frame, the adjacent (SSSI) condition was considered. Building on the analysis completed for the solitary 3-story (prototype) structure, the stiffened constitutive model was used under both the MS1-SSI structure footings and the MS2F_M mat foundation.

One aspect of the numerical model was revisited for this configuration. The MS2F_M elastic rocking wall was initially modeled as an “attached” condition, consistent with the approach deemed optimal for the MRF MS1-SSI. However, the results from the centrifuge indicated that the MS2F_M rocked significantly during the centrifuge test. Therefore, the MS2F_M mat was connected to the mesh with shear ($k_s$) and normal stiffnesses ($k_n$) equal to 0.5e7 Pa.

Results from these analyses are shown in Figure 5-30 through Figure 5-41. Centrifuge data recorded on the roof of the MS2F_M structure is plotted against the FLAC computed results for the roof of the numerical model. The base input motions for the physical (CGM) and numerical (FLAC) model as well as the centrifuge data recorded at the mid-level of the structure are included for reference. The FLAC computed acceleration response of the MS2F_M structure was consistently below that recorded in the centrifuge.
5.5.4 SSSI: MS2F_M Response

To begin, the responses of the MS2F_M shear wall structure are presented. Consistent with the manner in which results were presented in Chapter 4, the four types of motions are presented in four subsections. The suite of motions are first sorted by type (near-fault or ordinary events), then by intensity level (e.g., high or low).

5.5.4.1 Test-3, MS2F_M: Ordinary Low Intensity Motions

The responses to the ordinary, low-intensity motions of Test-3 were examined first. This category included five applications of the Joshua Tree (low-intensity) motion (JOS_L1 through JOS_L5), the Parachute Test Site (PTS) event, and two applications of the Chi Chi (TCU_L1 and TCU_L2) recordings.

As previously mentioned, the amplitudes of the dynamic responses of MS2F_M from the centrifuge were not achieved in the FLAC model for any motions. However, the spectral shape and time-domain responses for all Joshua Tree (low intensity) events were well captured by the FLAC model, as illustrated by the representative JOS_L5 (Figure 5-30). Similarly, the TCU_L motions’ frequency content was well captured, as was the acceleration response in the time domain (Figure 5-31).

The shape of the pseudo-acceleration response spectra from FLAC for the PTS motion, however, did not correspond with that of the laboratory. While the spectral shape for the period range of 0.5 s and greater predicted by FLAC was consistent with the centrifuge output, the numerical data exhibited frequency content in the shorter period range (approximately 0.2 s to 0.4 s) that was not observed in the centrifuge (Figure 5-32). This is consistent with previous analyses of the PTS motion for the 3-story (prototype) and 9-story (prototype) MRFs which found it to be extremely sensitive to small changes in the numerical model. The dynamic response of the model to the PTS record could be improved by unique tuning of the soil and structure for that motion. This was not undertaken for the MS2F_M structure in this research, as the robustness and universality of the developed numerical model were of interest (as explained in Chapter 4). Since the difficulty in capturing the response of this motion is consistent with previous work, the challenge was attributed to the motion and not the model.
<table>
<thead>
<tr>
<th>Description</th>
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<tbody>
<tr>
<td><strong>CGM</strong></td>
<td>Roof level (black)</td>
</tr>
<tr>
<td><strong>FLAC</strong></td>
<td>Roof level (pink)</td>
</tr>
</tbody>
</table>

Figure 5-30: Joshua Tree, low intensity (JOS_L5), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
Figure 5-31: Chi Chi, low intensity (TCU_L2), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
Figure 5-32: Parachute Test Site, low intensity (PTS), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
5.5.4.2 Test-3, MS2F_M: Ordinary High Intensity Motions

Second, ordinary high-intensity motions were examined. These included two only – the Joshua Tree (JOS_H) and the Chi Chi (TCU_H) motions. The JOS_H motion captured the overall spectral response, displaying amplification in the pseudo-acceleration response of MS2F_M at 0.58 s and 0.95 s, as in the centrifuge data (Figure 5-33).

Similarly, the numerical model output matched the recorded response of the MS2F_M structure to the TCU_H motion (Figure 5-34). The spectral shape was consistent with that of the low intensity version of the motion, with a pronounced amplification at approximately 0.5 s. Examination of the acceleration time history shows that the centrifuge model exhibited significantly larger amplitudes than those induced by TCU_H in the numerical model, but the records’ arrival times and seismic wave patterns are captured.
<table>
<thead>
<tr>
<th>MS2F_M Horizontal Acceleration at Roof: Parachute Test Site (JOS_H)</th>
<th>Description</th>
</tr>
</thead>
</table>
| ![Graph](image) | \( \lambda = 5\% \)  
- CGM, MS2 Roof  
- CGM, MS2 Mid-level  
- Base, CGM  
- MS2F_M, FLAC  
- Base, FLAC |

Figure 5-33: Joshua Tree, high intensity (JOS_H), MS2F_M with \( \beta = 0.5\% \) structural Rayleigh damping.
Figure 5-34: Chi Chi, high intensity (TCU_H), MS2F_M with β = 0.5% structural Rayleigh damping.
5.5.4.3 Test-3, MS2F_M: Near-Fault Low Intensity Motions

The next group of motions examined were the near-fault, low intensity events. These included two applications of the Sylmar Converter Station motion (SCS_L1 and SCS_L2), the West Valley College (WVC_L) recording, and the West Pico (WPI_L) motion. The numerical model captured the SCS_L motions well, as evidenced by both the pseudo-acceleration response spectra and the acceleration time history for SCS_L1 (Figure 5-34). The dynamic responses induced by the WPI_L and WVC_L motions were equally good fits to the centrifuge data (Figure 5-36 and Figure 5-37). Consistent with all results from the MS2F_M structure, the amplitudes of the responses were smaller than recorded. However, the frequency content of each motion closely mirrored that recorded in the centrifuge. While the overall amplitude may not match, the trends and patterns of response from the laboratory were captured by the numerical model for this suite.
Figure 5-35: Sylmar Converter Station, low intensity (SCS_L1), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
Figure 5-36: West Valley College, low intensity (WVC_L), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
Figure 5-37: West Pico, low intensity (WPI_L), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
5.5.4.4 Test-3, MS2F_M: Near-Fault High Intensity Motions

The last group of motions were the near-fault, high intensity quakes. With short duration, intense records that often induce nonlinearity in both soil and structural systems. These motions are arguably the most challenging type of motion to model due to the significant nonlinearity in the soil and structural responses to high intensity motions. In Test-3, the motions included in this suite were Rinaldi Recording Station (RRS), Lucerne (LCN), Port Island, modified (PRImod), and two Sylmar Converter Station events (SCS_H1 and SCS_H2).

The RRS motion was well-captured (Figure 5-38). The frequency content was accurately described by the numerical model predictions, and the amplitudes of the response were also close. Examination of the acceleration time histories from the MS2F_M structure shows some period shifting in the numerical model output after approximately 30 s; however, that is after strong shaking has ceased, and has little effect on the overall response to dynamic input in the range of engineering interest. The fit for this motion is one of the best in the MS2F_M suite.

The LCN record was also well-captured, as shown in the pseudo acceleration response spectra (Figure 5-39). However, the first significant pulse (19 s in acceleration time history) was underpredicted in the time domain. The analysis results for the PRImod and SCS_H1 events were consistent with previous findings. The spectral shape of the MS2F_M response was captured by the numerical model, but the overall amplitude of the response was smaller than that recorded in the centrifuge (Figure 5-40 and Figure 5-41, respectively).
<table>
<thead>
<tr>
<th>Description</th>
<th>Figure 5-38: Rinaldi Recording Station (RRS), MS2F_M with $\beta = 0.5%$ structural Rayleigh damping.</th>
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<td>CGM, MS2 Mid-level</td>
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<td>MS2F, FLAC</td>
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<td>Base, FLAC</td>
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**MS2F_M Horizontal Acceleration at Roof Rinaldi Recording Station (RRS)**
### MS2F_M Horizontal Acceleration at Roof
Lucerne (LCN)

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<tr>
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<th>$S_a$ (g)</th>
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</tr>
<tr>
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<td>3.5</td>
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</table>

- **CGM, MS2 Roof** (black)
- **CGM, Ms2 Mid-level**
- **Base, CGM**
- **MS2F, FLAC** (pink)
- **Base, FLAC** (dotted pink)

**Figure 5-39**: Lucerne (LCN), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.

**Description**

- **CGM**
  - Roof level (black)
- **FLAC**
  - Roof level (pink)
Figure 5-40: Port Island Modified (PRImod), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
Figure 5-41: Sylmar Converter Station (SCS_H1), MS2F_M with $\beta = 0.5\%$ structural Rayleigh damping.
5.5.5 SSSI: Adjacent MS1-SSSI Response

The difference between the MS1-SSSI (adjacent, SSSI) and MS1-SSI (solitary, SSI) structures is somewhat subtle, but still clearly visible in the centrifuge data. For almost all motions, the response of the single structure (SSI) was consistently lower than that observed in the SSSI condition. Also, the dynamic response induced by many motions exhibited a change in spectral shape from the SSI to the SSSI case (e.g., period lengthening and amplification). MS2F_M clearly affects the response of the adjacent structure.

As before, the results are presented in four sections. Near fault and ordinary ground motions are examined separately. Within each of these, the events are divided by intensity level (e.g., high or low). The dynamic responses for the SSI case are plotted as well, for reference.

5.5.5.1 SSSI: Adjacent MS1-SSSI Response, Ordinary Low Intensity

Data from the suite of ordinary, low-intensity motions is shown below, including one of the five Joshua Tree motions (JOS_L5), the Parachute Test Site (PTS) event, and one of the two Chi Chi recordings (TCU_L2). Unlike Test-1 and Test-2, however, these are neither the most linear nor the least damaging events. Instead, the Joshua Tree motion achieved complete resonance with the soil-structure system, leading to some of the highest accelerations and spectral responses recorded during Test-3. Unsurprisingly, the results from the centrifuge and the numerical model show some differences in dynamic response for these motions.

Examining the results for the JOS_L5 motion summarizes several key findings from the Test-3 analyses (Figure 5-42). First, the resonant condition is difficult to capture without highly detailed information regarding damage level, soil and structural properties, and system monitoring. Since this is a triple resonant condition ($T_m \sim T_{soil} \sim T_{SSI} = 0.6$ s), it is even more difficult to capture the responses of the structural models. Second, the amplitudes of the dynamic responses achieved in the centrifuge are consistently greater than those achieved in the FLAC model. The structures in Test-3 exhibited rocking, sliding, and three-dimensional effects that the plane-strain numerical model could not capture. Third, despite these challenges, the pseudo-acceleration response spectral shapes, acceleration-time histories, and trends (e.g., the more intense motions in the CGM induce more intense reactions in the FLAC model) were successfully modeled.
Figure 5-42: Pseudo-acceleration spectral response, MS1-SSI and MS1-SSSI comparison of SSI and SSSI (JOS_L5)
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<td><strong>FLAC</strong></td>
</tr>
<tr>
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</tr>
<tr>
<td>Solitary structure (green)</td>
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Chi Chi, Low Intensity (TCU_L2)
Horizontal acceleration at roof

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Figure 5-43: Pseudo-acceleration spectral response, MS1-SSI and MS1-SSSI comparison of SSI and SSSI (TCU_L2)
Figure 5-44: Pseudo-acceleration spectral response, MS1_SSI and MS1_SSSI; comparison of SSI and SSSI (PTS)
Adjacent MS1-SSSI Response: Ordinary High Intensity Motions

Next, the SSSI responses to ordinary high-intensity motions were examined. These included two only motions, the Joshua Tree (JOS_H) and Chi Chi (TCU_H) motions. The JOS_H motion exhibited a reversal of the trends observed in the low-intensity ordinary motions. The solitary structure had a larger maximum acceleration response than the adjacent (Figure 5-45). This does not mean that the single structure sustained more damage in that event; indeed, the lower overall amplitude of the SSSI spectral response may indicate that more nonlinearity was induced in the soil and structural system. Also, at longer periods (e.g., 0.85 s to 1.25 s), the adjacent structure exhibited greater pseudo-acceleration responses.

The TCU_H record exhibited a similar trend (Figure 5-46). The adjacent configuration yielded a lower overall spectral response than that of the single MRF in both the numerical and physical models. While the overall spectral response was captured well by the FLAC model, the peak in response visible at approximately 0.6 s was slightly underestimated in both the SSI and SSSI configurations in FLAC. The spike in the pseudo-acceleration response spectra from FLAC centered at 0.74 s seems to indicate some period lengthening in the numerical model that was not present in the centrifuge experiment measurements. However, the overall response estimated by FLAC is close to that observed in the centrifuge experiments, and the trends from SSI to SSSI are captured reasonably well.
Figure 5-45: Pseudo-acceleration spectral response, MS1-SSSI and MS1-SSI comparison of SSI and SSSI (JOS_H)
Figure 5-46: Pseudo-acceleration spectral response, MS1-SSSI and MS1-SSI comparison of SSI and SSSI (TCU_H)
5.5.5.3 *Adjacent MS1-SSSI Response: Near-Fault Low Intensity Motions*

The next group of motions examined were the near-fault, low intensity events. These included two applications of the Sylmar Converter Station motion (SCS_L1 and SCS_L2), the West Valley College (WVC_L) recording, and the West Pico (WPI_L) motion.

The SCS_L2 event in the centrifuge produced similar dynamic responses for the MRFs in both the SSI and SSSI configurations (Figure 5-47). The adjacent structure had a slightly higher maximum pseudo-acceleration response value at approximately 0.6 s, while the spectral shape was almost identical for the two. The FLAC model produced similar trends. The dynamic response was slightly higher at the 0.6 s period for the SSSI mesh when compared with the SSI model. However, the maximum spike in the response centered at 0.74 s clearly visible in the FLAC output was absent in the centrifuge data. This may indicate that there was some period lengthening due to SSSI effects in the numerical simulations that were not observed in the centrifuge test.

Examination of the WVC_L motion showed both similarities and differences between the FLAC and centrifuge outputs (Figure 5-48). The spectral shapes from the centrifuge data and the numerical model output were quite similar for both structural conditions. The two also agreed in the trend of the maximum pseudo-acceleration response observed at 0.9 s; the adjacent (SSSI) response was larger than the solitary (SSI) response in FLAC, as in the centrifuge. In the shorter period range (approximately 0.6 s), the numerical simulations yielded spectral responses greater for the SSI case than that observed in the centrifuge experiments.

The WPI_L motion showed a significant change between the adjacent and single MS1-SSI and MS1-SSSI structures (Figure 5-49). The adjacent condition was considerably lower than the solitary condition. FLAC did not capture this difference in amplitude; both conditions reached roughly equivalent maximum pseudo-acceleration response values. The model did, however, capture the period shift and the spectral shape shift from SSI to SSSI.
Figure 5-47: Pseudo-acceleration spectral response, MS1-SSSI and MS1-SSI comparison of SSI and SSSI (SCS_L2)
Figure 5-48: Pseudo-acceleration spectral response, MS1S_SF80 and MS1-SSI comparison of SSI and SSSI (WVC_L)
**West Pico, low intensity (WPI_L)**  
**Horizontal acceleration at roof**

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<th>Description</th>
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<th>FLAC</th>
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<td>Adjacent structure</td>
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**Figure 5-49:** Pseudo-acceleration spectral response, MS1-SSSI and MS1-SSI comparison of SSI and SSSI (WPI_L)
5.5.5.4 Adjacent MS1-SSSI Response: Near-Fault High Intensity Motions

The last group of motions were the near-fault, high intensity quakes. With short duration, intense records that often induce nonlinearity in both soil and structural systems, these are arguably the most difficult to capture. In Test-3, the motions included in this suite were Rinaldi Recording Station (RRS), Lucerne (LCN), two Sylmar Converter Station events (SCS_H1 and SCS_H2) and Port Island, modified (PRImod).

The RRS motion spectral shape was well captured (Figure 5-50). The two configurations yielded almost identical responses in the geotechnical centrifuge experiments. The solitary (SSI) configuration had a slightly lower maximum spectral response that the adjacent configuration (4.14 g versus 4.29 g). The numerical model did not capture this effect; instead, 4.2 g was recorded in the SSI solitary case and 4.1 g was achieved in the SSSI case.

Contrastingly, the LCN event was well-captured by the FLAC simulations, with spectral shape matched for both configurations (Figure 5-51). The SSI scenario achieved a slightly lower maximum pseudo-acceleration amplitude than the SSSI case, and this was captured by the numerical model.

The SCS H motion exhibited one of the most significant spectral changes in the Test-3 MS1-SSI to MS1-SSSI data (Figure 5-52). The physical experiment’s maximum dynamic response was larger in the 0.6 s to 0.7 s period range for the SSI configuration; this was captured well by the numerical simulations (note the lowered response in the adjacent model at this value). The spike in response centered at approximately 0.9 s was lower for the adjacent configuration in the centrifuge data, but the numerical model calculated a greater change from SSI to SSSI than was observed during testing. While the trend is correct, the absolute value decrease is larger in numerical simulation that observed in the centrifuge experiments.

Lastly, the responses to the PRImod motion was not captured as well as with the other motions (Figure 5-53). The amplification centered at approximately 1 s (the period range from 0.85 s to 2.25 s) was missed by the FLAC model. The single MRF (centrifuge) response had a slightly higher amplitude at 0.65 s than that of the adjacent frame, which was captured by the FLAC model. Also, the spike in spectral response at approximately 1 s was slightly lower for the SSSI configuration in the centrifuge data, an effect which was observable in the numerical simulations.
Figure 5-50: Pseudo-acceleration spectral response, MS1-SSI and MS1_SSI_SF80 comparison of SSI and SSSI (RRS)
Figure 5-51: Pseudo-acceleration spectral response, MS1-SSI and MS1-SSSI comparison of SSI and SSSI (LCN)
Figure 5-52: Pseudo-acceleration spectral response, MS1-SSI and MS1-SSSI comparison of SSI and SSSI (SCS_H)
Port Island, modified (PRImod)
Horizontal acceleration at roof

<table>
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<tr>
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<td>CGM</td>
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<tr>
<td>Solitary structure (black)</td>
</tr>
<tr>
<td>FLAC</td>
</tr>
<tr>
<td>Adjacent structure (blue)</td>
</tr>
<tr>
<td>Solitary structure (green)</td>
</tr>
</tbody>
</table>

Figure 5-53: Pseudo-acceleration spectral response, MS1-SSI and MS1-SSSI comparison of SSI and SSSI (PRImod)
5.6 Summary of SSSI and SSI (HBM04)

The results from centrifuge Test-3 illustrate several of the key issues involved in dynamic SSI and SSSI when the structure and soil are a resonant condition. Structural models studied in this test were designed with a fixed-base fundamental period which corresponded to the fundamental site period ($T_{site}$-$T_{bldg}$). This condition led to a significant inertial component of observed soil-structure interaction for all ground motions applied in the suite. While all motions exhibited a significantly amplified response when compared with past tests, some were extraordinarily sensitive to this condition. In particular, the Joshua Tree (low intensity) motion (JOS_L) caused a dynamic response of more than five times that observed in Test-1 and Test-2 for a similar, single degree of freedom MRF.

The FLAC numerical simulations consistently underestimated the amplitudes of the dynamic responses of the structures for Test-3. A sensitivity analysis of several components of the numerical model was conducted; this included an examination of the shear wave velocity profile, interface elements, fixed-base fundamental period of the frame, and constitutive model parameters. The results from this indicated:

1) The constitutive model exhibited excessive damping at shear strains greater than 0.1%. This caused some deamplification of the dynamic structural response at the roof level. This was addressed by changing two parameters within the constitutive model to achieve a stiffer soil response. These parameters were used under the foundation elements of the structures.

2) The resonant condition was difficult to capture in FLAC. The fundamental periods of vibration of the elastic rocking wall (MS2F_M) and moment-resisting frames (MS1-SSSI and MS1-SSI) were not perfectly captured, and the flexible-base periods likely changed throughout the testing sequence. Similarly, the $V_s$ profile changed from motion to motion during the spin (as indicated by the Bender element data, Chapter 3). These changes were small, but for a resonant condition, significant. The impact of this on the dynamic response estimated from the numerical model illustrates the challenges of capturing resonance.

3) The 3-story (prototype) structures were well captured in both the solitary (MS1-SSI) and adjacent (MS1-SSSI) conditions. Overall, changes in the pseudo-acceleration response spectra were captured from the SSI to SSSI conditions, and trends were matched for most of the motions. The maximum accelerations observed in FLAC were not matched (the numerical model output was consistently below the centrifuge data); this is likely due to the challenges associated with capturing resonance of the system.

Three-dimensional effects were not captured by the plane-strain assumption in the FLAC simulations, necessarily impacting the accuracy of the simulations. Despite the limitations due to the two-dimensional condition, the numerical model was able to capture trends in dynamic response and pseudo-acceleration response spectra for both SSI and SSSI conditions.
6 SSSI in Prototypical Scenarios

6.1 Introduction to Prototypical SSSI

The “Shaking of a City Block” centrifuge testing sequence provides a rich data set with which to calibrate an advanced numerical simulation tool, such as FLAC, which can then be used to investigate the seismic response of several prototype adjacent structures. Careful numerical analysis of dozens of scenarios with varying foundation types, elevations, ground motions, and structural configurations would provide invaluable insights. Through the back-analyses already discussed in this thesis and the familiarity gained through analysis of the centrifuge tests, confidence in the model’s ability to capture dynamic response was established. Therefore, the final step in this research program is to examine the responses of more realistic, prototypical SSSI scenarios.

6.2 Prototype Model Background

Two special moment resisting frames (SMRFs) were selected for use in this analysis from the work of Ganuza (2006). The first structure was a 3-story frame, with a 4-bay by 6-bay footprint (Figure 6-1). This structure was the source of the 3-story (prototype) MRF design examined in the centrifuge experiments. It had a first-mode fundamental period of 1.08 s, and a second-mode fundamental period of 0.39 s. The second structure was a 5-story SMRF, which was founded on a square 5-bay by 5-bay area; the first translational period of that structure was 1.53 s and the second was 0.52 s. No version of this structure was examined in the centrifuge experiments.

Foundations were not specified in the work of Ganuza (2006) for any of these structures. Ganuza was instead focused on the superstructure responses; the buildings were all modeled with rigidly attached (3-story) or simple (5-story) connections. Therefore, for this analysis, a mat foundation was assumed for the structures. As both of these SMRFs were assumed to be founded on competent dense, dry sand deposits, a mat was a reasonable foundation type.

6.3 Prototype Model Construction

The numerical models for this analysis were developed with the same procedure used for the SMRFs in the centrifuge, with one exception, which was the tuning procedure. For the centrifuge models, the targeted fixed base fundamental period of vibration was achieved by varying the density and thicknesses of the structural members. Mass was preserved, but the dimensions of the beams and columns were changed until the targeted first mode of vibration was matched. In contrast, the specific structural members used for each component of the prototypical structures were of interest. The wide flange steel beams were specified for girders and columns (interior and exterior) by floor in Ganuza (2006). The configurations particular to each structure were replicated in the FLAC analyses.

A concrete slab 5.5 in thick was added to each floor level. Lastly, a 20 psf live load was applied to each floor level (Krawinkler, 1999). The total mass of the superstructure was calculated from these assumptions to ensure that the value matched that of the prototypical target (Ganuza, 2006). For the 3-story, the reactive weight was 6,500 kips, while the 5-story was 11,100 kips.
To tune the numerical models, the stiffness of the structures was altered a different way – through optimization of the Young’s modulus (E) for both lateral and vertical members. Since the geometry of the members was kept as specified, the mass distribution remained unchanged from the target structures. The fundamental period of vibration was identified for each value of E with an FFT (as outlined in Chapter 4) until the structure’s fixed-base fundamental period was matched.

![Figure 6-1: (a)The plan view of the 3-story frame, (b) the floor plan of the 5-story and 9-story frames, and (c) the elevation view of each frame (Ganuza, 2006)](image)

6.3.1 Numerical Model: LAM3

For the 3-story structure, five W-beams were used; structural properties are listed in Table 6-1. An inner section of the structure was taken for the plane strain analysis in FLAC (equivalent to bays 2-6 in Figure 6-1(a)). The distributed load for a unit width was computed with MathCAD, as was done for Tests 1 – 3 (see Chapter 4 for a complete description of the procedure). First and second mode fundamental periods of 1.08 s and 0.4 s, respectively, were achieved and used for analysis. A plastic moment was specified for each member based on the plastic modulus ($Z_x$) and a steel yield stress of 50 ksi. The total weight of the superstructure was 6,500 kips.
Table 6-1: Structural member properties for 3-story structure (LAM3)

<table>
<thead>
<tr>
<th>Designation</th>
<th>A (in²)</th>
<th>Iₓₓ (in⁴)</th>
<th>Iᵧᵧ (in⁴)</th>
<th>Zₓ (in⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column (exterior)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W14x257</td>
<td>75.6</td>
<td>3400</td>
<td>1290</td>
<td>487</td>
</tr>
<tr>
<td>Column (interior)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W14x311</td>
<td>91.4</td>
<td>4330</td>
<td>1610</td>
<td>603</td>
</tr>
<tr>
<td>Girders</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W24x62</td>
<td>18.2</td>
<td>1550</td>
<td>34.5</td>
<td>153</td>
</tr>
<tr>
<td>W30x116</td>
<td>34.2</td>
<td>4930</td>
<td>164</td>
<td>378</td>
</tr>
<tr>
<td>W33x118</td>
<td>34.7</td>
<td>5900</td>
<td>187</td>
<td>415</td>
</tr>
</tbody>
</table>

6.3.2 Numerical Model: LAM5

The 5-story structure (LAM5) required ten W-beams. Structural properties for these beams are listed in Table 6-2. As for the 3-story structure, an inner bay was chosen for the plane strain analysis in FLAC (equivalent to bays 2-5 in Figure 6-1(b)). The distributed load for a unit width was computed with MathCAD, as was done for Tests 1 – 3 (see Chapter 4 for a complete description of the procedure). The prototype structure’s fundamental periods of vibration were 1.53 s (first mode) and 0.52 s (second mode). The numerical model achieved 1.5 s (first mode) and 0.49 s (second mode) fundamental periods of vibration. Again, plastic moments for each structural component were specified for each member based on the plastic modulus (Zₓ) and a steel yield stress of 50 ksi. Due to time constraints, a minimal value of Rayleigh damping (3%) was applied to the structure to prevent ringing (application of structural element damping decreases the timestep by a minimum of an order of magnitude – see discussion in Chapter 4).

Table 6-2: Structural member properties for 5-story structure (LAM3)

<table>
<thead>
<tr>
<th>Designation</th>
<th>A (in²)</th>
<th>Iₓₓ (in⁴)</th>
<th>Iᵧᵧ (in⁴)</th>
<th>Zₓ (in⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column (exterior)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W14x211</td>
<td>62.0</td>
<td>2660</td>
<td>1030</td>
<td>390</td>
</tr>
<tr>
<td>W14x311</td>
<td>91.4</td>
<td>4330</td>
<td>1610</td>
<td>603</td>
</tr>
<tr>
<td>Column (interior)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W14x283</td>
<td>83.3</td>
<td>3840</td>
<td>1440</td>
<td>542</td>
</tr>
<tr>
<td>W14x398</td>
<td>117.0</td>
<td>6000</td>
<td>2170</td>
<td>801</td>
</tr>
<tr>
<td>W14x426</td>
<td>125.0</td>
<td>6600</td>
<td>2360</td>
<td>869</td>
</tr>
<tr>
<td>Girders</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W21x62</td>
<td>18.3</td>
<td>1330</td>
<td>57.5</td>
<td>144</td>
</tr>
<tr>
<td>W27x129</td>
<td>37.8</td>
<td>4760</td>
<td>184</td>
<td>395</td>
</tr>
<tr>
<td>W27x146</td>
<td>42.9</td>
<td>5630</td>
<td>443</td>
<td>461</td>
</tr>
<tr>
<td>W33x141</td>
<td>41.6</td>
<td>7450</td>
<td>246</td>
<td>514</td>
</tr>
<tr>
<td>W33x152</td>
<td>44.7</td>
<td>8160</td>
<td>273</td>
<td>559</td>
</tr>
</tbody>
</table>
6.4 Prototype Earthquake Ground Motions

These prototypical analyses used five ground motions from the original list of earthquake recordings considered for use in the centrifuge experiments. As discussed in Chapter 3, the experimental motions for Tests - 1 through - 3 were modified for use in the centrifuge. This included removal of key frequencies (e.g., those which corresponded to the natural frequencies of vibration of the arm) and amplitude scaling. The ground motions for this prototypical scenario correspond to the original time histories available from the PEER database (http://peer.berkeley.edu/products/strong_ground_motion_db.html). The only modification made to the acceleration time histories was the application of an amplitude scale factor. Amplitude scaling several of the motions allowed comparison of the prototypical responses in this chapter to those observed in the centrifuge (Tests-1 through -3). These acceleration time histories were then converted to shear stress time histories for use with quiet boundaries in FLAC.

Table 6-3: Ground motion suite: prototype analysis

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Location of Recording (Designator)</th>
<th>Type of Motion (Intensity)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landers (1992)</td>
<td>Joshua Tree (JOSL)</td>
<td>Ordinary (Low)</td>
</tr>
<tr>
<td></td>
<td>Lucerne (LCN)</td>
<td>Near Fault (High)</td>
</tr>
<tr>
<td>Northridge (1994)</td>
<td>Rinaldi Recording Station (RRS)</td>
<td>Near Fault (High)</td>
</tr>
<tr>
<td></td>
<td>Sylmar Converter Station (SCSL)</td>
<td>Near Fault (Low)</td>
</tr>
<tr>
<td>Loma Prieta (1989)</td>
<td>Saratoga West Valley College (WVCL)</td>
<td>Near Fault (Low)</td>
</tr>
</tbody>
</table>

6.4.1 FLAC Treatment of Ground Motions

Previously, ground motions had been applied directly to the base of the numerical model as acceleration time histories; that method approximates a “rigid base” condition in FLAC. For the centrifuge experiments, this was a realistic approach. The impedance contrast between the soil and the flexible shear beam container (FSB2.2) was more than an order of magnitude; therefore, the wave energy was reflected back into the model by the base of the centrifuge, just as occurs with a fixed-base condition in FLAC.

This scenario could be encountered outside the laboratory if modeling a low-velocity soil column immediately above a layer of competent high-velocity bedrock (e.g., very high impedance contrast). Then, the reflection of seismic energy back into the FLAC model from the bottom of the mesh would be realistic, as in the centrifuge environment. However, this is not usually the situation encountered in practice, nor is it the expected condition of the prototypical soil column modeling the Southern California region.

The compliant rock base is best approximated with a compliant base simulation, which is the other dynamic time history input option offered by FLAC. This utilizes a quiet boundary (explained in Chapter 4) at the base. This boundary is not fixed. Therefore, the dashpots lining the bottom of the mesh are able to absorb any downward propagating waves, as would a real soil profile. This condition prohibits the use of an acceleration time history as an input since the
motion of the base cannot be proscribed (the grid points must be free to move, by definition of the quiet boundary). Instead, the dynamic time history must be applied as a shear stress-time history.

To model a seismic event with this approach, first the desired acceleration time history must be converted to a velocity time history. Then, this time history must be converted to a shear stress history for use in FLAC with the following Equation 6-1:

\[
\tau(t) = 2(\rho C_s) v_{su}(t)
\]

The shear stress-time history (\(\tau\)) is dependent on the material density (\(\rho\)), the shear wave velocity (\(C_s\)), and the velocity time history (\(v_{su}(t)\)). A factor of two is introduced since the viscous dashpots which compose the quiet boundary in FLAC absorb one half of the stress upon application.

6.5 Numerical Analysis Results of LAM3

All ground motions were applied at the base of the numerical mesh as shear stress time histories. Each corresponded to an intensity level used in Test-1 through Test-3; several of the motions required amplitude scaling to achieve the target. The ordinary Joshua Tree recording (JOS) was applied as a low intensity motion (scaled to a PGA of 0.07 g). Both the Lucerne (high intensity) and Saratoga West Valley College (low intensity) motions were unscaled, with PGA values of 0.25 g and 0.26 g, respectively. The Rinaldi Recording Station (RRS), another high-intensity near-fault motion, was scaled down to a PGA of 0.25 g. Finally, the near-fault Sylmar Converter Station (SCS) was converted to a low-intensity motion, with a PGA of 0.18 g.

6.5.1 Isolated LAM3: pseudo-acceleration spectra responses

Ordinary and near-fault events were propagated through the prototypical soil profile with an isolated LAM3 structural configuration (SSI). As shown in Figure 6-2, a 121 by 29 mesh was used for this analysis. The mesh was deliberately made larger than necessary to allow for placement of an adjacent LAM3 to examine SSSI later (Section 6.5.2).
In the tradition of the previous centrifuge analyses, the dynamic response of the system to the low-intensity version of the Joshua Tree recording (JOS_L) was examined first (Figure 6-3). An initial observation that may be drawn from the pseudo-acceleration response spectra is evidence of period lengthening from a fixed-base fundamental period estimate ($T_N$) to a soil-structure system period ($T_{SSI}$). The maximum response of the 3-story frame is shifted from the fixed-base fundamental period of vibration identified during the calibration phase of model construction, $T_N = 1.08$ s, to just under 1.5 s. This is consistent with the results of Test-1, Test-2, and Test-3. For example, examination of the pseudo-acceleration spectral responses of the 3-story (prototype) SMRF in Test-1 shows period lengthening from the expected $T_N = 0.91$ s (configuration 1, Test-1) to $T_{SSI} = 1.08$ s for the JOS_L motion. Additionally, the amplitude of pseudo-acceleration response in Figure 6-3, just over 1.0 g, is similar to that observed in the centrifuge for this motion. The numerically modeled response of the 3-story (prototype) SMRF (with configuration 1) in Test-1 achieved a maximum of 1.0 g.

Next, the near-fault motions were examined: Sylmar Converter Station and Saratoga West Valley College station (both at low intensity levels) and the high-intensity motions Rinaldi Recording Station (RRS) and Lucerne (LCN). The dynamic response to the LCN motion is presented as representative for this set (Figure 6-4). As for the JOSL low-intensity ordinary motion, some period lengthening was observed; again, this was consistent with the effects observed in the centrifuge. As an example, the first mode fixed base period of the 3-story (prototype) MRF in Test-1 lengthened to $T_{SSI} = 1.25$ s during the LCN motion; in this prototypical analysis, it lengthened to 1.48 s. The amplitude achieved in this prototypical configuration was slightly higher than that modeled during Test-1 for the LCN motion. However, the peak $S_a$ value achieved at the roof level of each structure was similar: 2.45 g for Test-1 and 2.8 g for the prototype 3-story frame, LAM3 (shown in Figure 6-4). Additionally, the spectral shapes of the responses for both models were in close agreement.
Figure 6-3: Joshua Tree (low intensity), LAM3 isolated, SSI condition ($\lambda_{\text{structure}} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Figure 6-4: Lucerne (LCN) LAM3 isolated, SSI condition ($\lambda_{structure} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
6.5.2 Adjacent LAM3: pseudo-acceleration spectra responses

The seismic response of two identical adjacent structures (SSSI) was examined next. Two LAM3 structures were placed 1 m apart on a soil profile identical to that of the solitary condition, which was well within the expected region of interaction, 0.5 the width of the foundation (Jiang and Yan, 1998) (Figure 6-5). The dynamic responses of the two structures were examined and compared with the solitary condition. Rayleigh damping of 3% was applied to the structures during these analyses.

![Figure 6-5: LAM3 dual configuration (SSSI), structural damping 3%](image)

Pseudo-acceleration response spectra and acceleration time histories were recorded and plotted at the base of the numerical model and at the roof level of the 3-story structure (LAM3). Two engineering design parameters of interest (EDPs), the peak “transient” and “residual” roof drifts, were also computed for the structure. As described in Chapter 4, the peak “transient” roof drift assisted in quantifying the response of the system during strong shaking (it is described by Equation 4-7 [Chapter 4]). The peak “residual” roof drift (Equation 4-8), which is an estimate of the difference in the position of the superstructure roof before and after shaking, was also calculated. For every metric, the dynamic response of the 3-story (LAM3) structure was observed to be either (1) unchanged or (2) slightly lower in the adjacent (SSSI) case than in the solitary (SSI) configuration.

The Joshua Tree (low intensity) ordinary motion (JOS_L) displayed negligible difference between SSI and SSSI (Figure 6-6). The acceleration time histories showed no period lengthening, and the spectral responses were almost identical. There was slightly greater amplification at the site period (e.g., 0.5 sec) for the solitary structure.

In contrast, a near-fault, low intensity motion, Sylmar Converter Station (SCS_L) did exhibit differences between the SSI and SSSI configurations (Figure 6-7). The pseudo-
acceleration spectrum for the SSSI condition (red trace) was lower than that estimated for the SSI condition (blue trace) for all periods below 2 s. This effect was especially significant near the peak observed response at 1.4 s. Similarly, the high-intensity Rinaldi Recording Station motion induced larger accelerations at the roof of the LAM3 structure in the short period range (i.e., less than 1.5 s) for the solitary structure (SSI) than the adjacent frame (SSSI) (Figure 6-8).

The two engineering design parameters, peak “residual” and “transient” drifts, were computed for the 3-story structure (LAM3) in (1) the solitary structural (SSI) and (2) the adjacent (SSSI) configurations. Each of these was compared in graphical (Figure 6-9 and Figure 6-10) and tabular form (Table 6-4). The plots of “transient” peak roof drift corroborate the observations made from examination of the spectral and time history responses. The 3-story structure’s EDPs computed for the low intensity motions exhibited little change between SSI and SSSI configurations (e.g., JOS_L). The higher intensity motions induced a lower response in the adjacent configuration (SSSI) than in the solitary structure (SSI).

These findings were consistent through all motions examined (Appendix C). The effects of SSSI were not large for this ordinary, low-rise frame founded on competent soil with a non-resonant first-mode fundamental period of vibration. The slight SSSI effects that were observed for the structures was either beneficial, lowering the pseudo acceleration response spectra in the short period regions, or negligible.
Figure 6-6: Joshua Tree (low intensity), LAM3 comparison: solitary SSI (blue) and adjacent SSSI (red) ($\lambda_{structure} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Figure 6-7: Sylmar Converter Station, low intensity (SCS_L), LAM3 comparison: solitary SSI (blue) and adjacent SSSI (red) ($\lambda_{\text{structure}} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Figure 6-8: Rinaldi Recording Station (RRS), LAM3 comparison: solitary SSI (blue) and adjacent SSSI (red) ($\lambda_{structure} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Table 6-4: LAM3 engineering design parameters for single (SSI) and adjacent (SSSI) structures.

<table>
<thead>
<tr>
<th>Motion ID</th>
<th>Peak Roof Drift</th>
<th>Residual Roof Drift</th>
<th>Peak Roof Drift</th>
<th>Residual Roof Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Δ\text{COM}) / \Delta_{\text{MAX}} m/11.8 m</td>
<td>(Δ\text{COM}) / Δ_{n,\text{res}} m/11.8 m</td>
<td>(Δ\text{COM}) / Δ_{n,\text{res}} m/11.8 m</td>
<td>(Δ\text{COM}) / Δ_{n,\text{res}} m/11.8 m</td>
</tr>
<tr>
<td>JOS_L</td>
<td>0.81%</td>
<td>0.24%</td>
<td>0.76%</td>
<td>0.20%</td>
</tr>
<tr>
<td>WVC_L</td>
<td>4.33%</td>
<td>0.22%</td>
<td>4.14%</td>
<td>0.13%</td>
</tr>
<tr>
<td>SCS_L</td>
<td>1.67%</td>
<td>0.73%</td>
<td>1.59%</td>
<td>0.54%</td>
</tr>
<tr>
<td>LCN</td>
<td>3.70%</td>
<td>1.13%</td>
<td>3.45%</td>
<td>0.96%</td>
</tr>
<tr>
<td>RRS</td>
<td>3.80%</td>
<td>0.13%</td>
<td>3.39%</td>
<td>0.23%</td>
</tr>
</tbody>
</table>

Figure 6-9: LAM3 “transient” drift computed at roof level, single (green) and adjacent (red) configurations.

Figure 6-10: LAM3 “residual” drift computed at roof level, single (green) and adjacent (red) configurations.
6.5.3 Adjacent LAM3: pseudo-acceleration spectra responses of tuned superstructure

The adjacent structural configuration examined in Section 6.5.2 exhibited either (1) a slightly beneficial or (2) a negligible effect on the dynamic response of the 3-story frames. In a realistic configuration, it is equally, if not more, likely that a structure will be located near buildings with completely different dynamic properties. Therefore, another numerical analysis was conducted with a different adjacent low-rise structure to explore the changes in dynamic response.

To accomplish this, an alternate low-rise structure needed to be selected. Two components that had the largest impact on the level of SSI achieved during earlier analyses were (1) the dimensionless wave parameter, $1/\sigma$, and the other SSI predictive equations, and (2) resonance of the structure with the soil column. These two aspects of the system were explored separately to generate a new 3-story frame.

First, the SSI predictive equations were considered. To achieve the threshold at which significant SSI would be expected ($1/\sigma \geq 0.2$), the fixed-base fundamental period of the 3-story frame on the soil profile used in this analysis needed to be shortened to at least 0.13 s. Such a fixed-base fundamental period was not realistic for this steel frame. Thus, the design focus was shifted to the second aspect: resonance with the soil column. A new 3-story MRF was created with a footprint and mass identical to the original 3-story frame, but with a fundamental period of vibration of 0.63 s. This new structure ($T_N = 0.63$ s) was positioned beside the original 3-story frame ($T_N = 1.08$ s) as a second adjacent configuration in FLAC. While the dimensionless wave parameter, $1/\sigma$, increased for the newly tuned 3-story frame ($T_N = 0.63$ s), it was still far below the value established as the threshold at which significant interaction is expected. The value of $1/\sigma$ was estimated to double from the original 3-story frame ($T_N = 1.08$ s) value of $1/\sigma = 0.02$ to the tuned 3-story structure ($T_N = 0.63$ s) value of $1/\sigma = 0.04$.

Direct comparison of the dynamic response of the original 3-story structure ($T_N = 1.08$ s) beside (1) the resonant frame ($T_N = 0.63$ s) and (2) the original frame ($T_N = 1.08$ s) allowed the researcher to observe some of the differences in SSSI between each configuration. Pseudo-acceleration response spectra and acceleration time histories at the base and roof level of the structure were computed, as before. Peak “transient” and “residual” drifts were also computed. Seismic responses from two of the motions, JOS_L and RRS, are presented in Figure 6-11 and Figure 6-13, respectively.

For the JOSL motion, the dynamic response of the 3-story original structure ($T_N = 1.08$ s) exhibited no noticeable change between the two SSSI configurations. Despite the change in fundamental period of vibration of the adjacent, tuned 3-story structure ($T_N = 0.63$ s), the SSSI effects on the original 3-story structure ($T_N = 1.08$ s) were unaltered. Similarly, the acceleration time history at the roof of the original 3-story structure ($T_N = 1.08$s) was perfectly matched for both adjacent configurations (i.e., an adjacent 3-story frame with either $T_N = 1.08$s or $T_N = 0.63$s). The EDPs confirmed these observations, as they also showed no noticeable change between the two configurations.

The low-intensity near-fault motions did show a slight increase in dynamic response for the adjacent resonant structure ($T_N = 0.63$ s) condition over the adjacent original ($T_N = 1.08$ s) structure configuration. A representative motion, SCS_L, is presented in Figure 6-12. The
response of the baseline original 3-story LAM3 structure shows slight amplification at the peak response for the adjacent resonant condition (adjacent building with $T_N = 0.63$ s) when compared with the adjacent original condition (adjacent building with $T_N = 1.08$ s). While small, this indicates that SSSI is affecting the response of the system differently for the adjacent resonant condition than for the adjacent original condition.

The high intensity near-fault events, including RRS, also showed a slight change in dynamic response between the two adjacent configurations. For the RRS motion, the dynamic response of the original 3-story frame ($T_N = 1.08$ s) increased from the adjacent original configuration (i.e., both MRFs with $T_N = 1.08$ s) to the adjacent tuned structure configuration (i.e., one MRF with $T_N = 1.08$ s, one MRF with $T_N = 0.63$ s). The peak response in the pseudo-acceleration response spectrum (roof level) increased from 4.9 g for the two original frames (both $T_N = 1.08$ s) to 5.2 g for the original frame adjacent to the tuned frame ($T_N = 1.08$ s and $T_N = 0.63$ s, respectively). The peak “transient” roof drift also showed slight variation, with the analysis of adjacent original and tuned structures ($T_N = 1.08$ s and $T_N = 0.63$ s, respectively) plotting slightly higher values than those associated with the two original frames (both $T_N = 1.08$ s).
Figure 6-11: Joshua Tree (low intensity), LAM3 comparison: tuned adjacent 3-story structure ($T_s = 0.63$ s and $T_n = 1.08$ s [green]) and original adjacent 3-story ($T_s = 1.08$ s [red]) ($\lambda_{structure} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Northridge, 1994  
(Sylmar Converter Station)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
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<td>Near fault</td>
<td>Low</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LAM3, Roof, Resonant</th>
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</thead>
<tbody>
<tr>
<td>LAM3, Foundation, Resonant</td>
</tr>
<tr>
<td>LAM3, Original, Roof</td>
</tr>
<tr>
<td>LAM3, Original, Foundation</td>
</tr>
</tbody>
</table>

Figure 6-12: Sylmar Converter Station, low intensity (SCS_L), LAM3 comparison: tuned adjacent 3-story structure ($T_a = 0.63$ s and $T_n = 1.08$ s (green)) and original adjacent 3-story ($T_a = 1.08$ s (red)) ($\lambda_{structure} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Figure 6-13: Rinaldi Recording Station, high intensity (RRS), LAM3 comparison: tuned adjacent 3-story structure ($T_n = 0.63$ s and $T_n = 1.08$ s [green]) and original adjacent 3-story ($T_n = 1.08$ s [red]) ($\lambda_{structure} = 3\%$). Acceleration time histories at (1) the base of the mesh (recorded at the bottom of the numerical model) and (2) the center of the roof of the 3-story structure are plotted for reference.
Table 6-5: LAM3 engineering design parameters for adjacent resonant (SSSI) and adjacent original (SSSI) structures.

<table>
<thead>
<tr>
<th>Motion ID</th>
<th>Peak Roof Drift</th>
<th>Residual Roof Drift</th>
<th>Peak Roof Drift</th>
<th>Residual Roof Drift</th>
</tr>
</thead>
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<td>IOS, L</td>
<td>0.76%</td>
<td>0.21%</td>
<td>0.76%</td>
<td>0.20%</td>
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<td>WVC, L</td>
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<td>0.10%</td>
<td>4.14%</td>
<td>0.13%</td>
</tr>
<tr>
<td>SCS, L</td>
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<td>0.50%</td>
<td>1.59%</td>
<td>0.54%</td>
</tr>
<tr>
<td>LCN</td>
<td>3.37%</td>
<td>1.1%</td>
<td>3.45%</td>
<td>1.08%</td>
</tr>
<tr>
<td>RRS</td>
<td>3.46%</td>
<td>0.20%</td>
<td>3.39%</td>
<td>0.23%</td>
</tr>
</tbody>
</table>

Figure 6-14: LAM3: peak “transient” roof drift of (1) the original 3-story structure ($T_N = 1.08$ s) adjacent to a tuned 3-story structure ($T_N = 0.63$ s) [green] and (2) adjacent original 3-story structures (both with $T_N = 1.08$ s) [red].

Figure 6-15: LAM3: peak “residual” roof drift of (1) the original 3-story structure ($T_N = 1.08$ s) adjacent to a tuned 3-story structure ($T_N = 0.63$ s) [green] and (2) adjacent original 3-story structures (both with $T_N = 1.08$ s) [red].
6.6 Numerical Analysis Results of LAM5

The 5-story frame (LAM5) had a considerably more complex structural composition than the 3-story structure (LAM3). Ten W-beams were used for the columns and beams of LAM5, while only five were needed for LAM3. However, the numerical analysis procedure implemented for the 5-story model was identical to that used for the 3-story model. Ground motions, amplitude-scaled to the centrifuge input values and converted to shear stress time histories, were applied to the base of the numerical model. The Joshua Tree motion (JOS) (at a low intensities), the Lucerne (LCN) motion, Rinaldi Recording Station (RRS) motion, and the low-intensity versions of the Saratoga West Valley College (WVC) and Sylmar Converter Station (SCS) motions were examined.

6.6.1 Isolated LAM5: pseudo-acceleration spectra responses

The seismic responses due to two input earthquake motions are presented in this section for the case with 2% Rayleigh damping applied to the superstructures; the seismic responses for the undamped case to other earthquake ground motions are included in Appendix C. The first seismic response is plotted for the near-fault, low-intensity motion, Sylmar Converter Station (SCS_L) ground motion. Amplification was observed near the mean period of the motion, \( T_m = 1.12 \) s, as well as in the 2.5 s region. A spike at approximately 0.7 s was also present in the response; while visible in the 3-story (LAM3) response, it was considerably larger for this structure. This was likely due to the second mode of vibration of the LAM5 structure (0.52 s). Some period lengthening due to SSI is expected, which would have moved the second mode of vibration to a larger value (longer period).

The second seismic response for the LAM5 structure examined was for a high-intensity, near-fault motion, Lucerne (LCN). The dynamic response of the superstructure is reminiscent of the response observed in the free-field analysis (Chapter 3). Amplification in the 0.6 s to 0.7 s range and at 1 s was observed in the seismic site response analysis, and it is also present in the LAM5 pseudo-acceleration response spectrum. The 0.6 s to 0.7 s spike corresponded to both the site period and the lengthened second mode of vibration. As for the SCS_L motion, amplification was also observed at a longer period. For this more intense motion, which induced more nonlinearity, the amplification occurred at a slightly longer period (e.g., 3.18 s for LCN and 2.76 s for SCS_L).

The stiffness-proportional component of Rayleigh damping significantly reduces the critical timestep in explicit numerical computations (Belytschko, 1983). The FLAC manual elaborates, "As the damping ratio corresponding to the highest natural frequency is increased, the timestep is reduced. This can result in a substantial increase in runtimes for dynamic simulations." (Itasca, 2005). Therefore, due to temporal limitations and in the interest of efficiency, only two of the five ground motions were applied to the LAM5 structure with structural Rayleigh damping (Figure 6-16 and Figure 6-17). The undamped results from the remaining motions are included in Appendix C. The results for both an undamped superstructure (red trace) and a damped structure (blue trace) are compared for the LCN motion (Figure 6-18). Some structural damping is required; without the realistic application of structural damping, high frequency noise and unrealistic amplitudes of dynamic response are estimated. Since Rayleigh damping in FLAC is essentially frequency-independent, the overall spectral shape is unchanged – it is only the amplitude that is altered.
Figure 6-16: Sylmar Converter Station (low-intensity), LAM5 during SSI, with structural Rayleigh damping ($\lambda_{\text{structure}} = 2\%$) (blue).
Figure 6-17: Lucerne (high-intensity), LAM5 during SSI, with structural Rayleigh damping ($\lambda_{\text{structure}} = 2\%$) (blue).
Northridge, 1994
(Sylmar Converter Station)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near-fault</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 6-18: Sylmar Converter Station (low-intensity), LAM5 comparison: SSI, with structural Rayleigh damping ($\lambda_{\text{structure}} = 2\%$ [blue]) and with no structural Rayleigh damping ($\lambda_{\text{structure}} = 0\%$ [red]).
### 6.7 Summary of LAM3 and LAM5 Results

In summary, the dynamic response results from the prototypical 3-story SMRF model were comparable to those observed of the MS1F_SF80 frame used in Test-1 and Test-2. Since the centrifuge model was a simplified, single-degree-of-freedom system based on LAM3, this comparison confirmed the validity generalizing the observations made of the experimental model, because the centrifuge model captured many of the key aspects of the seismic response of the prototype structure.

The effects of SSSI were considered in FLAC for two structural configurations. One analysis examined two identical 3-story SMRFs (each with $T_N = 1.08$ s) separated by 1 m. The other configuration examined the response of one original 3-story SMRF ($T_N = 1.08$ s) located immediately adjacent to a 3-story SMRF that was tuned to achieve resonance with the soil column ($T_N = 0.63$ s). The effects of SSSI for this low-rise, SMRF structure founded on competent dense, dry sand were observed to be either beneficial, slightly lowering the amplitudes of seismic response, or negligible for the two original 3-story frames (each with $T_N = 1.08$ s). Conversely, the original structure ($T_N = 1.08$ s) paired with the tuned structure ($T_N = 0.63$ s) yielded a slightly higher response than that observed in the other SSSI configuration for all but the lowest intensity motions (e.g., JOS_L).

Therefore, the effects of SSSI for these typical, low-rise moment resisting frames do not seem to be of primary importance. This is consistent with observations made by other researchers (Stewart et al., 1999), as SSI predictive equations, including the dimensionless wave parameter ($1/\sigma$), were far below the threshold indicative of a high likelihood of significant interaction. However, it is important to note that there was some difference between the two adjacent configurations examined, with a slight increase in dynamic response for the adjacent resonant case. As mentioned, these cases were such that SSSI was expected to be small to negligible – but effects were still observed. Considering that such a small adjustment to the properties of one low-rise, ordinary structure on a mat foundation could induce an identifiable and consistent difference in the dynamic response of an adjacent low-rise building, it is important to recognize that the effects of adjacent structures on even the most ordinary of buildings may be significant at times and thus should not be universally ignored.

The 5-story structure (LAM5) exhibited a response consistent with expectations from previous work. The overall response included amplification at the first and second modes of vibration of LAM5, with peak values centered around the site period (0.6 s) and the SSI-lengthened second mode of vibration of the structure. Lastly, expression of the mean period of the motion (variable, based on motion) was apparent for each record examined.
7 Conclusion

7.1 Summary

Dense urban environments are becoming omnipresent in today’s increasingly global society. Many of these population centers are located in active seismic zones. Managing this confluence of natural dynamic activity and human urbanization is one of the foremost challenges facing modern civil engineers.

The field of earthquake engineering has been quantifying the effects of SSI since the seminal work of several researchers in the 1970s (e.g., Jennings, 1970; Luco and Contesse, 1973; Seed et al., 1975). The examination of a single structure founded on a soil profile has yielded many insights into the interaction between a structure and the soil beneath it. However, these insights have been limited by a key shortcoming, because they consider only a single, solitary building. With the trend toward increasing urbanization, the dynamic response of an isolated structure is increasingly divorced from the dense urban centers with clusters of adjacent buildings that are commonly encountered today.

Therefore, structure-soil-structure interaction (SSSI) research has recently become more prominent. Although this topic does not yet command a level of attention equal to that assigned to investigating SSI effects, it is attracting more interest. There are examples of buildings interacting with adjacent structures through the subsurface that show significant alterations to the dynamic responses of all components of the soil-structure system (e.g., Menglin, 2011). For these cases, the SSSI phenomenon can be significant.

This dissertation focused on expanding the knowledge base in the area of SSI and SSSI. Through the NSF-funded NEES City-Block (NCB) research project (of which this dissertation is a part), this was accomplished in three steps. A suite of laboratory case histories were generated in a carefully monitored laboratory environment. These case histories were then used to calibrate and evaluate the capabilities of a commonly employed numerical simulation tool. Finally, more realistic geometries, too complicated for the centrifuge testing environment, were examined with the calibrated numerical simulation tool.

This dissertation addressed steps two and three in that multi-level approach. Numerical geotechnical modeling of (1) three of six planned centrifuge tests in the NCB project and (2) examination of a more realistic prototypical structure were completed and documented in this dissertation. The widely used two-dimensional, plane-strain geotechnical software program, FLAC (Itasca, 2005) was utilized to accomplish this through the direct dynamic analysis approach.

Experimental data and numerical results were examined in both spectral and time domains. The capabilities of constitutive models to capture the dynamic site response of a free-field soil column were examined to ensure that the subsurface could be modeled accurately. Then, the dynamic responses of several moment resisting frames (MRFs) on deep and shallow foundations were examined (Test-1 and Test-2). For the structures analyzed in Test-1 and Test-2, EDPs of interest were computed for both centrifuge and numerical data. The structural models in these tests exhibited kinematic interaction and provided insight into some of the key effects of SSI and SSSI.
A third centrifuge test that focused on inertial SSI and SSSI effects was also examined. The responses of two MRFs and an elastic shear wall yielded insight into the challenges of successfully modeling resonant conditions. For cases of extreme SSSI effects and multi-component resonance, numerical results were observed to be highly sensitive to small changes in the model parameters.

Lastly, two prototypical frame structures were analyzed. These MRFs exhibited dynamic responses similar to those observed in the laboratory testing environment. Examination of the SSSI and SSI conditions for a three-story structure was also conducted. For the combinations of adjacent low-rise structures founded on a dense sand soil deposit, SSSI effects were found to be insignificant or only slight beneficial or detrimental for the five earthquake ground motions used for dynamic analysis. Additional analyses are warranted to investigate fully SSSI effects for a larger number of adjacent structural configurations.

7.2 Findings

The research presented in this dissertation provided insight into the capabilities and limitations of centrifuge and numerical modeling of SSI and SSSI. Constitutive model selection, physical model complexities, and comparison of physical and numerical models were discussed. Key findings from the analyses presented in this dissertation are highlighted below.

- A free-field seismic site response analysis was conducted with FLAC for a suite of ground motions examined in a centrifuge environment. Both near-fault and ordinary motions were included, spanning a variety of intensity levels. A Mohr-coulomb based constitutive model with hysteretic damping (UBCHYST; Naesgaard, 2011) was employed to capture the nonlinear, stress dependent seismic response of the dry, dense \( (D_r = 80\%) \) Nevada sand profile considered in the experimental study. Pseudo-acceleration response spectra and acceleration time histories were examined at the base and surface of the soil profile. These metrics were used to quantify the relative success of the numerical model at capturing the free-field dynamic response observed in the laboratory. Ordinary motions, at both high and low intensities, were captured very well by the numerical simulations. For the low-intensity near-fault motions, the amplitude of response was consistently overpredicted; however, the shapes of the pseudo-acceleration response spectra were matched. The most challenging motions to model, near-fault high intensity motions, were well–captured overall; the peak responses from just two ground motions (Port Island, modified and Sylmar Converter Station) were not well captured.

- The two-dimensional, plane strain numerical simulations were able to approximate the key response aspects of the inherently three-dimensional, moment resisting frame structures used in the centrifuge experiments with the implementation of several modeling guidelines. Dynamic responses of both near-fault and ordinary motions, at varying intensity levels, were successfully captured for the three-story (prototype) MRF on spread footings and the nine-story (prototype) MRF on a deep foundation in both Test-1 (SSI) and Test-2 (SSSI). Engineering design parameters (EDPs) of interest, including peak roof drifts and maximum displacements and accelerations, were computed for both centrifuge data and numerical model outputs; the observed laboratory values were captured and the trends were matched reasonably well. Pseudo-acceleration response
spectra were also computed. Again, the spectra from the measured and computed motions were consistent.

- The significant SSSI effects observed in the centrifuge test (Test-2) were captured for both a three-story and a nine-story MRF. Changes in frequency content from the solitary structure to one with adjacent structures were captured for both structures (shown in pseudo-acceleration response spectra). Trends in measured and computed engineering demand parameters, including peak roof drifts and maximum displacement and accelerations, were in agreement.

- Numerical analyses comparing the dynamic response of a three-story MRF with and without an adjacent nine-story MRF founded on a deep basement (Test-2) were conducted. Differences between these configurations were observed in both the frequency content and acceleration amplitudes of the response. Flexible-base period lengthening was observed and the amplitude of dynamic response was lowered for the solitary structure case relative to the SSSI (adjacent structures) case.

- The plane-strain numerical model showed limitations in capturing resonance and strong SSSI effects for some shakes during Test-3. When the structures and the underlying soil column were in resonance, the numerical output was consistently lower than that observed in the centrifuge experiments. This has implications for numerical modeling of seismic analysis when resonance is expected. There are also additional issues to explore in terms of the capabilities of the centrifuge container to respond realistically when excited by intense input motions that are in resonance with the soil deposit in the container.

- The multi-component resonant condition with three-dimensional effects in Test-3 was difficult to capture in the two-dimensional, plane strain numerical model. Several sensitivity studies were conducted on parameters within the FLAC environment for the resonant (Test-3) condition. These included (1) the shear wave velocity profile, (2) interface elements in FLAC, (3) the fixed-base fundamental period of vibration estimate for a structure, and (4) constitutive model parameters. The results indicated that the constitutive model exhibited excessive damping at shear strain values greater than 0.1%.

- A three-story prototypical structure (LAM3) upon which the single-degree-of-freedom (SDOF) three-story (prototype) MRF model used in the centrifuge experiments was based was investigated through FLAC analyses. The dynamic response of the LAM3 structure was similar to that of the simplified SDOF for the considered ground motions. Since the LAM3 design was used to develop the structural models used in the centrifuge, the dynamic responses of the two were expected to be similar. Therefore, confirmation that intensity levels of response, spectral shapes, and acceleration time histories from the LAM3 configuration paralleled the centrifuge output provided confidence in the previous experimental work described in Mason (2011).

- Numerical analyses of two non-resonant, prototypical structures with shallow foundations (LAM3) were conducted. The results from this work indicated that for the considered
low-rise structures founded on a competent, non-resonant soil column, for the ground motions used in the analysis, which included both near-fault and ordinary motions at high and low intensities, SSSI effects were either insignificant or only slightly beneficial or detrimental. As an example, for the case wherein the adjacent prototypical structures did not have identical properties and were shaken by near-fault, high intensity motions, the structure that was in resonance with the soil deposit induced slight amplification in the dynamic response of the adjacent original LAM3 structure relative to the case with two adjacent identical LAM3 structures.

7.3 Recommendations for Future Research

The earthquake engineering field encompasses many topics that have yet to be explored fully. The area of SSSI is one of these, and myriad research topics remain. With respect to numerical modeling of SSSI, some of the most pertinent to this dissertation are enumerated below.

- The most obvious is the examination of the Test-3 data suite in a fully three-dimensional software program (e.g., FLAC3D). The ability of the model to capture the response of the MRF founded on spread footings would be of great interest to better understand the seismic response of structures during intense inertial interaction.

- Limited work has been performed on investigating the seismic response of structures in resonance with the soil profile for the SSSI case. Analysis from Test-3 indicates that models extrapolated from calibration to non-resonant behavior (e.g., in this research, Test-1 and Test-2 data) may predict a different dynamic response than would actually occur. Without the centrifuge data, the doubling of the spectral response of the MRFs examined would have been considered reasonable, when in reality, the centrifuge response was increased by factors up to five times the non-resonant response.

- Additionally, more work needs to be completed to understand the influence of the centrifuge system when extreme model responses are induced. The resonant condition (Test-3) exhibited unexpected amplitudes of dynamic response, which were not matched by a calibrated, two-dimensional numerical tool. This may be due to three-dimensional effects that the numerical model was unable to capture, sensitivity of the resonant condition to system components and the numerical model’s inability to perfectly capture all aspects of the system, or both. However, it is also possible that the centrifuge box, soil, and arm may have introduced extraneous energy during resonance that was transmitted into the data. Additional work in this area would yield valuable insight into the limitations and effects of the centrifuge as a research tool.

- There is a need for development of better boundary condition models for comparison with centrifuge output. The centrifuge is a powerful tool, particularly for earthquake engineers. As the industry moves toward increased use of this tool (e.g., Kutter et al., 2008) numerical modeling of the data for calibration and validation purposes will likely grow in importance. The boundary conditions associated with the laminar box are complex (Ilankatharan and Kutter 2010; Choy 2011), and currently, a generalized model
capable of capturing the effects of the edges of the container does not exist. While the current modeling techniques are good, there is a need for a better model of this condition.

- Liquefaction is another aspect of geotechnical numerical modeling that needs to be examined for the SSSI condition. The work of Dashti and Bray (2012) is an excellent example of the work currently being done to better understand the mechanisms of SSI and liquefaction. To the author’s knowledge, geotechnical numerical research has been limited to rigid superstructures, SDOF systems, or surface loads. As illustrated by the widespread damage in New Zealand (2010-2011) and Japan (2011) due to recent seismic events, the response of structure-soil-structure systems to cyclic mobility can be dramatic and devastating. The ability of prevalent analytical tools to capture SSSI effects and the significant nonlinearity resulting when soil liquefies with a direct approach needs to be evaluated further.

In conclusion, the field of earthquake engineering is a rapidly advancing field. Engineers are continuing to learn about the interactions between the soil column, superstructures, and the foundations that form the interface between them. As populations increase and construction continues in highly seismic zones, the role of SSI and SSSI will only grow in importance.
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Appendix A

A.1 UBCHYST Calibration:

Shear modulus reduction and damping curves from UBCHYST for each depth of the soil profile are presented below. Note that for layers which fell between discrete reported Darendeli (2001) curve values (e.g., 0.25atm, 1atm, etc.), the higher and lower laboratory curves were plotted and used to bound the condition.
Shear Strain (%) vs. Damping (%)

- 3.5m to 5.9m
- Darendeli (2001), 0.25 atm
- Darendeli (2001), 1 atm

Damping (%)

- 0.0001
- 0.001
- 0.01
- 0.1
- 1

Shear Strain (%)

- 0.0001
- 0.001
- 0.01
- 0.1
- 1
Shear Strain (%)

5.9m to 8.0m
Darendeli (2001), 0.25atm
Darendeli (2001), 1 atm

Damping (%)

5.9m to 8.0m
Darendeli (2001), 0.25atm
Darendeli (2001), 1 atm
G'/G'_{\text{max}} vs. Shear Strain (%) for 10.1m to 12.2m and Darendeli (2001) at 0.25atm and 1 atm.

Damping (%) vs. Shear Strain (%) for 10.1m to 12.2m and Darendeli (2001) at 0.25atm and 1 atm.
Shear Strain (%)

Damping (%)

Darendeli (2001), 1 atm
Darendeli (2001), 4 atm

14.3m to 16.4m
G/G_{max} vs. Shear Strain (%)

Shear Strain (%)

Damping (%) vs. Shear Strain (%)

Darendeli (2001), 1 atm
Darendeli (2001), 4 atm

20.6m to 22.7m
A.2 SHAKE2000 and FLAC Comparison:

Results from the SHAKE2000 and FLAC comparison are shown below. At the lowest shaking level (0.0001g), the results presented in the Itasca manual were replicated. As shaking increased, output from the two software programs began to diverge. At the highest levels of shaking, FLAC yielded a significantly lower response than that predicted by SHAKE2000.

![FLAC vs. SHAKE (0.0001 g max)](image)

Figure A-1: Sand and clay profile FLAC vs. SHAKE comparison (scaled to 0.0001g maximum)
Figure A-2: Sand and clay profile FLAC vs. SHAKE comparison (scaled to 0.001g maximum)

Figure A-3: Sand and clay profile FLAC vs. SHAKE comparison (scaled to 0.01g maximum)
Figure A-4: Sand and clay profile, FLAC vs. SHAKE comparison (scaled to 0.1g maximum)
Figure A-5: Sand profile, FLAC vs. SHAKE comparison (scaled to 0.001g maximum)

Figure A-6: Sand profile, FLAC vs. SHAKE comparison (scaled to 0.01g max)
Figure A-7: Sand profile, FLAC vs. SHAKE comparison (scaled to 0.1 g max)
Appendix B

B.1 Test-1 (HBM02) Mean and Predominant Period Estimates from Seismosignal and SHAKE2000:
The mean ($T_m$) and predominant ($T_p$) periods from two computer software suites were calculated for all motions in the centrifuge sequence. These estimates were computed from the values recorded during shaking at the base of the container (horizontal accelerometer “HA2”). The $T_m$ values were used as the center frequencies for application of Rayleigh damping to the mesh.

Table B-1: Ground motion suite, mean and predominant periods

<table>
<thead>
<tr>
<th>Motion, Date run</th>
<th>$T_p$, $T_m$ from Seismosignal (Base, Input Motion HA2)</th>
<th>$T_p$, $T_m$ from SHAKE2000 (Base, Input Motion HA2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>JOS_L, 8-11-2011</td>
<td>$T_p = 0.66$ sec; $T_m = 0.84$ sec</td>
<td>$T_p = 0.67$ sec; $T_m = 0.847$ sec</td>
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<tr>
<td>TCU, 8-11-2011</td>
<td>$T_p = 0.26$ sec; $T_m = 0.455$ sec</td>
<td>$T_p = 0.265$ sec; $T_m = 0.459$ sec</td>
</tr>
<tr>
<td>RRS, 8-11-2011</td>
<td>$T_p = 0.30$ sec; $T_m = 0.754$ sec</td>
<td>$T_p = 0.29$ sec; $T_m = 0.766$ sec</td>
</tr>
<tr>
<td>PTS, 8-12-2011</td>
<td>$T_p = 0.34$ sec; $T_m = 0.836$ sec</td>
<td>$T_p = 0.345$ sec; $T_m = 0.842$ sec</td>
</tr>
<tr>
<td>SCS_L1, 8-12-2011</td>
<td>$T_p = 0.94$ sec; $T_m = 1.120$ sec</td>
<td>$T_p = 0.90$ sec; $T_m = 0.816$ sec</td>
</tr>
<tr>
<td>LCN, 8-12-2011</td>
<td>$T_p = 0.94$ sec; $T_m = 1.125$ sec</td>
<td>$T_p = 0.935$ sec; $T_m = 1.113$ sec</td>
</tr>
<tr>
<td>JOS_L2, 8-13-2011</td>
<td>$T_p = 0.68$ sec; $T_m = 0.821$ sec</td>
<td>$T_p = 0.675$ sec; $T_m = 0.828$ sec</td>
</tr>
<tr>
<td>WVC_L, 8-13-2011</td>
<td>$T_p = 0.52$ sec; $T_m = 0.944$ sec</td>
<td>$T_p = 0.525$ sec; $T_m = 0.945$ sec</td>
</tr>
<tr>
<td>SCS_H, 8-13-2011</td>
<td>$T_p = 0.60$ sec; $T_m = 0.753$ sec</td>
<td>$T_p = 0.59$ sec; $T_m = 0.768$ sec</td>
</tr>
<tr>
<td>JOS_H, 8-13-2011</td>
<td>$T_p = 0.56$ sec; $T_m = 0.787$ sec</td>
<td>$T_p = 0.57$ sec; $T_m = 0.79$ sec</td>
</tr>
<tr>
<td>WPI, 8-13-2011</td>
<td>$T_p = 0.40$ sec; $T_m = 1.142$ sec</td>
<td>$T_p = 0.405$ sec; $T_m = 1.15$ sec</td>
</tr>
<tr>
<td>JOS_L3, 8-13-2011</td>
<td>$T_p = 0.68$ sec; $T_m = 0.836$ sec</td>
<td>$T_p = 0.675$ sec; $T_m = 0.843$ sec</td>
</tr>
<tr>
<td>WPI_H, 8-13-2011</td>
<td>$T_p = 0.44$ sec; $T_m = 1.153$ sec</td>
<td>$T_p = 0.44$ sec; $T_m = 1.162$ sec</td>
</tr>
<tr>
<td>PRI_mod, 8-13-2011</td>
<td>$T_p = 0.42$ sec; $T_m = 0.694$ sec</td>
<td>$T_p = 0.425$ sec; $T_m = 0.698$ sec</td>
</tr>
<tr>
<td>SCS_L2, 8-13-2011</td>
<td>$T_p = 0.90$ sec; $T_m = 0.809$ sec</td>
<td>$T_p = 0.905$ sec; $T_m = 0.815$ sec</td>
</tr>
<tr>
<td>TCU_H, 8-13-2011</td>
<td>$T_p = 0.30$ sec; $T_m = 0.428$ sec</td>
<td>$T_p = 0.37$ sec; $T_m = 0.430$ sec</td>
</tr>
<tr>
<td>WVC_H, 8-13-2011</td>
<td>$T_p = 0.44$ sec; $T_m = 0.991$ sec</td>
<td>$T_p = 0.44$ sec; $T_m = 0.988$ sec</td>
</tr>
</tbody>
</table>
B.2 Computer Code for Analysis

This section contains the MathCAD computer code used for conversion of the three-dimensional MS1-1 MRF structure to a two-dimensional model.

\[ n_{\text{scale}} = 0.55 \]

**MS1S 3D Prototype Mass Calculations**

Footing weight: based on design-tables-latest.docx

**Configuration**

\[ f_{t\text{all}} = 3.14 \text{kg} \]

\[ m_{1\text{proto}} = \left( \frac{f_{t\text{all}}}{4} \right) \left( n_{\text{scale}} \right)^3 \]

\[ m_{1\text{proto}} = 1.30604575 \times 10^7 \text{kg} \]

Centered Mass: based on Structural_Configurations_and_Model_Parameters.txt

**Configuration**

\[ \text{mass}_1 = 4.1 \text{kg} \]

\[ m_{\text{proto}} = \left( \text{mass}_1 \right) \left( n_{\text{scale}} \right)^3 \]

\[ m_{\text{proto}} = 6.8380125 \times 10^6 \text{kg} \]

Superstructure mass (beams and columns): based on design-tables-latest.docx

**Configuration**

\[ m_{s1} = 5.95 \text{kg} - 4.1 \text{kg} \]

\[ m_{s1\text{proto}} = \left( m_{s1} \right) \left( n_{\text{scale}} \right)^3 \]

\[ m_{s1\text{proto}} = 3.0613 \times 10^7 \text{kg} \]

**Total Mass (3-D mass)**

**Configuration**

\[ M_{11} = m_{\text{proto}} + m_{s1\text{proto}} + 4m_{f\text{proto}} \]

\[ M_{11} = 1.51234875 \times 10^6 \text{kg} \]

**MS1S 2D Prototype Mass Calculations**
Configuration:

\[ M_{\text{total}fg} = \frac{(4 - m_{\text{proto}}) \cdot m}{L_z} \]

\[ A_{\text{FL}fg} = h_{fg} \cdot l_{\text{unit}} \]

\[ \rho_{\text{FL}fg} = \frac{M_{\text{total}fg}}{2 \cdot A_{\text{FL}fg} \cdot l_{fg}} \]

\[ M_{\text{FL}fg} = 2 \cdot A_{\text{FL}fg} \cdot l_{fg} \cdot \rho_{\text{FL}fg} \]

\[ M_{\text{FL}fg} = 4.74925 \times 10^4 \, \text{kg} \]  
FLAC

\[ M_{\text{total}fg} = 4.74925 \times 10^4 \, \text{kg} \]  
CGM

\[ t = \left( \frac{m_{\text{proto}}}{L_z} \right) \cdot \left[ \frac{1}{(2 \cdot l_{\text{unit}} \cdot h_{\text{c}} \cdot \rho_{\text{col}}) + (l_{\text{unit}} \cdot b \cdot \rho_{\text{col}})} \right] \]

\[ t = 0.568539326 \, \text{m} \]

\[ \rho_{\text{com}} = \left( \frac{m_{\text{proto}} \cdot m}{L_z} \right) \cdot \left( \frac{1}{t \cdot l_{\text{unit}}} \right) \]

\[ \rho_{\text{com}} = 1.093394022 \times 10^4 \, \text{kg/m}^3 \]

\[ \text{mass}_{\text{protoPlaneStrain}} = \frac{\text{mass}_{\text{proto}} \cdot m}{L_z} \]

\[ \text{mass}_{\text{FLAC}} = \rho_{\text{com}} \cdot t \cdot l_{\text{unit}} \cdot h_{\text{c}} \cdot \rho_{\text{col}} + t \cdot l_{\text{unit}} \cdot l_{\text{b}} \cdot \rho_{\text{b}} + 2 \cdot A_{\text{FL}fg} \cdot l_{fg} \cdot \rho_{\text{FL}fg} \]

\[ \rho_{\text{b}} = \rho_{\text{com}} + \rho_{\text{col}} \]

Check:

\[ n_{\text{FLAC}} = 2 \cdot l_{\text{unit}} \cdot h_{\text{c}} \cdot \rho_{\text{col}} + t \cdot l_{\text{unit}} \cdot l_{\text{b}} \cdot \rho_{\text{b}} + 2 \cdot A_{\text{FL}fg} \cdot l_{fg} \cdot \rho_{\text{FL}fg} \]
\[ M_{11, \text{Planestrain}} = \frac{M_{11}}{L_2} \]

\[ M_{11, \text{Planestrain}} = 1.3748625 \times 10^7 \text{kg} \]

\[ m_{\text{FLAC}} = 1.3748625 \times 10^7 \text{kg} \]

Material Properties for FLAC, based on above values:

\[ l = 0.568539326 \text{m} \]

\[ l_c = \left( \frac{1}{12} \right) \text{unit}^3 \]

\[ A_c = 0.015314411 \text{m}^2 \]

\[ l_b = \left( \frac{1}{12} \right) \text{unit}^3 \]

\[ A_b = 0.015314411 \text{m}^2 \]

\[ l_{tg} = \left( \frac{1}{12} \right) \text{unit} (b_{tg})^3 \]

\[ A_{tg} = 0.046792969 \text{m}^2 \]

\[ p_b = 1.230894022 \times 10^4 \text{ kg/m}^3 \]

\[ p_{col} = 1.375 \times 10^3 \text{ kg/m}^3 \]

\[ F_{FL, \text{tg}} = 6.541666667 \times 10^7 \text{ kg/m}^3 \]

Configuration 3: Calculations for 1-Story

Contact Stresses

\[ M_{11} = 1.51234875 \times 10^6 \text{kg} \]

\[ \sigma_{11} := \frac{M_{11}}{4 \times (4.4m \times 4.4m)} \]

\[ \sigma_{11} = 1.915169792 \times 10^5 \text{Pa} \]

\[ m_{\text{FLAC}} = 1.3748625 \times 10^7 \text{kg} \]

\[ \delta_{y1} := \frac{m_{\text{FLAC}}}{2 \times (1m \times 4.4m)} \]

\[ \sigma_{11} = 1.532135834 \times 10^5 \text{Pa} \]

Yield moment, plane-strain condition

\[ P_{\text{nom, fuse}2} = \frac{47.520575}{5.5m} \text{ N m} 55^3 \]

\[ P_{\text{nom, fuse}1} = \frac{70.7964975}{5.5m} \text{ N m} 55^3 \]

\[ P_{\text{nom, fuse}2} = 1.437497394 \times 10^6 \text{N} \]

\[ P_{\text{nom, fuse}1} = 2.141594049 \times 10^6 \text{N} \]

\[ P_{\text{nom, Uncut}} = \frac{90.106858}{5.5m} \text{ N m} 55^3 \]

\[ P_{\text{nom, Uncut}} = \frac{32.013}{5.5m} \text{ N m} 55^3 \]

\[ P_{\text{nom, Uncut}} = 2.725732455 \times 10^6 \text{N} \]

\[ P_{\text{nom, Uncut}} = 968.39325 \text{kN} \]
% Spectral response of fixed-base fundamental period, based on
% acceleration time history output

clear
fid=fopen('f_dt.txt');
C=textscan(fid,'%n%n','headerlines',4,'delimiter','/t');
n_cyc=C{1};
time=C{2};
close(fid);

fid=fopen('Top_xacc.txt');
C=textscan(fid,'%n%n','headerlines',4,'delimiter','/t');
n_cyc=C{1};
Node11_xacc=C{2};
close(fid);
T = time(11)-time(10)
Fs = 1/T
L = length(Node11_xacc)

figure(1)
plot(time,Node11_xacc,'--gr','LineWidth',2)
xlabel('Time (s)'),ylabel('Acceleration (m/s2)')

figure(2)
NFfT = 2^nextpow2(L); % Next power of 2 from length of acc-time history
Y = fft(Node11_xacc,NfFt)/L;
f = (Fs/2)*linspace(0,1,NfFt/2+1);
plot(f,2*abs(Y(1:NfFt/2+1)));% Next power of 2 from length of acc-time history
title('Single-Sided Amplitude Spectrum of y(t)')
xlabel('Frequency (Hz)')
ylabel('|Y(f)|')
B.3 Test-1 (HBM02): MS1F_SF80 (COM) Responses
This section presents the acceleration time histories from each ground motion for Test-1 (HBM02), MS1F_SF80. Each plot is titled (with ground motion name). The figures include: (1) pseudo-spectral responses at the center of mass (COM) of MS1-1 (MS1F_SF80) for two FLAC configurations and the recorded CGM data, the acceleration time histories used to compute each of the spectral traces.
Figure B-1: Test-1, MS1-1: Joshua Tree, low intensity (JOS_L1), center of roof mass (COM)
Chi Chi, 1999 (TCU078)
Motion 1

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>1</td>
<td>2.05%</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure B-2: Test-1, MS1-1: Chi Chi, low intensity (TCU_L), center of roof mass (COM)
Northridge, 1994 (Sylmar Converter Station: SCSL, Motion 1)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>( \beta )</th>
<th>SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>1</td>
<td>4.84%</td>
<td>Low</td>
<td></td>
</tr>
</tbody>
</table>

Figure B-3: Test-1, MS1-1: Sylmar Converter Station, low intensity (SCS_L1), center of roof mass (COM)
Northridge, 1994 (Rinaldi Recording Station: RRS)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>β, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>1</td>
<td>8.75%</td>
<td>High</td>
</tr>
</tbody>
</table>

Figure B-4: Test-1, MS1-1: Rinaldi Recording Station, low intensity RRS), center of roof mass (COM)
Figure B-5: Test-1, MS1-1: Landers (LCN), high-intensity motion, center of roof mass (COM)
Northridge, 1994 (Joshua Tree: JOSL2, Motion 2)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>2</td>
<td>3.14%</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure B-6: Test-1, MS1-1: Joshua Tree (JOSL2), low-intensity motion, center of roof mass (COM)
Northridge, 1994 (Sylmar Converter Station: SCSL2, Motion 2)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>2</td>
<td>2.89%</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure B-7: Test-1, MS1-1: Sylmar Converter Station (SCSL2), low-intensity motion, center of roof mass (COM)
Loma Prieta, 1989 (Saratoga West Valley College, WVCL)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>β, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>2</td>
<td>3.31%</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure B- 8: Test-1, MS1-1: Saratoga West Valley College (WVCL), low-intensity motion, center of roof mass (COM)
Figure B-9: Test-1, MS1-1: Sylmar Converter Station (SCS_H), high-intensity motion, center of roof mass (COM)
Northridge, 1994 (Newhall West Pico: WPI_L)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near Fault</td>
<td>2</td>
<td>7.28%</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure B-10: Test-1, MS1-1: Newhall West Pico (WPI_L), low-intensity motion, center of roof mass (COM)
Figure B-11: Test-1, MS1-1: Joshua Tree (JOS_H), high-intensity motion, center of roof mass (COM)
Northridge, 1994 (Joshua Tree: JOSL3, Motion 3)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>$\beta$, SSI</th>
<th>Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>3</td>
<td>2.71%</td>
<td>Low</td>
<td></td>
</tr>
</tbody>
</table>

Figure B-12: Test-1, MS1-1: Joshua Tree (JOSL3), low-intensity motion, center of roof mass (COM)
Chi Chi, 1999 (TCU-78:TCUH)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Config</th>
<th>SSI Damping</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>3</td>
<td>3.36%</td>
<td>High</td>
</tr>
</tbody>
</table>

Figure B-13: Test-1, MS1-1: Chi Chi (TCU_H), high-intensity motion, center of roof mass (COM)
B.4 Test-1 (HBM02): MS3-1 (MS3F_B) Roof Responses

This section presents the acceleration time histories from each ground motion for Test-1 (HBM02), MS3-1 (MS3F_B). Each plot is titled (with ground motion name). The figures are presented in the following order (from top): (1) COM responses, (2) free-field responses, and (3) base input. Data for each of these locations is presented from (1) the centrifuge (“CGM” legend designator), FLAC configuration with applied structural Rayleigh damping (“T=2.69, Damp” legend designator), and a FLAC undamped configuration (“T=2.69”, legend designator).

Figure B-14: Test-1, MS3-1: Joshua Tree (low-intensity) roof (COM), free-field, and base responses
Figure B- 15: Test-1, MS3-1: Rinadli Recording Station roof (COM), free-field, and base responses
Figure B- 16: Test-1, MS3-1: Parachute Test Station roof (COM), free-field, and base responses
Figure B-17: Test-1, MS3-1: Sylmar Converter Station (low-intensity) roof (COM), free-field, and base responses
Figure B-18: Test-1, MS3-1: Lucerne (COM), free-field, and base responses
Figure B-19: Test-1, MS3-1: Joshua Tree (low-intensity, #2) roof (COM), free-field, and base responses
Figure B-20: Test-1, MS3-1: Sylmar Converter Station (high intensity) roof (COM), free-field, and base responses
Figure B-21: Test-1, MS3-1: Joshua Tree (high intensity) roof (COM), free-field, and base responses
Figure B- 22, MS3-1: Test-1: West Pico (low intensity) roof (COM), free-field, and base responses
Figure B-23: Test-1, MS3-1: Joshua Tree (low intensity, #3) roof (COM), free-field, and base responses
Figure B-24, MS3-1: Test-1: West Pico (high intensity) roof (COM), free-field, and base responses
Figure B-25: Test-1, MS3-1: Port Island, modified, roof (COM), free-field, and base responses
Figure B-26: Test-1, MS3-1: Chi Chi (high intensity) roof (COM), free-field, and base responses
B.5 Test-2 (HBM03): Acceleration time histories of base input, free-field, MS1-2 (MS1F_SF80 COM), and MS3-2 (MS3F_B COM)

This section presents the acceleration time histories associated with each ground motion for Test-2 (HBM03) for MS1-2 (MS1F_SF80) and MS3-2 (MS3F_B). Each plot is titled (with ground motion name). The figures are presented in the following order (from top): (1) base input, (2) free-field responses, (3) MS1F_SF80 COM and (4) MS3F_B COM. Horizontal acceleration time histories for each of these locations are presented from (1) the centrifuge (“CGM” legend designator) and (2) a FLAC undamped configuration (“FLAC” legend designator). Note that no structural Rayleigh damping was applied to any of these configurations.

Figure B-27: Test-2: Joshua Tree (low intensity, #1) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-28: Test-2: Joshua Tree (low intensity, #2) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-29: Test-2: Rinaldi Recording Station base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-30: Test-2: Rinaldi Recording Station (#2) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-31: Test-2: Chi Chi (low intensity) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-32: Test-2: Chi Chi (low intensity, #2) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-33: Test-2: Parachute Test Site, base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-34: Test-2: Parachute Test Site (#2), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-35: Test-2: Sylmar Converter Station, base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-36: Test-2: Sylmar Converter Station (#2), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-37: Test-2: Lucerne, base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-38: Test-2: Lucerne (#2), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-39: Test-2: West Valley College (low-intensity), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-40: Test-2: West Valley Converter (high intensity), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B- 41: Test-2: Sylmar Converter Station (high intensity) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-42: Test-2: Joshua Tree (high intensity), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-43: Test-2: West Pico, base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-44: Test-2: Port Island (modified), base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Figure B-45: Test-2: Chi Chi (high intensity) base input, free-field responses, MS1-2 (MS1F_SF80 COM) roof, and MS3-2 (MS3F_B COM) roof.
Appendix C

C.1 Prototypical 5-Story Structure Responses (LAM5)

Dynamic responses of a prototypical 5-story structure (LAM5, Ganuza 2006) are presented below for three ground motions. Each of these analyses were completed in FLAC with no structural Rayleigh damping. Results of the remaining motions (LCN and SCSL) may be found in Chapter 6 of this dissertation.
Landers, 1992
(Joshua Tree: JOS_L)

<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure C-1: Joshua Tree (low intensity), LAM5 comparison: SSI (blue) ($\lambda_{structure} = 0\%$)
<table>
<thead>
<tr>
<th>Type of Motion</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near-fault</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure C-2: Saratoga West Valley College: LAM5 comparison: SSI (blue) ($\lambda_{structure} = 0\%$)
Figure C-3: Rinaldi Recording Station (RRS): LAM5 comparison: SSI (blue) ($\lambda_{structure} = 0\%$)