University of Nevada, Reno

Slab Vibration and Horizontal-Vertical Coupling in the Seismic Response of Irregular Base-Isolated and Conventional Buildings

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil and Environmental Engineering

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Abstract

Slab Vibration and Horizontal-Vertical Coupling in the Seismic Response of Irregular Base-Isolated and Conventional Buildings

Recent developments in seismic performance objectives that allow superior structural designs have highlighted the importance of providing continued functionality to nonstructural components and systems (NCSs). Modern isolation techniques have been successfully implemented in practice, as well as tested at both component and system levels, but testing of large scale isolated building models subjected to combined horizontal and vertical motions has been limited. Considerable evidence of slab vibration amplification has been reported in both fixed and isolated buildings, and studies have concluded that the response of floor slabs is sensitive to the slab vibration properties. Field evidence that supports this claim is inconclusive, as strong vertical excitation has not been reported in instrumented seismically-isolated buildings.

The experiments considered in this study were conducted on a full-scale, five-story, steel moment-frame building subjected to a number of 2D (horizontal only) and 3D (combined horizontal and vertical) strong earthquake records using the world’s largest shake table at E-Defense. The types of building configurations tested included (1) triple friction pendulum bearings (TPB), (2) a hybrid combination of lead-rubber bearings and cross-linear bearings (LRB/CLB), and (3) conventional (fixed at the base).

The datasets corresponding to each building configuration have been permanently archived as separate standalone experiments under NEES TIPS/Project No. 571, and are currently publicly accessible through the DesignSafe-CI (cyberinfrastructure) as part of the Natural Hazards Engineering Research Infrastructure (NHERI). The archiving,
organization and documentation of this comprehensive dataset (about 211 GB), are described in detail. These unique, publicly accessible datasets are the backbone of this study.

Follow up investigations focused on the specimen isolated with a hybrid combination of lead-rubber and cross-linear bearings (LRB/CLB), and fixed at the base. The seismic response of the buildings was investigated with particular emphasis on slab vibration amplifications in response to vertical excitation and a horizontal-vertical (H-V) coupling effect observed in both buildings. The H-V coupling appeared as a significant amplification of horizontal floor accelerations observed during 3D shaking compared to 2D, and it was partially attributed to the strong asymmetry of the building that was enhanced by supplemental mass placed at the roof to represent equipment or a roof penthouse. The experimentally observed slab accelerations and the H-V coupling effect were accurately simulated through a 3D model of the specimen using standard software and modeling assumptions. These assumptions included the use of the insertion point method with end joint offsets to represent composite behavior of the floor system model consisting of frame elements for beams/girders and shell elements for floor slabs, as well as adequate distribution of floor masses through refined discretization. H-V coupled modes were positively identified through modal analysis, and verified with evaluation of floor spectral peaks.

A supplemental study focused on the evaluation of factors that may influence slab vibration and/or induce a horizontal-vertical (H-V) coupled response of buildings with mass irregularities. Parameters that influence the vertical response of the floor system and subsequent H-V coupling effect were investigated through computational simulations of a
3D numerical model of a hypothetical 3-story building both base isolated with lead-rubber bearings (LRBs) and conventionally configured. The parameters investigated included superimposed mass induced eccentricities, vertical acceleration intensity, vertical stiffness of the isolators, and slab design assumptions (i.e. slab stiffness/mass variations). Induced mass eccentricities were observed to influence the vertical and H-V coupling response, but had an unpredictable influence on slab acceleration amplifications. In addition, the direct implications of vertical slab vibrations and the H-V coupling behavior on the design forces of nonstructural components and systems (NCSs) was evaluated, and design modifications that account for these effects are proposed for consideration.
To myself.

*Je pense, donc je suis*

*Cheers!*

---

To my grandparents,

*Fedora, Sergio, Dulce Milagros and Jorge*
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Chapter 1. Introduction

1.1 Motivation and Background

1.1.1 Response of Nonstructural Components and Systems (NCSs)

The elements of a building that are not part of the load carrying system, such as piping, partition walls, electrical systems, equipment, suspended ceilings, etc., are known as nonstructural components and systems (NCSs). These components provide the necessary operational and functional capabilities of the building, making them crucial to the seismic performance objectives of the facility. Ever since the 1971 San Fernando earthquake, the associated downtime due to damage to NCSs, particularly for essential facilities (e.g. hospitals, fire stations), has become an increasingly alarming topic within both the research and practicing engineering communities (Wasilewski 1998, Kircher 2003, Griffin 2006, OSHPD 2010). Field observations after recent major earthquake events have reported extensive damage to NCSs, resulting in threats to life safety, economic losses and functionality losses. For instance, after the 1994 Northridge earthquake, about $9 billion were reported in losses due to nonstructural damage alone as noted by Whittaker and Soong (2003). Moreover, out of all the hospitals that were partially or completely closed as a result of the 2010 Chile earthquake, 83% lost partial or complete functionality exclusively due to nonstructural damage (Miranda et al. 2012). Similarly, extensive damage to NCSs in different facilities was reported after the 2011 off the Pacific coast of Tohoku earthquake (Mizutani et al. 2012), and even in some cases, the loss of human life had been attributed to falling ceiling boards (Motosaka et al. 2012).
1.1.2 E-Defense Experiments

With the goal of promoting rapid adoption of base isolation systems in Japan and the U.S., a full-scale, five-story, steel moment-frame building was subjected to a number of bidirectional and bidirectional-plus-vertical ground motions using the world’s largest shake table. The test program was completed in August 2011 at Hyogo Earthquake Engineering Research Center (E-Defense) of Japan’s National Research Institute of Earth Science and Disaster Prevention (NIED). The project was jointly funded by the U.S. National Science Foundation (NSF) Network for Earthquake Engineering Simulation (NEES) research program and NIED, in collaboration with the NEES Tools to Facilitate Widespread Use of Isolation and Protective Systems (TIPS) project (TIPS 2013), and the NEES Grand Challenge project. The test bed specimen was shaken with two seismic isolation systems and in the fixed-base configuration. In total, the building structure was subjected to 41 sinusoidal and earthquake simulations including 13 distinct earthquake records.

The primary objective of the test program was to demonstrate, at full scale, the effectiveness of base isolation to protect the building structure, the nonstructural components, and the contents in very rare earthquakes. The nonstructural components included an integrated system of interior walls, suspended ceilings, fire-sprinkler piping, and a precast concrete cladding column cover, all of which were assembled using U.S. construction techniques. These nonstructural components, as well as two enclosed areas staged with contents, were located within the 4th and 5th stories of the building.

The two isolation systems were: 1) triple friction pendulum bearings (TPB), and 2) a hybrid combination of lead-rubber bearings and cross-linear bearings (LRB/CLB). Each
isolation system design was developed by the project team in conjunction with the bearing supplier to meet distinct objectives, and the two systems were not developed with the objective of being compared. The recently developed TPBs were selected for testing, because they provide very large displacement capacity and can be designed under the Performance Based Earthquake Engineering (PBEE) paradigm to target various levels of protection under multiple earthquake scenarios (Fenz and Constantinou, 2008; Morgan and Mahin, 2011). Given the history of investigation of elastomeric bearings for nuclear applications, the hybrid isolation system evolved from the need to verify the stability and load-carrying capacity of elastomeric bearings at displacements representative of extended design basis ground motions. The isolation systems were strategically selected to accommodate a relatively lightweight (relative to typical bearing design) and asymmetric superstructure in order to demonstrate the general ability to extend seismic resiliency to challenging configurations.

The earthquake excitations were selected to excite the isolation devices close to their physical limit of travel, along with stability considerations specific to the hybrid system. A final objective was to determine the influence of vertical excitations on the performance of seismically-isolated buildings. In all three support configurations, the building was subjected to a variety of 2D (horizontal only) and 3D (combined horizontal and vertical) excitations, some with large vertical components.

1.1.3 Lessons Learned from Shake-Table Tests and Supporting Studies on Buildings with NCSs

Effective seismic performance evaluation of NCSs is paramount to provide adequate seismic design. Recent NEES projects have focused on system-level response evaluation
of NCSs on full-scale shake table experiments, to better understand their performance under seismic excitation. Chen et al. (2016) & Pantoli et al. (2016) reported on a series of 1D shaking experiments on a full-scale, five-story reinforced concrete building, conducted at the University of California, San Diego (UCSD) NEES site (NEES@UCSD). The specimen was furnished with a broad variety of NCSs including complete and operable egress, mechanical and electrical systems, facades, and architectural layouts, allowing a more comprehensive representation of the dynamic environment of these systems and their interactions. Attained peak roof accelerations ranged from 0.08 to about 1 g, resulting in mostly minor damage to the ceiling-sprinkler piping systems, roof mounted equipment, and restrained contents. Soroushian et al. (2016) reported on a series of 1D shaking experiments on a full-scale two-story steel braced frame test bed conducted at the University of Nevada, Reno NEES site (NEES@UNR). The subassemblies in the 1D experiments conducted at NEES@UNR were subjected to realized peak floor accelerations ranging between 0.14 and 2.5 g, where a more severe extent of damage to these components was observed under higher acceleration demands (over 1 g), particularly in the ceiling-sprinkler piping systems. Soroushian et al. (2015) also reported on the 2D and 3D shaking experiments conducted at E-Defense described in Section 1.1.2. In both of these studies, full-scale partition-ceiling-sprinkler piping subassemblies were designed and tested in multiple configurations to understand the dynamic environment of these systems, as well as their interaction with the structural components when subjected to a variety of floor drifts and accelerations. The subassemblies installed at the fifth floor and roof were subjected to peak floor accelerations ranging between 0.12 and 1.22 g. In response to 2D motions, ceiling-piping systems reflected only slight to moderate damage, yet significant
damage was observed from 3D motions, particularly those with large vertical excitation (Soroushian et al. 2015). These observations from the 3D experiments at E-Defense suggest a strong influence of vertical input on the response of acceleration sensitive NCSs.

One of the most effective and well-known mitigation strategies implemented to protect both the structure and its NCSs from seismic demands is seismic isolation. Modern isolation techniques have been successfully implemented in practice, as well as tested at both component and system levels. Kasai et al. (2013), for instance, reports field evidence on the good performance of seismic protection technologies. The experiments conducted at NEES@UCSD and E-Defense also investigated the effect of base isolation on the seismic response of NCSs. Chen et al. (2016) observed increased damage in nonstructural components in the fixed-base building when compared to the building isolated with high damping rubber isolators, particularly for the higher amplitude 1D motions. In the E-Defense experiments, Ryan et al. (2015) noted that the attenuation of floor drifts and accelerations provided by the isolation system is of great benefit to the response of NCSs, particularly evident during 2D simulations, which never induced any damage. In both of these studies, the effectiveness of horizontal floor acceleration reductions provided by the isolation system were validated, as prior investigations have consistently proven the benefits of its implementation in contrast to conventional structures. Under 3D excitation, however, damage included falling of ceiling panels and subsequent weakening of the grid system members (with most of the damage located below the middle of the slabs), permanent twisting of sprinkler-piping armover drops, and increased damage to the ceiling system due to compression posts used in the lateral bracing (Soroushian et al. 2015). Damage mechanisms and extent of damage under 3D excitations were very similar for the
three configurations tested, suggesting the response of NCSs to vertical shaking is independent of building support conditions.

Aside from the E-Defense studies (Ryan et al. 2015, Soroushian et al. 2015), others have also noted the impact strong vertical ground excitation has on the vibration response of floor systems in both fixed-base and isolated structures (Papazoglou and Elnashai 1996, Bozorgnia et al. 1998, Furukawa et al. 2013). Considerable evidence of slab vibration amplification has been reported in both fixed and isolated buildings, showing vertical input can be a significant source of damage to NCSs and building contents. Furukawa et al. 2013 reported vertical acceleration amplifications from locations adjacent to columns (already amplified from the ground) to the center of slabs, by a factor of 1.8 to 2.6. The superstructure tested was arranged with a wide variety of medical appliances, furniture, service equipment, and NCSs. Significant disruption and damage to building contents and equipment was observed, but damage to suspended ceilings, piping and sprinkler systems, and partition walls was not observed. In summary, these studies have concluded that the slab response, and in turn the NCSs’ response, is sensitive to the slab vibration properties as well as the acceleration demands (e.g. large vertical acceleration relative to horizontal acceleration), rather than the type of configuration (i.e. isolated or fixed-base).

Field evidence that supports amplifications of slab vibrations in isolated buildings is inconclusive, as most reports from instrumented buildings do not mention vertical response, reported vertical excitation has not been considered to be strong, and instrumentation that can record slab vibrations is very limited. For instance, Kasai et al. 2013 never reported significant damage to NCSs. A peak vertical acceleration of 0.76 g was observed in a base-isolated building during the 2004 Niigata-ken Chuetsu earthquake
(Saito 2006), for which no damage was reported. In the 2011 Christchurch earthquake, free-field vertical accelerations of 0.6 g were recorded near the Christchurch Women’s Hospital (Bradley and Cubrinovski 2011), which was not instrumented, and major damage to NCSs was not observed. Data from three buildings discussed in Bozorgnia et al. 1998 indicated that peak vertical accelerations recorded at the center of slabs were amplified by factors of 2.7 to 6.4 relative to the ground. Besides these sparse examples, instrumented buildings’ few vertical accelerometers are generally located away from the center of floor slabs or they lack them entirely, which makes it difficult to corroborate experimental findings.

Another possible source of damage to NCSs may arise from a phenomenon recently observed during the E-Defense experiments. The behavior was validated by analysis, and identified as a horizontal-vertical (H-V) coupling effect between the horizontal and vertical response of the specimen tested, both in the fixed-base and isolated configurations (Ryan et al. 2013, Dao and Ryan 2014). This interaction between the horizontal and vertical response of the structure was observed to be strongest in the TPB configuration, yet it was evident in all three configurations as the horizontal floor accelerations were amplified during 3D excitations compared to 2D. The strong coupling evidence in the TPB configuration was shown to result from the friction mechanism in the bearings (Ryan and Dao 2015). In the past, analytical studies have reported an increase of several response parameters of structures isolated with friction bearings subjected to 3D excitation in comparison to 2D (Shakib and Fuladgar 2003, Panchal et al. 2010). The H-V coupling observed in the LRB/CLB and fixed-base configurations has not yet been accurately predicted and explained. Modal and computational response history analysis indicated
evidence of coupled horizontal-vertical modes that corroborated the test data. A mode with frequency of 10 Hz or $T = 0.1$ sec was positively identified and other modes may have contributed (Ryan et al. 2013). Politopoulos and Moussallam (2012) introduced this concept of modal coupling for base-isolated structures by means of a simple two-degree-of-freedom model, highlighting that the behavior is enhanced by the asymmetry of the superstructure. Findings were that the coupling in the E-Defense building was partially attributed to the strong asymmetry of the building (i.e. unequal bay widths, staircase located in a corner) and enhanced by asymmetrically configured supplemental mass at the roof to represent equipment or a roof penthouse. Thus, these findings raise the question of whether the behavior could arise in more realistically configured buildings.

Understanding the response of floor systems to vertical excitation and possible coupled response of the structure is necessary in order to achieve higher performance objectives where damage to NCSs must be avoided to provide continued functionality. Prior analytical studies have accurately predicted the horizontal floor accelerations and floor spectra in the E-Defense building (Ryan et al. 2013, Dao and Ryan 2014, Ryan and Dao 2015), but none have focused on replicating the vertical response. Higher acceleration amplifications at the roof slab than for the 5th floor slab (dominant slab vibration frequencies of about 7 and 10 Hz respectively), suggest that individual local vertical modes were activated at each floor level. Previous studies have verified single and multi-mode response of the slabs, however, the intensity of these local vertical modes has not been accurately predicted, and analysis has been limited to the 5th floor and roof slab responses. The modeling approach described in Ryan et al. (2013) considered nonlinear frame elements with composite sections using extensive calibrations to improve the simulation
model, although the superstructure was designed to remain elastic. In addition, composite sections with an effective slab width do not provide predictions of slab vibration at the middle of floor slabs, where acceleration amplifications are believed to be largest. The authors of this study hypothesize that improved predictions of the experimentally observed vertical and H-V coupled response can be achieved with standard modeling assumptions using common linear frame and shell elements and less rigorous modeling details.

1.1.4 Design of NCSs

Guidelines in current practice for determining seismic design forces of NCSs are presented in Chapter 13 of the ASCE7 code (ASCE 2016). Alternatively, the ICC-AC156 testing standard procedure (ICC, 2010) can be used to determine the seismic design forces of NCSs. This procedure defines design spectra for each direction, for acceptance criteria for components with fundamental frequencies $\geq 1.3$ Hz, where the vertical spectral acceleration is equal to two-thirds of the horizontal. The design equation for the NCS anchorage force includes a horizontal force amplification factor as a function of building height that can be interpreted as the amplification of horizontal floor accelerations relative to the ground. Soroushian et al. (2016) found the floor acceleration amplification factors recommended by the code (i.e. maximum of 3 at the roof level) overestimated those derived from the 1D experiment results. Regarding the vertical direction, Ryan et al. (2015) reported that vertical acceleration from the ground to the middle of floor slabs was amplified by an average factor of 3-6. The effect of slab flexibility on the vertical response of the floor system, and ultimately the NCSs, is inherently neglected in the code, which assumes a vertical acceleration amplification factor of one. An additional amplification factor in the design equation accounts for probable spectral amplification of the floor or
slab acceleration associated with the inherent flexibility of the NCS. Soroushian et al. (2015) found close correlation between the code component amplifications (i.e. maximum of 2.5) and the experimental results in the horizontal direction but not the vertical. Thus, it is imperative to investigate whether NCSs’ design equations accurately consider all possible amplification effects in both horizontal and vertical directions.

1.2 **Dissertation Organization and Objectives**

Following this introduction, three main chapters (Chapters 2-4) are presented, concluding with a final dissertation summary and overall conclusions chapter (Chapter 5). Each of the three main chapters (2-4) corresponds to a stand-alone journal article, and thus they can be read in any order. The paper in Chapter 2 was published in 2015, while the remaining two articles (Chapters 3 and 4) have been submitted for review and possible publication as of January 1, 2017. Each chapter indicates the corresponding publishing journal. The objectives for each chapter are discussed in turn:

The objectives of Chapter 2 are to describe the E-Defense experiments and the generated datasets, and to highlight the potential for re-use of this data by the earthquake engineering community. Each dataset includes unprocessed and derived data, metadata pertinent to the experiment, and supporting documentation to assist with interpretation of the data.

In Chapter 3, the H-V coupling effect and vertical response on the E-Defense LRB/CLB and fixed-base configurations are investigated through computational simulations of a SAP2000® model of each building. The objectives of the study described in this chapter are: (1) to demonstrate and explain H-V coupling in the LRB/CLB and fixed-base configurations, (2) to quantify slab vibration amplifications over floor height in
isolated and fixed-base buildings subjected to 3D excitation, (3) show that standard modeling techniques can predict H-V coupling, and (4) show that composite floor system vertical response can be predicted through simple modeling assumptions.

Motivated by the E-Defense experiment findings, Chapter 4 investigates the factors that increase slab vibrations and influence or induce an H-V coupled response, as well as the direct implications of vertical floor system response and the H-V coupling behavior on the response of NCSs. The H-V coupling effect on buildings with irregular or asymmetric mass distributions are investigated through computational simulations of a SAP2000® model of a hypothetical building base isolated with LRBs and fixed at the base (conventional). The objectives of this chapter are to: (1) evaluate the effect of building asymmetries on the H-V coupling through induced eccentricities in the form of superimposed mass, (2) evaluate parameters that influence the vertical response of the floor system, including vertical acceleration intensity, modeling assumptions of the isolators, and slab flexibility, and (3) evaluate current code provisions for design forces of NCSs in both horizontal and vertical directions.
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Chapter 2. **DATA PAPER: Data from the Network for Earthquake Engineering Simulation/E-Defense Collaborative Test Program on Innovative Isolation Systems and Nonstructural Components**

*Note. This chapter is a stand-alone paper published in 2015 by Earthquake Spectra, Vol. 31 No. 2, pp. 1195-1209.*

**Abstract**

The datasets generated from E-Defense shake table experiments of a full-scale, five-story building tested with two innovative isolation systems and in the fixed-base configuration are available in the Network for Earthquake Engineering Simulation (NEES) Project Warehouse. The experiments included an integrated subassemblage of partition walls, suspended ceilings, and sprinkler piping on the fourth and fifth floors of the building; a full-story pre-cast concrete cladding column cover; and enclosed areas staged with building contents. Totaling approximately 211 GB between three experiments, the data set includes isolator forces and displacements, floor accelerations and story drifts, local accelerations and relative displacements at strategic locations on the nonstructural components, as well as extensive videos and inspection photos. The project was a collaboration between U.S. researchers of the NEES Tools for Isolation and Protective Systems (TIPS) and NEES Nonstructural Grand Challenge projects and Japanese researchers from the National Research Institute of Earth Science and Disaster Prevention. Data set DOIs are 10.4231/D3X34MR7R, 10.4231/D3SB3WZ43, and 10.4231/D3NP1WJ3P. [DOI: 10.1193/083113EQS241]

**2.1 Introduction**

Base isolation is one of the most effective measures to protect building structures and their nonstructural components from earthquake ground motions. Development of modern
seismic isolation techniques started in the 1960’s in New Zealand (Skinner et al., 1993). Various forms of elastomeric bearings, such as natural rubber bearings, high damping rubber bearings, and lead-rubber bearings, have been commercially implemented since the 1980’s. The friction pendulum bearing is a newer base isolation device that was first developed in the late 1980’s (Zayas et al., 1987) and whose implementation to buildings started with a seismic retrofit project in 1994. Currently, a wide range of base isolation devices are commercially available and are being implemented in practice. However, wide acceptance of base isolation has not yet occurred, partly because their cost benefit is not well understood in the structural engineering community. Most system level testing of isolated buildings has been performed on reduced-scale bare-frame models that cannot simulate the effects of realistic floor system response or nonstructural components/contents. In addition, testing of isolated building models subjected to combined horizontal and vertical motions has been limited.

With the goal of promoting rapid adoption of base isolation systems in Japan and the U.S., a full-scale, five-story, steel moment-frame building was subjected to a number of bidirectional and bidirectional-plus-vertical ground motions using the world’s largest shake table. The test program was completed in August 2011 at Hyogo Earthquake Engineering Research Center (E-Defense) of Japan’s National Research Institute of Earth Science and Disaster Prevention (NIED). The project was jointly funded by the U.S. National Science Foundation (NSF) Network for Earthquake Engineering Simulation (NEES) research program and NIED. The testbed specimen was shaken with two seismic isolation systems and in the fixed-base configuration. In total, the building structure was
subjected to 41 sinusoidal and earthquake simulations including 13 distinct earthquake records.

The primary objective of the test program was to demonstrate, at full scale, the effectiveness of base isolation to protect the building structure, the nonstructural components, and the contents in very rare earthquakes. The nonstructural components included an integrated system of interior walls, suspended ceilings, fire-sprinkler piping, and a precast concrete cladding column cover, all of which were assembled using U.S. construction techniques. These nonstructural components, as well as two enclosed areas staged with contents, were located on the fourth and fifth floors of the building.

The two isolation systems were: 1) triple friction pendulum (TP) bearings, and 2) a hybrid combination of lead-rubber (LR) bearings and cross linear (CL) bearings. Each isolation system design was developed by the project team in conjunction with the bearing supplier to meet distinct objectives, and the two systems were not developed with the objective of being compared. The recently developed TP bearings were selected for testing, because they provide very large displacement capacity and can be designed under the Performance Based Earthquake Engineering (PBEE) paradigm to target various levels of protection under multiple earthquakes scenarios (Fenz and Constantinou, 2008; Morgan and Mahin, 2011). Given the history of investigation of elastomeric bearings for nuclear applications, the hybrid isolation system evolved from the need to verify the stability and load-carrying capacity of elastomeric bearings at displacements representative of extended design basis ground motions. The isolation systems were strategically selected to accommodate a relatively lightweight (relative to typical bearing design) and asymmetric
superstructure in order to demonstrate the general ability to extend seismic resiliency to challenging configurations.

The earthquake excitations were selected to excite the isolation devices close to their physical limit of travel, along with stability considerations specific to the hybrid system. A final objective was to determine the influence of vertical excitations on the performance of seismically-isolated buildings. In all three support configurations, the building was subjected to a variety of XY (horizontal only) and 3D excitations, some with large vertical components.

The objective of this paper is to describe the experiments and the generated datasets, and to highlight the potential for re-use of this data by the earthquake engineering community. The datasets generated from this investigation are permanently archived and publicly accessible in the NEES Project Warehouse under NEES TIPS/Project No. 571. The data and metadata corresponding to each building configuration have been archived as separate standalone experiments, as follows: Experiment 3 - TP bearing configuration (Ryan et al. 2013a), Experiment 4 – hybrid LR/CL bearing configuration (Ryan et al. 2013b), and Experiment 5 – fixed-base configuration (Ryan et al. 2013c). Each dataset includes unprocessed and derived data, metadata pertinent to the experiment, and supporting documentation to assist with interpretation of the data. The other experiments archived under Project No. 571 are independent of the full scale test program and thus are not described here.

2.2 Experimental Setup and Metadata

The testbed specimen was a 5-story, 2 bay x 2 bay steel moment frame building (Figure 2-1(a)) constructed for a previous test program, which had been designed and detailed
according to Japanese code and design practice (Kasai et al., 2010). The building specimen was approximately 16 m (52.5 ft) tall, and 10 by 12 m (32.8 by 39.3 ft) in plan with an asymmetric bay configuration in the long direction (Figures 2-1(b) and 2-1(c)). All primary beam-column connections were fully welded, restrained moment connections. The floor system consisted of reinforced concrete slabs cast on corrugated metal decking in floors 2-5 and cast on a flat steel deck on the roof. The concrete slabs were connected to primary beams by shear studs to provide composite behavior. For the NEES/E-Defense program, steel plates weighing 535 kN (120 kips) were placed on the roof in an irregular configuration to enhance the asymmetry of the building specimen. Including the additional roof mass and the participating mass at the base level, the superstructure weighed about 5,300 kN (1,200 kips). The natural period of the building in the fixed-base configuration was about 0.68 sec, determined by system identification from white noise shaking.

2.3 Isolation Systems

The design of the two isolation systems was influenced by the building’s light weight and asymmetry. The TP isolation system incorporated 9 identical TP bearings (Figure 2-2(a)), one beneath each column, as shown in the plan layout in Figure 2-3. The properties of these isolators were chosen to accommodate the largest near fault ground motions that could be replicated by the E-Defense shake table. The bearings were 1.4 m (4.6 ft) in diameter, with a displacement capacity of 1.13 m (3.7 ft) at a base shear coefficient $V/W = 0.275$, where $V =$ isolator shear force and $W =$ axial force due to building self-weight. Although the target peak displacement of the TP bearings was about 1 m (3.3 ft), a peak displacement of about 0.7 m (2.3 ft) was observed during testing.
The second isolation system, featuring 4 LR bearings (Figure 2-(b)), was designed to accommodate the extended design basis (DB) shaking at a representative nuclear power plant site with soft soil in the eastern U.S. For nuclear plant design, the DB earthquake shaking is associated with a Mean Annual Frequency of Exceedance (MAFE) of no less than $1 \times 10^{-4}$ (return period of 10,000 years) while the extended DB is associated with a MAFE of $1 \times 10^{-5}$ (a return period of 100,000 years). To provide the desired isolation period
shift and accommodate the overturning moments, the LR bearings were supplemented with 5 CL bearings (Figure 2-2(c)). The LR bearings were each 0.7 m (2.3 ft) in diameter with a 102 mm (4 in.) lead core and a shape factor $S = 29$. The CL bearings are essentially low friction sliders that share the vertical load with the LR bearings without increasing the stiffness or base shear of the isolation system. Furthermore, the CL bearings enhanced the system stability by allowing a redistribution of axial forces between LR and CL bearings, while also providing tension resistance. The placement of the LR and CL bearings is indicated in Figure 2-3.

Figure 2-3. Isolation system layout for the TP bearing and the LR/CL bearing experiments
Each TP and LR bearing was supported on an array of load cells assembled between two custom-made steel plates. These load cell assemblies, isolators and finally the testbed building were bolted to the shake table and to each other in sequence. The CL bearings were constructed on built-up I beams and therefore bolted directly to the shake table. The testbed building was constructed on column base spreader assemblies (Figure 2-1(a)), and bolted directly to the shake table for the fixed-base configuration.

2.4 Nonstructural Components

A partition-ceiling-sprinkler piping sub-assembly was designed and installed in a nearly identical configuration over the fourth and fifth floors of the building specimen, which were expected to experience the maximum floor accelerations (Soroushian et al., 2012). Each floor sub-assembly included approximately 90 m (300 ft) of partition walls, 83.6 m² (900 ft²) of suspended ceiling with lay-in tiles, and fire sprinkler piping with a riser pipe, a main run, and 3 sprinkler branch lines that included drops. The suspended ceiling system was designed per ASTM E580/E580M-11ae1 standards (ASTM, 2011). The grid was constructed using an exposed tee system with main runners and cross tees supported by suspension wires. The main runners and cross tees were attached to the wall molding using seismic clips with different attachment details on opposing boundaries. The piping system included one 76 mm (3 in.) diameter riser pipe, one 64 mm (2.5 in.) diameter main run and three (North-South) 32 mm (1.25 in.) and 25 mm (1 in.) diameter branch lines per floor. The branch lines incorporated straight drops, armover drops, and flexible hose drops. The piping system was supported by sway bracing at the ends of the main run and diagonal splay wires at the ends of the branch lines. The partition wall details were selected based on the most commonly used commercial and institutional partition walls. Full connection
detailing was provided for fourth floor partitions while slip track connection detailing (which allows vertical movement of the top track relative to the studs and gypsum board) was provided for fifth floor partitions. Full height partitions were approximately 2.7 m (9 ft) tall. Further details of the nonstructural components can be found in Soroushian et al. (2012).

Two areas enclosed by self-standing partial height partition walls were used to stage a hospital-themed and an office-themed room with a variety of furniture and other loose items (Figure 2-4). Both the hospital room at the fourth floor and the office room at the fifth floor were built on top of the concrete weight block highlighted and designated as ‘1’ on Figure 2-1(b).

![Figure 2-4. (a) Hospital room (fourth floor) and (b) office room (fifth floor).](image)

Two full-story precast concrete column cover panels were fabricated and tested to evaluate the effectiveness of slotted steel connections to allow inter-story drift. The return cover 3D shaped panel and flat panel were installed at the southwest corner between the fourth and fifth floors. The cladding panels, designed according to U.S. practice, represent one type of standard façade that are designed to accommodate drift through racking of individual panels. A vertical seismic joint was installed between the two panels. Further
details describing the design and testing of the cladding panels can be found in McMullin et al. (2012).

2.5 Accessing Metadata

Metadata conveying the details of the experimental setup can be accessed directly in the NEES Project Warehouse, Project No. 571, under the ‘Experiments’ tab of each experiment, as shown in Figure 2-5. The Experimental Report summarizes the test setup, instrumentation, test schedule, data organization and derived responses; and it should be consulted first by anyone contemplating reuse of the data. The ‘Drawings’ subfolder (See Figure 2-5), also filed under ‘Documentation’, includes selected drawings of the specimen and bearing setup (as applicable), sensor layout and the global coordinate system. Complete design and construction drawings are saved as pdf documents within the ‘Documentation’ section, as indicated in Figure 2-5. Other experimental setup metadata under ‘Documentation’ include applicable drawings and specifications for the testbed specimen (modified for each test configuration), isolation bearings, connection assembly, partition-ceiling-piping sub-assemblies, and cladding panel sub-assemblies. A description of the testing facility is included, since it is not part of the NEES network of experimental facilities. Relevant published papers and reports will be maintained in a ‘Papers’ subfolder of each experiment, under ‘Documentation’ (Figure 2-5).

Photos with descriptive names have been organized into self-explanatory subfolders within the ‘Documentation/Photos’ folder of each experiment (See Figure 2-5). Examples of photo subfolders include ‘Pretest Assembly’ and ‘Post Test Bearing Inspection’. For Experiment 3, a ‘Construction’ subfolder documents the construction and assembly process preceding the formal start of the series of experiments by about 2 months, and is
organized by date. Select videos related to the experimental setup are found in the ‘Documentation/Videos/Movies’ folder of each experiment (Figure 2-5). These videos include inspection of the TP bearings following their removal from the shake table (Exp. 3), scenes from transport and installation of the bearings and testbed building (Exp. 4), and documentation of repairs to the ceiling and piping systems preceding the simulation of the fixed-base configuration (Exp. 5).
2.6 Instrumentation and Data Collection

The response of the structure and nonstructural components was measured with 642, 482 and 387 channels of instrumentation for Experiment 3, Experiment 4, and Experiment 5, respectively. Base displacements, from which isolator lateral deformations were computed, were recorded using DTP-D-5KS wire potentiometers with a stroke limit of
±2.5 m for both isolation configurations. Vertical isolator displacements were measured by laser transducers in three corner bearings for Experiment 3 (TP bearing configuration) only. Forces in two lateral directions and the vertical direction were recorded by 7 or 9 tri-directional load cells, contained within the load cell assemblies described earlier, for each TP and LR bearing. Forces in the CL bearings (Exp. 4) were not measured.

For all three configurations, X and Y-direction story drifts were recorded at two locations (SE and NW quadrant of Figure 2-1(b)) on every floor by laser displacement transducers attached to a rigid vertical truss and directed to a reflecting plate hung from the beam of the floor above. Floor accelerations were recorded at each level by 3D accelerometers attached at the SE, NE and NW columns. Moreover, vertical slab vibrations were recorded in several of the floors. Strain gages were attached to the NE corner column throughout the height of the building. Accelerometers and displacement transducers were strategically located to measure responses of the nonstructural components including: accelerations of the ceiling grid members, ceiling panels, and ceiling braces; accelerations of the piping system and sprinkler heads; accelerations at the tops of partition walls; partition wall drifts; and relative displacements between the ceiling grid and the partition walls. Video cameras were placed to record movement of the following: the whole building from various viewpoints, the isolators (as applicable), the nonstructural components, and the staged contents. These videos have been archived for every trial and repetition, as noted in the following section, ‘Unprocessed Data’.

Aside from the metadata previously noted, also included in the ‘Documentation’ folder (Figure 2-5) are: instrumentation drawings for the structure, the partition-ceiling-piping sub-assemblies, and the cladding panel sub-assemblies (each in a separate file); and a
camera plan. Several of the cameras were fixed at permanent locations in the E-Defense laboratory and are not shown in the camera plan. In addition, photos of each installed sensor have been archived in the experiment ‘Photos/Instrumentation’ subfolder, also under ‘Documentation’ (Figure 2-5).

A sensor table can be accessed from both the ‘Experiments’ tab, under ‘Sensors’, and the ‘Documentation/Sensors’ subfolder (Figure 2-5); and it lists all the sensors and data channels used in the experiment. The data was routed simultaneously through a series of junction boxes that collected data from up to 64 channels, and the organization of data reflects this format. For each channel, the sensor table includes channel number, channel name, sensor label, sensor type, global coordinates, measurement unit, orientation, and instrument serial number. The naming convention for the channel name and channel number is described in Figure 2-6. The global coordinate system used for the location of each sensor reported in the sensor table is defined at the base of the southeast column, as shown in Figures 2-1(b) and 2-1(c). A supplementary drawing showing the coordinate system can be found in the experiment ‘Document/Drawings’ subfolder.

Channel Number Example:

Channel Name Example:

Figure 2-6. Channel number and name examples found in the experiment sensor table
Supplementary non-NEES compliant sensor tables, prepared and used by the project team during the test program, have been archived in each experiment ‘Documentation’ folder under the name of “Channels_Summary_(Experiment).xls”. These tables (spreadsheets) are an additional resource to understand the data organization. Each spreadsheet contains 3 individual sheets: “Table of Instruments” (summary of instruments used), “Sensor List” (alternative sensor table), and “Data Acquisition Chart”. Importantly, the “Data Acquisition Chart” sequentially lists the channels in each junction box by channel name. This quick reference chart is the most convenient way to identify the channels that are located in a particular unprocessed data file.

2.7 Unprocessed Data

The Unprocessed Data is located at the Trial/Repetition level of each experiment, and can be found in the ‘Unprocessed Data’ subfolder, under ‘Data’ as shown in Figure 2-5. For each Trial/Repetition, the Unprocessed Data includes sensor data, videos, and photos if applicable. For each repetition, the sensor data is organized into files corresponding to junction boxes, and each file contains data for up to 64 channels. The last two digits of the file name that precede .csv refer to the junction box number. The data was recorded at a sampling rate of 1000 Hz and output directly in engineering units. The first column of each data file stores the time, followed by data for junction box channels 1 to 64 in columns 2 to 65. There are 11, 10, and 9 junction box files for Experiment 3 (TP bearing configuration), Experiment 4 (hybrid LR/CL bearing configuration), and Experiment 5 (fixed-base configuration), respectively. The videos are found in the ‘Unprocessed Data/Videos/Movies’ subfolder and correspond to the individual cameras named in the camera plan, where SC, NC and EC indicate cameras recording the responses of the
structural system, the partition-ceiling-piping subassemblies, and the corner cladding panel assembly, respectively.

The simulation plan varied significantly for each experiment according to the independent objectives. All experiments included bidirectional as well as 3D recorded motions. The record selection for Experiment 3 (TP bearing configuration) was dominated by strong near-fault records, such as 1994 Northridge at Rinaldi Rec. Sta. (RRS) and Sylmar Sta., and 1995 Kobe at Takatori Sta. For Experiment 4 (hybrid LR/CL bearing configuration), the ground motions were dominated by simulated motions (seeded from recorded motions) that were developed to represent the site spectra at selected nuclear sites, and scaled up to meet the target displacement of the isolation system. Sine wave excitations were applied to both isolation configurations for bearing characterization. With limited tests available for Experiment 5 (fixed-base configuration), the records were selected to allow direct response comparisons with the isolation configurations. Three records were repeated in each configuration (1987 Superstition Hills at Westmorland, 2011 Tohoku at Iwanuma Sta., and 1994 Northridge at RRS), although the latter two records were applied at lower scale factors in Experiment 5 to minimize the risk of structural yielding. Unidirectional and 3D white noise excitations were applied throughout Experiment 5 for system identification.

The RRS record induced the largest recorded horizontal accelerations at the table level for each experiment, which were about 1.2g for Experiments 3, and 4 and 0.4g for Experiment 5. However, the motions that induced the largest displacement demands in the isolators were 1978 Tabas at Tabas Station for Experiment 3 (0.7 m or 28 in.) and the simulation at the Diablo Canyon site for Experiment 4 (0.55 m or 22 in.).
In a given experiment, any change in target motion or scale factor is characterized as a new trial, whereas a repeated test with the same target motion is characterized as a repetition. In summary, the data is stored in 21 trials for Experiment 3, 13 trials for Experiment 4, and 9 trials for Experiment 5. An overview of the trials and repetitions is given in Table 2-1. A comprehensive test log indicating the chronological sequence of applied motions, which is not necessarily reflected in the organization of trials and repetitions, is included in each Experimental Report and in Ryan et al. (2013d).

The nonstructural components and contents of the fourth and fifth floor were inspected after select Trial/Repetition sequences. Specifically, for Experiment 3 (TP bearings), the floors were inspected after Trial 5 and at the end of every test day (Trials 7, 14, 21). For Experiment 4 (hybrid isolation), the floors were inspected at the end of the two test days (Trial 8/Rep. 1 and Trial 2/Rep. 2). For Experiment 5 (fixed-base), the floors were inspected after each earthquake Trial/Repetition (Trials 4-8) (See Table 2-1). Technically, these inspections took place after the follow-up 3D white noise excitation, but the inspection photos have been associated with the preceding earthquake motion for which the shaking was stronger. All descriptively named inspection photos are located under the respective Trial/Repetition in the ‘Unprocessed Data/Photos’ subfolder, subcategorized into “Ceilings”, “Contents”, “Partition Walls 4” (fourth floor), “Partition Walls 5” (fifth floor) and “Piping”. With the exception of Exp. 3/Trial 5, after each inspection the following repairs were made as needed: contents were reset, dislodged and fallen ceiling panels were reset, deformed/buckled ceiling grid members were replaced, pipes were reinforced with ropes at the locations of broken pipe hangers. Additional measures were taken prior to the start of Experiment 5 to strengthen the ceiling system and reinforce the
piping, which are described in several video clips located in the ‘Documentation/Videos/Movies’ folder of Experiment 5. Despite these measures, cumulative damage to the nonstructural systems throughout the experiment sequence should be considered when evaluating the nonstructural response.
Table 2-1. Overview of the test program for each experiment as organized under the unprocessed data

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Date</th>
<th>Trial Number(s)</th>
<th>Repetition Number</th>
<th>Description / Inspections of Nonstructural Components</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 - TP</td>
<td>8/17/2011</td>
<td>1-2</td>
<td>1 (each)</td>
<td>Sine wave excitations</td>
</tr>
<tr>
<td>3 - TP</td>
<td>8/17/2011</td>
<td>3-7</td>
<td>1 (each)</td>
<td>Earthquake excitations/Inspections after Trial 5 &amp; 7</td>
</tr>
<tr>
<td>3 - TP</td>
<td>8/18/2011</td>
<td>8-14</td>
<td>1 (each)</td>
<td>Earthquake excitations/Inspections after Trial 14</td>
</tr>
<tr>
<td>3 - TP</td>
<td>8/19/2011</td>
<td>15-21</td>
<td>1 (each)</td>
<td>Earthquake excitations/Inspections after Trial 21</td>
</tr>
</tbody>
</table>

**Summary: 21 trials with 1 repetition each (21 simulations total)**

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Date</th>
<th>Trial Number(s)</th>
<th>Repetition Number</th>
<th>Description / Inspections of Nonstructural Components</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 - LR/CL</td>
<td>8/25/2011</td>
<td>1</td>
<td>1</td>
<td>Earthquake excitation</td>
</tr>
<tr>
<td>4 - LR/CL</td>
<td>8/25/2011</td>
<td>2</td>
<td>1</td>
<td>Sine wave excitation</td>
</tr>
<tr>
<td>4 - LR/CL</td>
<td>8/25/2011</td>
<td>3-8</td>
<td>1 (each)</td>
<td>Earthquake excitations/Inspections after Trial 8</td>
</tr>
<tr>
<td>4 - LR/CL</td>
<td>8/26/2011</td>
<td>9-13</td>
<td>1 (each)</td>
<td>Earthquake excitations</td>
</tr>
<tr>
<td>4 - LR/CL</td>
<td>8/26/2011</td>
<td>3</td>
<td>2</td>
<td>Earthquake excitation</td>
</tr>
<tr>
<td>4 - LR/CL</td>
<td>8/26/2011</td>
<td>2</td>
<td>2</td>
<td>Sine wave excitation/Inspections after this trial</td>
</tr>
</tbody>
</table>

**Summary: 13 trials with 2 repetitions for trials 2 and 3, and 1 repetition each for the rest (15 simulations total)**

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Date</th>
<th>Trial Number(s)</th>
<th>Repetition Number</th>
<th>Description / Inspections of Nonstructural Components</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>1</td>
<td>1</td>
<td>X-direction white noise excitation at start of tests</td>
</tr>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>2</td>
<td>1</td>
<td>Y-direction white noise excitation at start of tests</td>
</tr>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>3</td>
<td>10 total</td>
<td>3D-white noise excitations (before and after each earthquake excitation)</td>
</tr>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>4-8</td>
<td>1 (each)</td>
<td>Earthquake excitation/Inspections after each trial</td>
</tr>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>1</td>
<td>2</td>
<td>X-direction white noise excitation at end of tests</td>
</tr>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>2</td>
<td>2</td>
<td>Y-direction white noise excitation at end of tests</td>
</tr>
<tr>
<td>5 - FIXED</td>
<td>8/31/2011</td>
<td>9</td>
<td>1</td>
<td>Sine sweep/Inspections after this trial</td>
</tr>
</tbody>
</table>

**Summary: 9 trials with 2 repetitions for trials 1 and 2, 10 repetitions for trial 3, and 1 repetition each for the rest (20 simulations total)**
2.8 Derived Data

Relevant responses have been derived from multiple channels of the unprocessed data for each Trial/Repetition, except for the non-earthquake excitations of the fixed-base configuration (Exp. 5/Trials 1-3, 9). The derived responses, which can be found under ‘Data/Derived Data’ at each experiment Trial/Repetition level in the ‘Experiments’ tab (Figure 2-5), include bearing displacements and forces (Exp. 3 and 4 only), floor accelerations and story drifts at the geometric center of the building, and moments and axial forces in the northeast (NE) corner column (See Figure 2-1(b)). To derive the responses, all recorded data was filtered using a Low-Pass Butterworth filter with a cut off frequency of 25 Hz. These derived responses were not zeroed after every trial, and thus reflect accumulation of forces or displacements over the course of the experiment.

The horizontal displacements in each isolator were computed from the measured displacements in the wire pots, accounting for geometric effects as a result of the large displacement demand in the bearings. The X and Y-components of displacement were determined by subtracting the original coordinates of the isolators (from the beginning of the experiment) from the displaced coordinates. Vertical isolator displacements were only measured at the NE, NW, and SE located TP bearings (See Figure 2-3). For each bearing, two laser transducers were attached to frames that extended off the side of the building on either side of the bearing, and the total vertical displacement was computed as the average of the two channels. The laser transducers measured the total vertical movement, which is a combination of a) movement along the curved surface of the bearing, b) compression related displacement, and c) uplift. The X, Y and Z-displacements for each isolator are stored in files named “DlsisoX.csv”, where X is the corresponding location of the isolator
(e.g. NE). If the vertical displacement was not measured, this component is stored as a column of zeros.

For each TP and LR bearing (Exp. 3 and 4), the X, Y and Z-components of the recorded dynamic force from all load cells, which were zeroed prior to the start of each experiment, were summed to derive the corresponding dynamic reaction at the location of the load cells. This reaction was then modified by the inertia forces of the connection plate, recorded by accelerometers, and the bottom bearing plate, to derive the dynamic reaction at the isolator level. The dynamic reactions in the Z-direction were added to the static weights, determined by different methods for Exp. 3 and 4. These derived X, Y and Z-reactions for each isolator are stored in files named “RIsoX.csv”, where X is the corresponding location of the isolator (Table 2-2). Using the derived horizontal displacements and reaction forces discussed above, the force versus displacement or hysteresis loops from one of the trials is plotted for each LR bearing in the X-direction (Figure 2-7). The same derived data could be used to plot these bearing force-displacement relations in the Y-direction.

Column moments and internal forces in the NE column were derived from the strain gage data. Assuming linear-elastic response, these forces were obtained from the axial strains recorded at various section locations and then extrapolated to the column ends. For each Trial/Repetition, these forces are stored in a file named “ColumnEndForce.csv”. The channels within this file are listed in Table 2-2.
Table 2-2. Derived data files channel names

<table>
<thead>
<tr>
<th>Derived Data File</th>
<th>Channels</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>RIsoX.csv</td>
<td>Rx, Ry and Rz</td>
<td>X, Y and Z-reactions for each isolator</td>
</tr>
<tr>
<td>ColumnEndForce.csv</td>
<td>Mx(bottom), My(bottom), Nz(bottom), Mx(top), My(top), and Nz(top)</td>
<td>Moments Mx and My about the global X and Y-axes, and the axial force Nz, at the top and bottom of the story, respectively, for each of stories 1-5</td>
</tr>
<tr>
<td>FloorAcc.csv</td>
<td>TableAvg_X, TableAvg_Y, TableAvg_Z, 1F_Acc_X, 1F_Acc_Y and 1F_Acc_Z</td>
<td>Table accelerations and accelerations at the base (1st Floor) level (the pattern is repeated for floors 2-6, where 6 is the roof level)</td>
</tr>
<tr>
<td>FloorDrift.csv</td>
<td>1F_Drift_X and 1F_Drift_Y</td>
<td>Drifts across the first story (same pattern for stories 2-5)</td>
</tr>
</tbody>
</table>

![Figure 2-7. Sample force-displacement or hysteresis in X-Direction for each LR bearing for a simulated earthquake](image-url)
Floor accelerations at the geometric center of the building plan were derived for every floor level. The horizontal accelerations were determined by appropriate weighting of the measured accelerations at the SE, NW and NE corners of the floor near the columns (See Figure 2-1(b)). The vertical acceleration was computed by averaging the vertical acceleration recorded at these three corners, which represents the acceleration propagated through the columns and not in the middle of the floors. Accelerations at the geometric center of the shake table were computed as the average of accelerations measured by four tri-directional accelerometers installed at the SE, NE, NW and SW corners of the shake table. For each Trial/Repetition, the derived accelerations are stored in a file named “FloorAcc.csv” (Table 2-2).

Story drifts at the geometric center of the building plan were derived for every story. The story drifts in the X and Y-directions were interpolated from measured story drifts at locations SE and NW from the geometric center (Refer to Figure 2-1(b)). For every Trial/Repetition, the derived drifts are stored in a file named “FloorDrift.csv” (Table 2-2).

A more detailed explanation of the algorithms used to derive each response is included in Chapter 3 of the Experimental Report in the ‘Documentation’ folder of each experiment. Papers that provide a more thorough explanation of our interpretation of the results are also included in this folder. These papers discuss findings on the effectiveness of the isolation systems, horizontal and vertical response interaction, response of nonstructural components, and more.

2.9 Potential for Data Re-use and Concluding Remarks

The comprehensive dataset (about 211 GB) from the three experiment configurations that allows for direct comparison of the seismic response of a conventional lateral system
to seismic protective systems at a realistic scale and large seismic intensity has many opportunities for data re-use. First, researchers have access to extensive system-level dynamic data with which to develop and calibrate new component models. Examples include models for bearing response (force versus displacement), seismic-induced floor vibration, and various parts of the nonstructural components (e.g., the partition walls, ceilings, and piping alone or interacting together). This usage also extends to the structural system, which responded essentially in the linear elastic range but was influenced by the energy dissipation properties of the nonstructural components as well as the structural materials. Second, engineers may wish to compare the dynamic bearing data to the cyclic data generated during prototype and production tests, or assess the suitability of modeling assumptions and reliability of software programs to estimate bearing and structural responses for design. Finally, the dataset allows for easy visualization of protective system response for educational purposes.

In considering the potential for data re-use, note that the data set is unique with respect to its scale, integration of floor system effects, and nonstructural component/content response. Of the handful of full-scale multi-story building strong motion experiments conducted at E-Defense and elsewhere, this one, to our knowledge, incorporated the greatest variety of strong, multi-component (3D) earthquake records. This combination of factors means that a compelling story buried in the data is still being unearthed. By archiving and thoroughly documenting the data from these experiments in a reliable and reputable database (NEEShub), we have developed a resource that can pay dividends well beyond its initial use by the project team.
2.10 Acknowledgments

The authors acknowledge several individuals that contributed directly to the generation and archiving of the described datasets: Eiji Sato and Tomohiro Sasaki of E-Defense, Taichiro Okazaki of Hokkaido University, Nhan Dao of Ho Chi Minh City University of Architecture, and Siavash Soroushian and Camila Coria of University of Nevada, Reno. Funding for the experiments described in this paper was provided by the National Science Foundation through Grants No. CMMI-1113275 and CMMI-0721399, U.S. Nuclear Regulatory Commission through Contract NRC-HQ-11-C-04-0067, and National Institute of Earth Science and Disaster. Products were donated by Earthquake Protection Systems, Dynamic Isolation Systems, THK, Aseismic Devices Company, Hilti, USG Building Systems, Cemco Steel, Victaulic, and Tolco. The authors are grateful for this support.

2.11 References


Chapter 3. Computational Simulation of Slab Vibration and Horizontal-Vertical Coupling in a Full-Scale Test Bed Subjected to 3D Shaking at E-Defense

Note. This chapter is a stand-alone paper draft in preparation for Earthquake Engineering and Structural Dynamics

Abstract

This paper focuses on slab vibration amplifications and a horizontal-vertical (H-V) coupling effect observed in a full-scale 5-story moment frame test bed isolated with a hybrid combination of lead-rubber bearings and cross-linear bearings, and fixed at the base. Median slab vibration amplification factors observed over building height ranged from 2 at the 2nd floor to 7 at the roof. A significant amplification of horizontal floor accelerations was observed during 3D (combined horizontal and vertical) shaking compared to 2D (horizontal only) under comparable intensity input motions. The experimentally observed slab accelerations and the H-V coupling effect were accurately simulated through a 3D model of the specimen using standard software and modeling assumptions. The modeling assumptions included the use of the insertion point method with end joint offsets to represent composite behavior of the floor system model, which consisted of frame elements for beams/girders and shell elements for floor slabs, as well as adequate distribution of floor masses through refined discretization. The coupling behavior was partially attributed to the strong asymmetry of the building and enhanced by asymmetrically configured supplemental mass at the roof. H-V coupled modes were identified through modal analysis, and verified with evaluation of floor spectral peaks.

3.1 Introduction

The elements of a building that are not part of the load carrying system, such as piping, partition walls, electrical systems, equipment, suspended ceilings, etc., are known as
nonstructural components and systems (NCSs). These components provide the necessary operational and functional capabilities of the building, making them crucial to the seismic performance objectives of the facility. Ever since the 1971 San Fernando earthquake, the associated downtime due to damage to NCSs, particularly for essential facilities (e.g. hospitals, fire stations), has become an increasingly alarming topic within both the research and practicing engineering communities (Wasilewski 1998, Kircher 2003, Griffin 2006, OSHPD 2010). Field observations after recent major earthquake events have reported extensive damage to NCSs, resulting in threats to life safety, economic losses and functionality losses. For instance, after the 1994 Northridge earthquake, about $9 billion were reported in losses due to nonstructural damage alone as noted by Whittaker and Soong (2003). Moreover, out of all the hospitals that were partially or completely closed as a result of the 2010 Chile earthquake, 83% lost partial or complete functionality exclusively due to nonstructural damage (Miranda et al. 2012). Similarly, extensive damage to NCSs in different facilities was reported after the 2011 off the Pacific coast of Tohoku earthquake (Mizutani et al. 2012), and even in some cases, the loss of human life had been attributed to falling ceiling boards (Motosaka et al. 2012).

Effective seismic performance evaluation of NCSs is paramount to provide adequate seismic design. Recent Network for Earthquake Engineering Simulation (NEES) projects have focused on system-level response evaluation of NCSs on full-scale shake table experiments, to better understand their performance under seismic excitation. Chen et al. (2016) & Pantoli et al. (2016) reported on a series of 1D shaking experiments on a full-scale, five-story reinforced concrete building, conducted at the University of California, San Diego (UCSD) NEES site (NEES@UCSD). The specimen was furnished with a broad
variety of NCSs including complete and operable egress, mechanical and electrical systems, facades, and architectural layouts, allowing a more comprehensive representation of the dynamic environment of these systems and their interactions. Attained peak roof accelerations ranged from 0.08 to about 1 g, resulting in mostly minor damage to the ceiling-sprinkler piping systems, roof mounted equipment, and restrained contents. Soroushian et al. (2016) reported on a series of 1D shaking experiments on a full-scale two-story steel braced frame test bed conducted at the University of Nevada, Reno NEES site (NEES@UNR). Under the same project and scope, Soroushian et al. (2015) reported on a series of 2D (horizontal only) and 3D (combined horizontal and vertical) shaking experiments on a full-scale five-story steel moment frame test bed conducted at E-Defense, in collaboration with the NEES Tools to Facilitate Widespread Use of Isolation and Protective Systems (TIPS) project (TIPS 2013). In both of these studies, full-scale partition-ceiling-sprinkler piping subassemblies were designed and tested in multiple configurations to understand the dynamic environment of these systems, as well as their interaction with the structural components when subjected to a variety of floor drifts and accelerations. The subassemblies in the 1D experiments conducted at NEES@UNR were subjected to realized peak floor accelerations ranging between 0.14 and 2.5 g, where a more severe extent of damage to these components was observed under higher acceleration demands (over 1 g), particularly in the ceiling-sprinkler piping systems. Results from the E-Defense experiments provided further insight, where subassemblies installed at the 5th floor and roof were subjected to peak floor accelerations ranging between 0.12 and 1.22 g. In response to 2D motions, ceiling-piping systems reflected only slight to moderate damage, yet significant damage was observed from 3D motions, particularly those with
large vertical excitation (Soroushian et al. 2015). These observations from the 3D experiments at E-Defense suggest a strong influence of vertical input on the response of acceleration sensitive NCSs.

One of the most effective and well-known mitigation strategies implemented to protect both the structure and its NCSs from seismic demands is seismic isolation. Modern isolation techniques have been successfully implemented in practice, as well as tested at both component and system levels. Kasai et al. (2013), for instance, reports field evidence on the good performance of seismic protection technologies. The experiments conducted at NEES@UCSD and E-Defense also investigated the effect of base isolation on the seismic response of NCSs. Chen et al. (2016) observed increased damage in nonstructural components in the fixed-base building when compared to the isolated case (high damping rubber isolators were used), particularly for the higher amplitude 1D motions. At E-Defense, the types of building configurations tested included (1) triple friction pendulum bearings (TPB), (2) a hybrid combination of lead-rubber bearings (LRB) and cross-linear bearings (CLB), and (3) fixed at the base. Ryan et al. (2015) noted that the attenuation of floor drifts and accelerations provided by the isolation system is of great benefit to the response of NCSs, particularly evident during 2D simulations, which never induced any damage. In both of these studies, the effectiveness of horizontal floor acceleration reductions provided by the isolation system were validated, as prior investigations have consistently proven the benefits of its implementation in contrast to conventional structures. However, damage mechanisms and extent of damage under 3D excitations were very similar for the three configurations tested, suggesting the response of NCSs to vertical shaking is independent of building support conditions.
Several studies have also noted the impact strong vertical ground excitation has on the vibration response of floor systems in both fixed-base and isolated structures (Papazoglou and Elnashai 1996, Furukawa et al. 2013, Ryan et al. 2015, Soroushian et al. 2015). Considerable evidence of slab vibration amplification has been reported in both fixed and isolated buildings, showing vertical input can be a significant source of damage to NCSs and building contents. In summary, these studies have concluded that the slab response, and in turn the NCSs’ response, is sensitive to the slab vibration properties as well as the acceleration demands (e.g. large vertical acceleration relative to horizontal acceleration), rather than the type of configuration (i.e. isolated or fixed-base). Field evidence that supports this claim is inconclusive, as strong vertical excitation has not been reported in instrumented seismically isolated buildings.

Another possible source of damage to NCSs may arise from a phenomenon recently observed during the E-Defense experiments. The behavior was validated by analysis, and identified as a horizontal-vertical (H-V) coupling effect between the horizontal and vertical response of the specimen tested, both in the fixed-base and isolated configurations (Ryan et al. 2013, Dao and Ryan 2014). This interaction between the horizontal and vertical response of the structure was observed to be strongest in the TPB configuration, yet it was evident in all three configurations as the horizontal floor accelerations were amplified during 3D excitations compared to 2D. In the past, analytical studies have reported an increase of several response parameters of structures isolated with friction bearings subjected to 3D excitation in comparison to 2D (Shakib and Fuladgar 2003, Panchal et al. 2010). The strong coupling evidence in the TPB configuration was shown to result from the friction mechanism in the bearings (Ryan and Dao 2015). However, the H-V coupling
observed in the LRB/CLB and fixed-base configurations has not yet been accurately predicted and explained. Modal and computational response history analysis indicated evidence of coupled horizontal-vertical modes that validated the test data. A mode with frequency of 10 Hz or $T = 0.1$ sec was positively identified and other modes may have contributed (Ryan et al. 2013). The coupling behavior was partially attributed to the strong asymmetry of the building (i.e. unequal bay widths, staircase located in a corner) and enhanced by asymmetrically configured supplemental mass at the roof to represent equipment or a roof penthouse. The modeling approach described in Ryan et al. (2013) considered nonlinear frame elements with composite sections using extensive calibrations to improve the simulation model, although the superstructure was designed to remain elastic. The authors of this study believe improved predictions of the experimentally observed vertical and H-V coupled response can be achieved with standard software using common linear frame and shell elements.

Understanding the response of floor systems to vertical excitation and possible coupled response of the structure is necessary in order to achieve higher performance objectives where damage to NCSs must be avoided to provide continued functionality. Prior analytical studies have accurately predicted the horizontal floor accelerations and floor spectra in the E-Defense building (Ryan et al. 2013, Dao and Ryan 2014, Ryan and Dao 2015), but none have focused on replicating the vertical response. Higher acceleration amplifications at the roof slab than for the 5th floor slab (dominant slab vibration frequencies of about 7 and 10 Hz respectively), suggest that individual local vertical modes were activated at each floor level. Previous studies have verified single and multi-mode
response of the slabs, however, the intensity of these local vertical modes has not been accurately predicted, and analysis has been limited to the 5th floor and roof slab responses.

In this study, the H-V coupling effect and vertical response of the E-Defense LRB/CLB and fixed-base configurations are investigated through computational simulations of a SAP2000® model of each building. The objectives of this paper are to: (1) demonstrate and explain H-V coupling in the LRB/CLB and fixed-base configurations, (2) quantify slab vibration amplifications over floor height in isolated and fixed-base buildings subjected to 3D excitation, (3) show that standard modeling techniques can predict H-V coupling, and (4) show that composite floor system vertical response can be predicted through simple modeling assumptions.

3.2 Experimental Setup

3.2.1 Building Testbed

The test bed specimen used in the experimental program was a 5-story, 2 bay x 2 bay steel moment frame building (Figure 3-1(a)) which had been designed and detailed according to Japanese code and design practice (Kasai et al. 2010). The building specimen was approximately 16 m (52.5 ft) tall, and 10 by 12 m (32.8 by 39.3 ft) in plan with an asymmetric bay configuration in the long direction. Figure 3-1(b), (c) show the basic dimensions of the building and the assumed coordinate system for presentation of results, where the Z-axis is the vertical axis.

The columns were made of 35 cm x 35 cm (13.8 in x 13.8 in) hollow box sections with thickness varying from story to story. To connect the testbed to the shaking table and provide the stiffness required to “fix” the testbed at its base, column bases and grade girders were designed with special details. The column bases were detailed as steel boxes with
dimensions of 2.5 m x 2.5 m x 0.9 m (8.2 ft x 8.2 ft x 3 ft) (Figure 3-2(a)). Vertical stiffeners were installed inside the boxes (Figure 3-2(b)). The grade girders were bolted to the column bases and were the same height as the column bases (0.9 m or 3 ft). The foundation framing was braced in its plane as shown in Figure 3-2(c). The beams were either rolled or built-up I-sections. The primary beams consisted of a 3-segment bolted splice with a small-section segment at the middle and two large-section segments at the ends that were haunched (Figure 3-3(a)). All primary beam-column connections were fully welded, restrained moment connections.

Figure 3-1. (a) 5-story steel moment frame specimen, (b) typical plan view and relevant instrumentation from floors 2 to 5, and (c) elevation view
Reinforced concrete slabs were cast on corrugated steel decking in floors 2-5. The steel decking, seen in Figure 3-3(a), (b), had 75 mm (3 in) deep, 1.2 mm (0.05 in) thick corrugations and was oriented parallel to the y-direction; 80 mm (3.1 in) thick normal weight concrete was cast over the ribs of the deck. Typical reinforcement in the floor slabs was a single layer of \( \phi 10 \) mm @ 150 mm (\( \phi 0.4 \) in @ 5.9 in) rebar in both directions placed at the slab mid-thickness. The roof slabs were 150 mm (5.9 in) thick normal weight concrete cast on 1.2 mm (0.05 in) thick flat steel deck. Reinforcement for the roof slab included a layer of \( \phi 13 \) mm @ 200 mm (\( \phi 0.5 \) in @ 7.9 in) rebar in each direction at the top and bottom of the slab. The roof slab was nearly twice as thick as the floor slabs as it
was designed to carry roof-mounted equipment (e.g., air conditioning system, water tanks) and a penthouse. The concrete slabs were connected to primary beams by shear studs to provide composite behavior (Figure 3-3(a)). The measured compressive strength of standard samples of the normal weight concrete was 24 MPa (3.5 ksi).

3.2.2 Weights

The distribution of weight (including NCSs) on each floor is estimated in Figure 3-1(c). In addition to the weight of structural components and NCSs, additional mass in the form of concrete and steel blocks were installed to simulate permanent live load. Typical size and position of the concrete weights, which were built as a permanent part of the structure on floors 2 to 5 are shown in Figure 3-1(b). Steel plates weighing 535 kN (120 kips) were placed on the roof in an irregular configuration to enhance the asymmetry of the building specimen (Figure 3-4(a)). Each steel block included either 7 or 8 steel plates that were 2.1 m x 4.3 m x 0.025 m (6.9 ft x 14.1 ft x 0.08 ft). Including the additional roof mass and the participating mass at the base level, the superstructure weighed about 5,300 kN (1,200 kips).
Figure 3-3. (a) Beam, beam to column connection and floor deck, (b) floors 2-5 composite floor section with corrugated deck slab dimensions

Figure 3-4. (a) Location of steel weights at roof, (b) lead-rubber bearing, (c) cross-linear bearing
3.2.3 Isolation System

The design of the hybrid isolation system was influenced by the building’s light weight and asymmetry. The isolation system, which featured 4 LRBs (Figure 3-4(b)), was designed to accommodate the extended design basis (DB) shaking for a nuclear power plant at a representative soft soil site in the eastern U.S. For nuclear plant design, the extended DB earthquake shaking is associated with a return period of 100,000 years. To provide the desired isolation period shift and accommodate the overturning moments, the LRBs were supplemented with 5 CLBs (Figure 3-4(c)). The LRBs were each 0.7 m (2.3 ft) in diameter with a 102 mm (4 in.) lead core and a shape factor $S = 29$. The CLBs are low friction rolling bearings that share the vertical load with the LRBs without increasing the system base shear, and enhance stability at large displacements.

The 4 LRBs were placed beneath the edge columns, and the 5 CLBs beneath the center and corner columns. Nominal stiffness and strength parameters were provided by the manufacturers, in addition to the geometric properties. The LRBs and CLBs were designed with an estimated displacement capacity of 600 mm (23.6 in). Nominal parameters that defined the bilinear force-deformation relation of the LRBs in the horizontal direction include characteristic yield strength, $Q = 65.7$ kN (14.8 kips), post-yield stiffness, $K_d = 0.65$ kN/mm (3.7 kips/in), and yield displacement, $D_y = 11.3$ mm (0.44 in). The effective and post-yield isolation period of the LRBs were $T_M = 2.27$ sec (at a displacement of 300 mm or 11.8 in) and $T_d = 2.62$ s, respectively. In the vertical direction, the compressive stiffness of the LRBs, $K_v = 1,500$ kN/mm (8,566 kips/in). Nominal design properties of the CLBs include coefficient of friction, $\mu = 0.48\% - 0.62\%$, $D_y = 0.1$ mm (0.004 in), and $K_v =$
3,471 kN/m (19,821 kips/in). More detailed information on the design basis of the isolation system can be found in Ryan et al. (2013).

### 3.2.4 Ground Motion Selection

The earthquake excitations in this experiment were selected primarily to excite the isolation devices close to their physical limit of travel, along with stability considerations specific to the hybrid system. A final objective was to determine the influence of vertical excitations on the performance of seismically-isolated buildings.

Target spectra representing both the U.S. and Japan seismicity design practices were developed for selecting the ground motions. In accordance with the beyond design basis target, records representing the seismic hazard at the Vogtle (Central and Eastern U.S.) and Diablo Canyon (Western U.S.) sites were selected to produce LRB displacements close to their capacity. Vogtle (‘VOG’) and Diablo (‘DIA’) records were selected to demonstrate the capabilities of base isolation for safety related nuclear structures. Consistency of the LRB response was evaluated by repeating an early trial at the end of the test program (Ryan et al. 2013). Furthermore, the 1994 Northridge earthquake recorded at Rinaldi Receiving Station (‘RRS’), a motion with strong vertical excitation, was repeated as a 3D and 2D excitation in each building configuration to better comprehend the response of isolated and non-isolated buildings to strong vertical excitation.

Additional constraints on the test program were based on imposed safety limits by the test facility, which included limiting isolator displacements ≤ 550 mm (1.8 ft) for the LRB/CLB system and structural drift limits ≤ 1.2% for all configurations. A suitable MCE level earthquake that met the safety limitations for the LRB/CLB and fixed-base configurations could not be identified. Hence, ground motions representing a service level
and a design level were included, but the MCE comparison was removed from the test program. The Westmorland (‘WSM’) record from the 1987 Superstition Hills earthquake was selected as the service level event. The 1940 Imperial Valley earthquake, recorded at El Centro station (‘ELC’) was the selected design event, but was not applied to the fixed-base configuration due to limited facility time. In addition, Japan side collaborators requested that a large motion recorded during a Japan earthquake be commonly applied to each configuration. The Iwanuma (‘IWA’) record from the 2011 Tohoku earthquake was the selected Japan motion. IWA as well as RRS were applied to the fixed-base building at reduced scale factors in the horizontal direction.

The final test schedule included 2 days of shaking (15 simulations) for the LRB/CLB configuration, and 1 day of shaking (5 simulations in addition to white noise and sine sweep) for the fixed-base. Additional details on the ground motion selection and scaling, the complete test schedule, and a comparison of peak accelerations of the target motions and the realized shake-table motions can be found in Ryan et al. (2013).

Representative input excitations have been selected out of the test program for the results presented in this study. The excitations listed next are 3D input unless otherwise noted. For the LRB/CLB configuration, the records and corresponding amplitude scale factors applied to the original records include ELC 130%, VOG 100%, VOG 175%, DIA 95% (2D), and DIA 80%. For the fixed base configuration, the selected records are WSM 80%, RRS 35% (2D), RRS 35%, and RRS 35%(XY) 88%(Z).

### 3.3 Instrumentation and Data Processing

The response of the structure and nonstructural components was measured with 482 and 387 channels of instrumentation for the LRB/CLB and fixed-base configurations,
respectively. Response quantities relevant to the scope of this paper include peak floor and peak slab accelerations, as well as floor and slab spectral accelerations, for every floor level. Figure 3-1(b) shows the typical accelerometer layout for floors 2-6. All recorded data was filtered using a Low-Pass Butterworth filter with a cut-off frequency of 50 Hz. Floor (horizontal) accelerations were recorded at each level by triaxial accelerometers attached at the NE, NW and SE columns. The peak floor accelerations and floor spectral accelerations in each direction were determined from a single floor acceleration history obtained by averaging the recorded histories at these column locations in each direction. Slab (vertical) accelerations were recorded by vertical accelerometers attached at the geometric center of the NE, NW, and SE slabs underneath the floor deck, with the exception of the 5th floor NW and SE, as well as all roof vertical accelerometers, which were attached at the top surface of the deck. Peak slab accelerations and slab spectral accelerations were obtained directly from the recorded histories at the aforementioned instruments (Figure 3-1(b)). In addition to the floors, accelerations at the geometric center of the shake table were computed as the average of accelerations measured by four triaxial accelerometers installed at the NE, NW, SE and SW corners of the shake table.

3.4 Numerical Model of Specimen

A 3D frame model of the test bed structure was developed in SAP2000® (CSI 2014) (Figure 3-5(a)), in order to validate the experimental results through computational response history analysis. The model was developed considering linear behavior of the superstructure (secondary effects, e.g. P-Delta, were neglected) and nonlinear link elements for the LRBs and CLBs. Fast nonlinear analysis (FNA), which can be effectively utilized when nonlinear behavior is limited to a small number of link elements (CSI, 2014),
was selected to increase computational efficiency. All beams and columns were modeled with frame elements connected at end joints, and slabs were explicitly modeled as shell elements. Panel zones at the beam-column interface were not modeled, and end (length) offsets in the connection region were not considered, with the exception of end offsets of base column nodes to account for rigidity of the base level floor diaphragm. Steel and concrete materials were defined for the frame and shell elements, respectively. The compressive strength of concrete, which determined its elastic modulus, was specified as 21 MPa (3 ksi). The modulus of elasticity of concrete was adjusted to reflect the experimental results. The slab reinforcement was not modeled.

Moment restrained connections were assumed at all beam-column joints on every floor level. At the base frame, these fixed connections provide the adequate rigidity required for the isolation system. Secondary gravity beams, and members representing the staircase frame were modeled with moment releases at both ends. In the fixed-base configuration, gravity columns were pinned at the base. The haunch regions at the ends of the primary beams were approximated with a uniform section representing the outer dimensions of the haunch (Figure 3-5(b)). Column bases detailed as steel boxes with vertical stiffeners (Figure 3-2(b)) were modeled explicitly using section designer in SAP2000®.

The shell elements were defined with a thickness equal to the concrete thickness above the ribs of the deck (80 mm or 3.15 in) plus half the thickness of the concrete inside the ribs (37.5 mm or 1.48 in), totaling 117.5 mm (4.6 in) (Figure 3-5(c)-(d)). The insertion point and end joint offset method was selected to best represent composite floor behavior in the models. The insertion point defines the local axis along the length of frame members through a specified point on the cross section identified as a cardinal point in SAP2000®.
The top center point was selected as the insertion point for all gravity beams (Figure 3-5(d)). In addition, an end joint offset on the frame members equal to half the thickness of the shell element or 58.75 mm (2.3 in) was applied, to align the bottom of the shell element with the insertion point location. Area loads with associated mass were directly applied to shell elements to account for superimposed steel deck and nonstructural element loads, as well as for the supplemental steel plates at the roof. Frame loads with associated mass were directly applied to corresponding perimeter frame elements to account for the exterior cladding seen in Figure 3-1(a).

Concrete blocks installed on the floors to simulate permanent live load were modeled using layered shell elements combined with thick shell elements. Shaded areas shown in Figure 3-6(a) represent a 50 mm (1.97 in) thick Styrofoam layer present between the floor
slab and the concrete blocks (Figure 3-6(b)). In the model, the shaded areas were modeled with layered shell elements with a gap layer applied at the location of the Styrofoam, while the rest of the block was modeled as a thick shell element. The shell element mesh shown in Figure 3-6(c) was finely discretized to distribute floor masses over the floors more realistically.

The in-plane stiffness of the floor slabs was accounted for through the application of diaphragm constraints. In theory, all nodes at a given floor should be “slaved” to each other in plane to achieve the desired behavior; however, this constraint also affects the out-of-
plane behavior, and makes the floor system too stiff. A reasonable performance was achieved by reducing the number of constrained nodes to those of the primary beams only, as shown in Figure 3-6(c).

Damping ratios were specified in each mode of the numerical model to represent energy dissipation in the testbed building. For both the LRB/CLB and fixed-base models, a constant modal damping of 2.5% was found to give a good match between experimental and numerical results. However, average damping overrides of 6% and 4% were applied to the first and second (longest period) vertical modes, respectively, to calibrate the numerical model to the observed slab vertical response in the LRB/CLB configuration. The effect of these calibrations is discussed later in more detail.

Isolators were modeled as independent link elements, one beneath each column. The rubber isolator and friction isolator link elements were applied to the LRBs and CLBs, respectively. For the CLBs, the coefficient of friction was taken as 0.55%. For the LRBs, using the bilinear parameters \((K_d, Q_d)\) obtained from the tests conducted by the manufacturer to represent the bearing hysteresis loop did not provide sufficient accuracy in the numerical model for the range of displacement amplitudes observed in the test program. Ryan et al. (2013) opted for characterizing the bilinear parameters independently for each simulation in the test program to obtain higher accuracy in the numerical predictions of the isolator response (i.e. peak isolator displacements and forces). In addition, reducing the best fit \(Q_d\) by 25% was found to improve the displacement predictions across all simulations, hence this was applied to the final parameters. The characterized parameters \(Q_d, K_d,\) and \(D_y\) for each LRB and further details on the characterization procedure can be found in Ryan et al. (2013). Given the availability of
these characterized parameters, response history analysis for every simulation was performed considering these calibrations.

3.5 Modal Analysis

Mode shapes and frequencies of the numerical models were determined by modal analysis. For the LRB/CLB configuration, an effective stiffness (corresponding to $T_d = 2.62$ sec) was assigned to the isolator models based on the nominal parameters provided by the manufacturer.

Figures 3-7 and 3-8 illustrate select y-direction modes of the LRB/CLB and fixed-base numerical models, along with their associated frequencies and periods, respectively. In each figure, the (8) plots on the left represent horizontal structural modes, one of which is identified as a coupled mode, and the (8) plots on the right represent local vertical modes. Each mode shows the deflected shape of the testbed in 3D space, and the deflected shape of a single frame in the y-direction. The y-direction was selected due to several sources of stiffness and mass irregularity that were not present in the x-direction (e.g. unequal bay widths in y-direction, staircase at the SW corner of the building, and supplementary mass at the roof level on the east half of the building). The undeflected shape of the system is shown in gray lines for reference. The displacement at the base (isolation system) where applicable, and frame deflections can be seen in the plots.

As seen in Figure 3-7, mode 2 ($T = 2.68$ sec) represents the isolation mode that induces mainly shear deformation in the isolators, and almost no deflection in the superstructure. Mode 4 ($T = 0.34$ sec) represents the 1st structural mode, characterized by horizontal deflection in the isolation system, and linear deflection across the structural system with a single node (point of zero deformation) at approximately the 4th floor. Likewise, mode 7
represents the second structural mode, characterized by horizontal deflection of the structure with 2 nodes at approximately the 2nd and 5th floors, and some slab vibration across all floors. Mode 12 \((T = 0.11 \text{ sec})\) represents the 3rd structural mode, characterized by horizontal deflection with nodes at the 1st, 3rd and 5th stories mid-height, as well as slab vibration across all floors. The presence of considerable vertical slab vibration in mode 12 suggests an H-V coupled mode where the horizontal response of this mode could be amplified, particularly in ground motions with large vertical input. Modes 9, 14, 15 and 20 are local modes dominated by vertical vibration of individual floor slabs. Such modes are usually neglected when evaluating the dynamic characteristics of a building, but become more significant when the response to vertical excitation is evaluated. Modes 14, 15 and 20 also show some horizontal deflection of the structure. Mode 9 \((T = 0.14 \text{ sec})\) could be interpreted as the longest period vertical mode, characterized primarily by vertical slab vibration at the roof level at the location directly beneath the supplementary mass. Modes 14 \((T = 0.1 \text{ sec})\), 15 \((T = 0.08 \text{ sec})\) and 20 \((T = 0.08 \text{ sec})\) are local vertical modes dominated by slab vibration across all floors but strongest at the 5th floor and roof, 4th and 5th floor, and at the 2nd floor slabs, respectively.

From the fixed-base modal analysis (Figure 3-8), mode 1 \((T = 0.64 \text{ sec})\) represents the 1st structural mode, characterized by linear deflection across the structural system. The natural period of this mode in the test specimen was determined by system identification from white noise shaking as 0.69 sec (Ryan et al. 2013). In comparison to the system identification, this first mode response is slightly stiffer in the numerical model. Mode 4 \((T = 0.21 \text{ sec})\) represents the 2nd structural mode, characterized by deflection with 2 nodes at approximately the 1st and 5th floors. Mode 8 \((T = 0.12 \text{ sec})\), represents the 3rd structural
mode, with 3 nodes each at approximately the 1st, 3rd and roof. This mode also shows some slab vibration across most floors that could amplify the horizontal response of the structure (i.e. H-V coupled). Mode 15 ($T = 0.08$ sec) represents the 4th structural mode with considerable slab vibration, particularly at the 2nd and 3rd floors, suggesting also another H-V coupled mode. Modes 7, 11, 13, and 18 are local modes dominated by vertical vibration of individual slabs. Mode 7 ($T = 0.14$ sec) appears to be the longest period vertical mode, characterized primarily, as before, by vertical slab vibration at the roof level directly beneath the supplementary mass. Modes 11 ($T = 0.09$ sec), 13 ($T = 0.08$ sec), and 18 ($T = 0.08$ sec) are characterized by slab vibration across all floors, dominated by the 5th floor and roof, the 4th and 5th floor, and the 2nd floor slabs, respectively.

The vibration periods of all the vertical modes discussed (Figures 3-7 and 3-8) are in agreement with measured frequencies of various slabs reported in Ryan et al. (2015). Namely, the observed periods of slab vibration modes ranged from about 0.08 at the 2nd floor to 0.13 sec at the roof (7.7 to 12.5 Hz).
Figure 3-7. Select deflected mode shapes of the LRB/CLB model in 3-D, and one frame in y-direction, along with their associated frequencies and periods. The undeflected shape of the system is shown in gray lines.

- **Mode 2**: $f = 0.37$ Hz; $T = 2.68$ sec; Isolation mode
- **Mode 9**: $f = 6.98$ Hz; $T = 0.14$ sec
- **Mode 4**: $f = 2.92$ Hz; $T = 0.34$ sec; 1st structural mode
- **Mode 14**: $f = 10.47$ Hz; $T = 0.10$ sec
- **Mode 7**: $f = 5.99$ Hz; $T = 0.17$ sec; 2nd structural mode
- **Mode 15**: $f = 12.1$ Hz; $T = 0.08$ sec
- **Mode 12**: $f = 9.45$ Hz; $T = 0.11$ sec; 3rd structural (coupled) mode
- **Mode 20**: $f = 12.86$ Hz; $T = 0.08$ sec
Figure 3-8. Select deflected mode shapes of the fixed-base model in 3-D, and one frame in y-direction, along with their associated frequencies and periods. The undeflected shape of the system is shown in gray lines.

Mode 1: $f = 1.55$ Hz; $T = 0.64$ sec; 1\textsuperscript{st} structural mode

Mode 4: $f = 4.73$ Hz; $T = 0.21$ sec; 2\textsuperscript{nd} structural mode

Mode 8: $f = 8.67$ Hz; $T = 0.12$ sec; 3\textsuperscript{rd} structural mode

Mode 15: $f = 12.61$ Hz; $T = 0.08$ sec; 4\textsuperscript{th} structural (coupled) mode

Mode 7: $f = 7.04$ Hz; $T = 0.14$ sec

Mode 11: $f = 10.74$ Hz; $T = 0.09$ sec

Mode 13: $f = 12.13$ Hz; $T = 0.08$ sec

Mode 18: $f = 12.9$ Hz; $T = 0.08$ sec
3.6 Comparative Horizontal Response and H-V Coupling

3.6.1 Comparative Horizontal Response

In this section, the predicted structural horizontal responses of the LRB/CLB and fixed-base buildings are compared to the experimental results. Response quantities examined include floor and slab accelerations, as well as floor and slab spectral accelerations. The response of the isolation system is not discussed, but note that numerical displacements and forces in the LRBs were observed within 20% of the experimental results for all the simulations. The input acceleration histories for the numerical simulations discussed herein were those from the averaged filtered signals of the realized shake table accelerations. If of interest, other comparisons of the experimental and numerical simulation results for every excitation using other analysis software can be found in Ryan et al. (2013), and Dao and Ryan (2015).

The peak acceleration profile (peak acceleration vs. floor level) and normalized peak acceleration profile by input type in the y-direction for all earthquake simulations of the LRB/CLB and fixed-base configuration are shown in Figures 3-9 and 3-10, respectively. Each figure also compares test ((a) and (c)) against analysis ((b) and (d)) simulations. The normalized accelerations ((c) and (d)) are equivalent to amplification factors of the peak ground acceleration (PGA). Outliers are identified in Figure 3-9(a), which include on the low side, the service level motion WSM 80% (WSM80) in both horizontal directions (only y-dir. shown), and on the high side, RRS 88% (RRS88) in the y-direction as a result of the strong vertical input. With the exception of RRS88, larger peak ground accelerations led to greater attenuation of acceleration, as expected. Figure 3-9(c) indicates more clearly that the isolation system was very effective in attenuating the acceleration in the superstructure.
In addition, the presence of vertical excitation (i.e. 3D) motions results in a smaller decrease relative to the \( \text{PGA} \) when compared against 2D motions. With the same input, the analysis results (Figure 3-9(b), (d)) indicate the numerical model captures similar attenuation trends, with the exception of WSM80 (Figure 3-9(d)), where base and roof accelerations were overestimated. For the fixed-base configuration (Figure 3-10(a), (c)), the acceleration profiles in the \( y \)-direction were similar for all the excitations, characterized by an increase in acceleration over the height of the structure, with a maximum at the roof level. This trend is captured well by the numerical simulation (Figure 3-10(b), (d)). As observed in Figure 3-10, both 2D and 3D input produced comparable acceleration demands on the superstructure.

For a more direct comparison, the \( y \)-direction peak acceleration profiles are plotted for selected LRB/CLB representative motions in Figure 3-11, and for selected fixed-base representative motions in Figure 3-12. As observed in both figures, peak floor accelerations in the numerical model are quite accurate, generally within about 20% of the experimentally observed values. Less accuracy is observed for larger motions with 3D input (e.g. VOG175 and DIA80 in the hybrid configuration and RRS35(XY)-88(Z) in the fixed-base configuration), where roof peak accelerations in particular, are clearly underestimated. During DIA95 (2D), one of the largest motions applied to the hybrid configuration with \( \text{PGA} \) in the \( y \)-direction = 0.79g, the observed peak roof acceleration was 0.27g, which was a 66% decrease relative to \( \text{PGA} \). In comparison, the analysis predicted a 67% reduction (Figure 3-11). The acceleration profile for DIA95 (2D), indicates maxima at the base and the roof, suggesting participation of the first structural mode (i.e. mode 4), while, the profile for VOG175 indicates local minima at the 2\(^{\text{nd}}\) and 5\(^{\text{th}}\) floors, suggesting
participation of the second structural mode (i.e. mode 7). Other modes, however, may be participating. For the fixed-base configuration (Figure 3-12), the acceleration profiles in the y-direction are characterized by an increase in acceleration over height, with a maximum at the roof level, suggesting participation of the 1st structural mode (i.e. mode 1). In general, acceleration profile trends in both buildings are captured well by the numerical simulations.

Figure 3-9. Test vs. analysis (a)-(b) peak absolute and (c)-(d) normalized acceleration profile for all simulations in the LRB/CLB configuration in the y-direction, identified by input type.
Figure 3-10. Test vs. analysis (a)-(b) peak absolute and (c)-(d) normalized acceleration profile for all simulations in the fixed-base configuration in the y-direction, identified by input type.

Figure 3-11. Comparison of y-direction test and analysis peak acceleration profiles for: [L-R] ELC130, VOG175, DIA95 (2D), and DIA80.
Figure 3-12. Comparison of y-direction test and analysis peak acceleration profiles for: [L-R] WSM80, RRS35 (2D), RRS35, and RRS35(XY)-88(Z).

For further insight, the 5% damped y-direction floor spectra from the numerical and experimentally obtained acceleration histories are plotted for every floor for selected input motions in Figures 3-13 and 3-14, for the LRB/CLB and fixed-base configuration, respectively. In the DIA95 (2D) spectra (Figure 3-13(a)), which is easier to interpret due to lack of vertical input, three distinct peaks appear. A first peak appears on every floor at a period between 2 and 3 sec (0.33 to 0.5 Hz), which represents the isolation mode. Due to the nonlinearity of the isolation system, the period of this peak varies from record to record. A second peak appears on all floors except the 4th floor at a period between 0.3 and 0.4 sec (2.5 to 3.3 Hz), and represents the 1st horizontal structural mode in the y-direction. This mode (Figure 3-7, mode 4), has a node at the 4th floor. A third peak appears on all floors except the 2nd and 5th just below a period of 0.2 sec (5 Hz), and represents the 2nd structural mode in each direction (Figure 3-7, mode 7). These floor spectra plots confirm that participation of a 2nd structural mode in each direction (x-direction not shown), is non-
negligible in the acceleration response. The same first and second peaks in the DIA95 spectra, representing the isolation and first structural modes, also appear on the rest of the LRB/CLB sample motion spectra (Figure 3-13(b) and (c)). A third peak, representing the 2nd structural mode, can also be found in these spectra, but barely present in ELC130 (Figure 3-13(c)). A fourth peak, just above a period of 0.1 sec (10 Hz), and weakest at the 1st and 4th floors, is representative of the 3rd structural mode (Figure 3-7, mode 12), and appears in all floors in the y-direction in Figure 3-13(b), (c). The peaks of the numerically generated spectra generally occur at the same periods (or very close) to the spectra for the experimental data.
In the fixed-base spectra (Figure 3-14), the first peak, at a period of 0.7 sec (1.4 Hz) represents the first structural mode in each direction. This mode (Figure 3-8, mode 1) is characterized by increasing amplitude over the height. The second peak, just above a period of 0.2 sec (5 Hz), represents the second structural mode (Figure 3-8, mode 4), with nodes at the 1st and 5th floor. In the RRS35(YS)-88(Z) spectra (Figure 3-14(c)), a third and a fourth peak appear. The third peak, just above a period of 0.1 sec (10 Hz), represents the third structural mode (Figure 3-8, mode 8), with nodes at the 1st and 3rd floor. The fourth peak, at a period of 0.07 sec (14.3 Hz), appears stronger at the 2nd and 5th floor and represents the fourth structural mode, likely mode 15 (Figure 3-8). Since the third and
fourth structural modes appear in the RRS35(XY)-88(Z) and not in the RRS35, this confirms the third and fourth structural modes are H-V coupled. In other words, these modes are amplified as the vertical excitation is increased.

Figure 3-14. Test and analysis y-direction 5% damped spectral acceleration comparison for 2nd through 6th floors for (a) WSM80, (b) RRS35, and (c) RRS35(XY)-88(Z).

3.6.2 Horizontal-Vertical Coupling

As discussed in Section 3.6.1, horizontal floor accelerations were amplified for 3D records relative to 2D records. Part of this amplification was believed to be participation of H-V coupled modes (i.e. possibly the 2nd and 3rd horizontal structural modes in the LRB/CLB configuration, and the 3rd and 4th structural modes in the fixed-base) that were likely driven by the vertical acceleration, thus, the source of this amplification is best
understood by considering horizontal floor spectra. The experimentally recorded y-direction horizontal floor spectra are compared for DIA80 and DIA95 (2D) for the LRB/CLB configuration in Figure 3-15(a), and for RRS35 (2D), RRS35, and RRS35(XY)-88(Z) for the fixed base configuration in Figure 3-16(a). Note the difference in spectral amplitudes for the two figures. As seen in Figure 3-15(a), spectral acceleration peaks were amplified significantly for 3D relative to 2D at about 0.18 sec in the 1st, 3rd, 4th and 6th floors, which represents the 2nd structural mode (Figure 3-7, mode 7). Similarly, spectral peaks are also amplified for 3D relative to 2D at about 0.1 sec in all floors, with the smallest amplitudes at the 3rd and 4th floors, which represents the 3rd structural mode (Figure 3-7, mode 12). For the Rinaldi motions (Figure 3-16(a)), the y-direction spectral peaks were amplified much more strongly at 0.11 sec for RRS35(XY)-88(Z) relative to the other motions, suggesting participation of the 3rd structural mode (Figure 3-8, mode 8), and at 0.07 sec, with highest amplitudes at the 2nd and 5th floors, which suggests participation of the 4th structural mode (Figure 3-8, mode 15). The amplifications for RRS35 were more subtle in the y-direction, but followed a similar trend to RRS35(XY)-88(Z) (Figure 3-16(a)).

The numerically obtained horizontal floor spectra are compared for DIA80 relative to DIA95 (2D) for the LRB/CLB configuration in Figure 3-15(b), and for RRS35 (2D), RRS35, and RRS35(XY)-88(Z) for the fixed base configuration in Figure 3-16(b). As observed in Figure 3-15(b), the numerical model predicts amplification of the 3rd structural mode for 3D input relative to 2D ($T = 0.1$ sec), but not for the 2nd structural mode in the y-direction, where the spectral peak amplitudes are nearly the same for both input types. Figure 3-16(b), however, does show that the numerical model predicts amplification at both
periods of 0.11 sec and at 0.07 sec for 3D input relative to 2D in the fixed-base configuration.

The 3rd structural mode amplification ($T = 0.1$ sec) in the DIA80 motion, could have been driven by vertical excitation as considerable vertical slab vibration was observed on all floors for the y-direction mode (Figure 3-7, mode 12), particularly at the 3rd and 4th floors. To confirm which modes were being amplified, Figures 3-15(c) and 3-16(c) show numerically obtained spectra with increasing vertical acceleration intensity (0 to 100% of target) compared against the test simulation for DIA80 and RRS35(XY)-88(Z), respectively. The horizontal input was kept at 100% of the target for all the simulations. As observed in 3-15(c), only the 3rd structural mode shows increasing amplification with increasing vertical excitation, while other modes in both the x and y-direction remained essentially unchanged. This confirms that the 3rd structural mode in the LRB/CLB configuration (mode 12) is an H-V coupled mode, but not the 2nd structural mode (mode 7). Figure 3-16(c) shows increasing amplification with increasing vertical excitation at periods of around 0.11 sec and 0.07 sec, which correspond to the 3rd and 4th horizontal structural modes in the fixed-base configuration, previously identified in modal analysis.

As noted earlier, the atypical supplementary weight mounted at the roof level may have significantly influenced the H-V coupled response of the structure, and thus, its effect should be considered as an area of further exploration and refinement. Furthermore, successful replication of the response to combined horizontal and vertical excitations can confirm the possible sources of horizontal-vertical coupling and identify its implications, if any, on building configurations more representative of typical design practice.
Figure 3-15. Y-direction 5% damped spectral accelerations for 1st through 6th floors comparison for (a) DIA80 and DIA95 (2D) test, (b) analysis, and (c) DIA80 analysis with increasing vertical component intensity, and test simulation.
3.7 Vertical Floor System Vibration

Vertical accelerations in 3D simulations generated significant vertical vibration of the floor system for both isolated and fixed-base configurations. The substantial supplementary weight mounted at the roof level of the test bed structure is believed to have significantly influenced the response of the structure due to vertical excitations and the subsequent H-V coupled response of the structure previously discussed. To provide an overview of how vertical acceleration propagated from the shake table up through the building columns and into the floor slabs, peak vertical accelerations recorded in the shake table and middle of the floor slabs, or peak slab acceleration (PSA) for every 3D simulation.
are summarized for the NW and SE slab locations in Figure 3-17(a) and (d), respectively. Similarly, PSA normalized by the shake table vertical \( PGA \) (normalized \( PSAs \)) for the same locations are summarized in Figure 3-17(b) and (e), respectively. The normalized accelerations are equivalent to amplification factors of the vertical \( PGA \), and will be hereafter referred to as slab amplification factors. The simulations are numbered chronologically, where the first 9 correspond to the LRB/CLB configuration, and the last three to the fixed-base configuration. The last data point in every subplot, \( \theta \), corresponds to the median over all the simulations.

The trends for slab amplification factors were quite different for the NW and SE floor slabs (Figure 3-17(b) and (e)). The median slab amplification factor (across all floors) for the NW slabs was about 3 and did not increase much over the height of the building. The amplification factors for the SE slabs, on the other hand, were notably higher, and increased steadily over the height of the building (Figure 3-17(e)). The median amplification factor varied from about 3 on the 2\(^{nd} \) floor to about 6 on the roof. The differences in response of the NW and SE slabs appear to be related to period and damping trends identified in Ryan et al. (2015). For instance, the NW slab dynamic properties – lower period and larger damping ratios that did not vary much with height (period consistently near 0.07 sec) – were more favorable to vibration suppression, while the SE slab dynamic properties – period increasing and damping ratio decreasing with height (e.g., period increased from 0.07 sec at 2\(^{nd} \) floor to 0.13 sec at the roof) – closely corresponded to the intensification of acceleration amplification factors in the upper floors of the building. Figure 3-17(c) and (f) provide an overview of how the numerical model captured the general trends in the acceleration response in the slabs, showing slab amplification factors for every 3D
simulation at the NW and SE slab locations, respectively. As seen in these subfigures, the numerical model provides a good estimate of the median slab amplification factors at each floor, despite the limitations to predict PSA at certain floors for some of the simulations.

Figure 3-17. (a) Peak vertical acceleration in table and NW floor slab (test), (b) normalized NW slab acceleration (test), (c) normalized NW slab acceleration (analysis), (d) peak vertical acceleration in table and SE floor slab (test), (e) normalized SE slab acceleration (test), and (f) normalized SE slab acceleration (analysis); for each floor level and every 3D simulation in the NEES/E-Defense test.

For more direct comparisons, DIA80, VOG100, ELC130, (for the LRB/CLB configuration), WSM80, RRS35 and RRS35(XY)-88(Z) (for the fixed-base configuration) were selected as representative input excitations. As such, Figures 3-18 and 3-19 plot the 5% damped slab spectra, from the numerical and experimentally obtained acceleration histories, for the 2nd through the 6th floors, for the SE slab location, in each configuration, respectively. Due to the large range in peak spectral intensity, the y-axis scale in Figure 3-19(c) has been adjusted for each floor. As shown in Figures 3-18 and 3-19, many of the numerically and experimentally determined spectra were dominated by a single spectral peak, particularly at upper floors, whereas some spectral plots suggest a multi-mode
response of the slabs (e.g. Figure 3-18(a), 3\textsuperscript{rd} Floor). Single-mode slab vibration dominant frequencies are in agreement with previously identified localized vertical modes, i.e., around 7 Hz at the roof ($T = 0.14$ sec), around 10.5 Hz at the 5\textsuperscript{th} floor ($T = 0.1$ sec), and greater than 12 Hz at other locations ($T = 0.08$ sec). Overall, the numerical simulation model predicts, with enough accuracy, the amplitude of the peaks. The spectral peaks at the SE location, being of higher interest due to the supplementary roof weight location, were generally overestimated and underestimated at the 5\textsuperscript{th} and roof levels, respectively, for the LRB/CLB simulations. Prediction of these peaks was somewhat improved, but the problem not entirely eliminated, with the damping calibration.
Figure 3-18. Comparison of analysis and test simulation 5% damped slab spectral accelerations for the 2\textsuperscript{nd} through 6\textsuperscript{th} floors SE slab location for (a) DIA80, (b) VOG100, and (c) ELC130.
Figure 3-19. Comparison of analysis and test simulation 5% damped slab spectral accelerations for the 2nd through 6th floors SE slab location for (a) WSM80, (b) RRS35, and (c) RRS35(XY)-88(Z).
3.8 Conclusions

This paper has focused on investigating the seismic response of a 5-story moment frame building tested in a full-scale shake table experiment with a hybrid combination of lead-rubber bearings and cross-linear bearings (LRB/CLB), and fixed at the base. Motivated by achieving higher performance objectives, where damage to nonstructural components and systems (NCSs) must be avoided to provide continued functionality, slab vibration amplifications in response to vertical excitation and a horizontal-vertical (H-V) coupling effect observed in both buildings were identified as the areas in need of further investigation. The H-V coupling was demonstrated by a significant amplification of horizontal floor accelerations observed during 3D (combined horizontal and vertical) shaking compared to 2D (horizontal only). A 3D numerical model using industry standard software and modeling assumptions was developed to replicate the vertical response and H-V coupling effect in detail in both the LRB/CLB and fixed-base configurations. The numerical simulations supported the following conclusions in alignment with the objectives of this paper:

1. Peak floor accelerations in both buildings were captured well by the numerical simulation, with predictions generally within 20% of the experimental results for all the simulations in the test program. Reduced accuracy of the model was observed for motions with large vertical excitation.

2. Horizontal floor spectra were well matched by the numerical model, with differences mostly in amplitude of spectral peaks representing higher modes, primarily due to lack of precision in modeling damping within the superstructure.
3. Horizontal structural modes identified through modal analysis of both buildings were effectively matched to horizontal floor spectral peaks, including higher modes that appeared in the response only during 3D excitations, which suggested an H-V coupling behavior. The presence of each mode identified in the simulation provided a response comparable to the experimentally observed response in both buildings.

4. Higher modes driven by vertical excitation (i.e. H-V coupled) were confirmed by evaluation of floor spectra with increasing vertical acceleration intensity, while keeping the horizontal input at 100% of the target. A structural mode with a frequency of 9.45 Hz or $T = 0.11$ sec was positively identified as an H-V coupled mode in the LRB/CLB configuration. In the fixed-base, two structural modes with frequencies of 8.67 Hz ($T = 0.12$ sec) and 12.61 Hz ($T = 0.08$ sec) were also identified as H-V coupled. This confirms higher structural modes can be driven by vertical excitation in both isolated and fixed-base buildings.

5. Slab acceleration amplifications (i.e. vertical accelerations over height relative to peak ground acceleration) were also predicted with sufficient accuracy, with amplification factors ranging from 2 at the 2nd floor, to about 8 at the roof, validating the experimentally observed factors which ranged from 2 to 7. These amplifications were observed in both building configurations at slabs located directly beneath the supplementary roof mass.

6. Slab spectra obtained from the numerical simulations indicated most slab vibrations were dominated by a single-mode response, particularly at higher floors, which validated the experimental results. Localized vertical modes dominated by
vibration of individual slabs, identified in modal analysis, were in agreement with the dominant frequencies observed in the spectral response. Slab spectral peaks were accurately predicted at each floor, where evaluations favored the accuracy of predictions of floor slabs with dynamic properties that led to increased vibration (i.e. period increasing with height and damping ratio decreasing with height).

7. The numerical model predicted good estimates of experimentally observed peak floor and slab accelerations and spectra, as well as H-V coupling effects. Simple superstructure modeling assumptions used to effectively replicate these effects included a composite floor system model consisting of frame elements for beams/girders and shell elements for floor slabs, and the use of the insertion point method with end joint offsets to represent composite floor behavior. In addition, adequate discretization of the shell element mesh was necessary to distribute floor masses over the floors more realistically, and floor diaphragm constraints were limited to a few nodes. A constant modal damping of 2.5% was found to give a good match between experimental and numerical results. In the LRB/CLB configuration, relevant modal damping ratios were fine-tuned to match the experimental results at the 5th floor and roof slabs in particular. The assumptions demonstrated that slab vibration and H-V coupling can be predicted with standard modeling techniques without intensive refinement of details.

8. The vertical response in both buildings raises the question of the direct influence of significant mass eccentricities on the vertical response and subsequent H-V coupling effect. Thus, increased slab amplifications over building height may be particular to the atypical configuration of this test bed frame.
3.9 References


OSHPD (2010), *California Senate Health Committee Hearing on Hospital Seismic Safety*, The Office of Statewide Health Planning and Development, Sacramento, CA.


Chapter 4. Slab Vibration and Horizontal-Vertical Coupling in the Seismic Response of Irregular Base-Isolated and Conventional Buildings

Note. This chapter is a stand-alone paper draft in preparation for Journal of Earthquake Engineering

Abstract

This paper reports on the computational evaluation of factors that may influence slab vibration and/or induce a horizontal-vertical (H-V) coupled response of buildings with mass irregularities. Parameters that influence the vertical response of the floor system and subsequent H-V coupling effect are investigated through computational simulations of a 3D numerical model of a hypothetical 3-story building both base isolated with lead-rubber bearings (LRBs) and conventionally configured. While all the parameters evaluated that directly affect the vertical response show an unpredictable influence on slab acceleration amplifications over building height, induced mass eccentricities are observed to influence the vertical and H-V coupling response the most. H-V coupled modes are identified through modal analysis, and shown to influence the acceleration response through floor spectral peaks. In addition, the direct implications of vertical slab vibrations and the H-V coupling behavior on the design forces of nonstructural components and systems (NCSs) are evaluated, and modifications that account for these effects are proposed for consideration.

4.1 Introduction

Recent developments in modern seismic performance objectives that allow superior structural designs have highlighted the importance of providing continued functionality to the building elements known as nonstructural components and systems (NCSs). These elements, such as piping, partition walls, suspended ceilings, electrical systems,
equipment, etc., are not part of the load carrying system, yet they provide the necessary operational and functional capabilities of the facility. The 1971 San Fernando earthquake became the turning point for targeting building performance objectives that attempt to minimize or eliminate any associated downtime due to damage to NCSs, particularly for essential facilities (e.g. hospitals, fire stations). Field observations ever since have reported extensive damage to NCSs, resulting in threats to life safety, economic losses and functionality losses, propelling both the research and practicing engineering communities to take action and create awareness of this issue (Wasilewski 1998, Kircher 2003, Whittaker and Soong 2003, Foo and Lau 2004, Griffin 2006, OSHPD 2010, Miranda et al. 2012, Mizutani et al. 2012, Motosaka et al. 2012). An effective seismic mitigation measure that stemmed partly from these efforts is base isolation. This technology has slowly been adopted by the engineering community since the 1980s, to achieve improved building performance. Base isolation systems have been shown to provide superior protection of the structure, NCSs, and building contents.

Recent findings from full-scale shake table experiments have focused on system-level response evaluation of NCSs, taking designers a step closer to providing adequate seismic design of these elements. For instance, relevant experiments under the Network for Earthquake Engineering Simulation (NEES), include Chen et al. (2016), Pantoli et al. (2016), and Soroushian et al. (2015 and 2016), which have effectively evaluated common NCSs’ performance under 1D, 2D (horizontal only) and 3D (combined horizontal and vertical) seismic excitation. In addition, these experiments have also investigated the effect of base isolation on the seismic response of NCSs by testing the equipped building specimens in both isolated and conventional (fixed at the base) configurations. Key
observations from these series of experiments are summarized as follows. First, the horizontal floor acceleration reductions provided by the isolation systems were validated, and the benefit was reflected in limited damage to acceleration sensitive NCSs. Second, considerable evidence of slab vibration amplifications in the 3D experiments were reported in both the isolated and conventional buildings, suggesting the response of NCSs is strongly influenced by vertical shaking.

The impact of strong vertical excitation on the vibration response of floor systems in both isolated and conventional structures has been noted in past studies (Papazoglou and Elnashai 1996, Furukawa et al. 2013, Ryan et al. 2015, Soroushian et al. 2015). A relevant conclusion from these studies indicates that the NCSs’ response, is sensitive to the slab vibration properties as well as the acceleration demands (e.g. large vertical acceleration relative to horizontal), rather than the type of support conditions.

The shaking experiments described in Soroushian et al. (2015) were conducted on a full-scale five-story steel moment frame test bed at the E-Defense shake-table facility in Japan. The types of building configurations tested included (1) triple friction pendulum bearings (TPB), (2) a hybrid combination of lead-rubber bearings (LRB) and cross-linear bearings (CLB), and (3) conventional fixed-base. During these experiments, an interesting phenomenon was observed in all three configurations, when horizontal floor accelerations were amplified during 3D excitations compared to 2D. This behavior implies a potentially significant source of damage to NCSs, and it is particularly important in base isolated structures, where a higher performance is desired. In the past, analytical studies have reported an increase of several response parameters of structures isolated with friction bearings subjected to 3D excitation in comparison to 2D (Shakib and Fuladgar 2003,
Panchal et al. 2010). However, the effect had not been fully investigated and explained, nor experimentally validated. The behavior observed at E-Defense was identified by Ryan et al. (2013) and Dao and Ryan (2014) as a horizontal-vertical (H-V) coupling effect between the horizontal and vertical response of the building specimen, and provided analytical validation of the response. The coupling evidence was strongest in the TPB configuration, and it was shown to result from the friction mechanism in the bearings (Ryan and Dao 2015). In the other two configurations, the behavior was attributed to coupled horizontal-vertical modes, which was validated by modal and computational response history analysis in Guzman and Ryan (2017). Findings were that the coupling in the E-Defense building was partially attributed to the strong asymmetry of the building (i.e. unequal bay widths, staircase located in a corner) and enhanced by asymmetrically configured supplemental mass at the roof to represent equipment or a roof penthouse. This type of modal coupling in base-isolated buildings had also been identified in an analytical study by Politopoulos and Moussallam (2012) by means of a simple two-degree-of-freedom model, which highlighted that the coupling is enhanced by the asymmetry of the superstructure. Thus, these findings raise the question of whether the behavior observed in the E-Defense test bed could arise in more realistically configured buildings.

Guidelines in current practice for determining seismic design forces of NCSs are presented in Chapter 13 of the ASCE7 code (ASCE 2016). Alternatively, the ICC-AC156 testing standard procedure (ICC, 2010) can be used to determine the seismic design forces of NCSs. This procedure defines design spectra for each direction, for acceptance criteria for components with fundamental frequencies $\geq 1.3$ Hz, where the vertical spectral acceleration is equal to two-thirds of the horizontal. The design equation for the NCS
anchorage force includes a horizontal force amplification factor as a function of building height that can be interpreted as the amplification of horizontal floor accelerations relative to the ground. Soroushian et al. (2016) found the floor acceleration amplification factors recommended by the code (i.e. maximum of 3 at the roof level) overestimated those derived from the 1D experiment results. Regarding the vertical direction, Ryan et al. (2015) reported that vertical acceleration from the ground to the middle of floor slabs was amplified by an average factor of 3-6. Guzman and Ryan (2017) found analytical predictions of slab vibration amplifications to be of the same order as those observed experimentally. The effect of slab flexibility on the vertical response of the floor system, and ultimately the NCSs, is inherently neglected in the code, which assumes a vertical acceleration amplification factor of one. An additional amplification factor in the design equation accounts for probable spectral amplification of the floor or slab acceleration associated with the inherent flexibility of the NCS. Soroushian et al. (2015) found close correlation between the code component amplifications (i.e. maximum of 2.5) and the experimental results in the horizontal direction but not the vertical. Thus, it is imperative to investigate whether NCSs’ design equations accurately consider all possible amplification effects in both horizontal and vertical directions.

Motivated by the E-Defense experiment findings, this study will focus on investigating the factors that may influence or induce an H-V coupled response, as well as the direct implications of vertical floor system response and the H-V coupling behavior on the response of NCSs. The H-V coupling effect on buildings with irregular or asymmetric mass distributions are investigated through computational simulations of a SAP2000® model of a hypothetical building base isolated with LRBs and fixed at the base (conventional). The
objectives of this paper are to: (1) evaluate the effect of building asymmetries on the H-V coupling through induced eccentricities in the form of superimposed mass, (2) evaluate parameters that influence the vertical response of the floor system, including vertical acceleration intensity, modeling assumptions of the isolators, and slab flexibility, and (3) evaluate current code provisions for design force amplification factors of NCSs in both horizontal and vertical directions.

4.2 Building Design Assumptions

The building considered in this study is a hypothetical three-story steel braced-frame building designed by Forell/Elsesser Engineers, Inc. of San Francisco for loss estimation studies as part of the NEES TIPS project (TIPS 2013), and previously documented in Erduran et al. (2011). Designed to be minimally compliant to the 2006 International Building Code (IBC) (ICC 2006), ASCE 7-05 (ASCE 2005), and AISC 341-05 (AISC 2005), this office building (occupancy category II and importance factor $I = 1$) is assumed to be located near Los Angeles, CA (34.5 N, 118.2 W). Assuming stiff soil conditions (site class D with shear wave velocity $V_{s30} = 180$ to 360 m/s), the design spectral accelerations are $S_s = 2.2g$ (short period) and $S_1 = 0.74g$ (1.0 second period), where $g =$ acceleration due to gravity.

Designed and detailed as a special concentric braced frame (SCBF) [$R = 6$, drift ratio $\leq 2.5\%$], the lateral loads are carried by a single braced bay on each side of the building perimeter, while the remaining elements were designed to resist gravity loads only (Figure 4-1(a)) (Erduran et al., 2011). The characteristic yield strength of steel was assumed to be 345 MPa (50 ksi) for frame members and 318 MPa (46 ksi) for brace members. The building is 55 m $\times$ 36.6 m (180 ft $\times$ 120 ft) in plan, with story heights of 4.57 m (15 ft) and
column spacing of 9.15 m (30 ft) in each direction. Columns rest on 1.83 or 2.44 m (6 or 8 ft) square reinforced concrete (RC) footings, or 3.66 m (12 ft) square RC footings at the braced bay locations. Floor slabs are composite 82.55 mm (3.25 in) thick lightweight concrete over 50.8 mm (2 in) thick steel deck (Figure 4-3(a)). The total seismic mass considered from gravity loads includes the self-weight of frame members, self-weight and superimposed dead load of 3.11 to 3.21 x 10^{-3} MPa (65 to 67 psf) per floor, and 9.58 x 10^{-4} MPa (20 psf) for cladding around the building exterior. At the roof level, a superimposed dead load of 2.68 x 10^{-3} MPa (56 psf) was applied over a 18.3 m x 9.14 m (60 ft x 30 ft) region, assuming a penthouse configuration at that location (Figure 4-1(a)).

An additional detail of an ordinary concentric braced frame (OCBF) system [\( R = 1, \text{drift ratio} \leq 1.5\% \)] designed as an isolated building was also available and reported in Erduran et al. (2011). However, to facilitate direct comparison to the conventional building response behavior, the authors opted to consider an isolated version of the designed-as-fixed SCBF. For the isolated configuration, a base layer was added to the conventional building with the same base floor plan details as the OCBF. Table 4-1 lists the steel frame sections at the braced bay of both buildings.

The maximum displacement, \( D_M \), at the center of rigidity of the isolators in the Maximum Considered Earthquake (MCE) is computed as:

\[
D_M = \frac{g S_{M1} T_M}{4 \pi^2 B_M} \tag{4.1}
\]

where \( T_M \) is the effective isolation period; \( B_M \) is a damping modification factor; and \( S_{M1} \) is the 1-s spectral acceleration for the design event. Target values of \( T_M = 3.1 \) s and effective damping ratio \( \beta_M = 15\% \) were chosen for the MCE. The maximum (\( D_M \)) and total (\( D_{TM} \))
isolator displacement are reported in Table 4-2, where $D_{TM}$ accounts for displacement amplification due to accidental torsion (Equation 17.5-5 and 17.5-6 of ASCE 2005).

Table 4-1. Steel sections used in the buildings in U.S. customary units

<table>
<thead>
<tr>
<th>St. #</th>
<th>Column</th>
<th>Girder</th>
<th>Brace</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>1</td>
<td>W14 x 176</td>
<td>W27 x 84</td>
<td>HSS 12 x 12 x 5/8</td>
</tr>
<tr>
<td>2</td>
<td>W14 x 176</td>
<td>W30 x 99</td>
<td>HSS 10 x 10 x 5/8</td>
</tr>
<tr>
<td>3</td>
<td>W14 x 109</td>
<td>W36 x 150</td>
<td>HSS 8 x 8 x 1/2</td>
</tr>
</tbody>
</table>

Table 4-2. Characteristics of the isolation system

<table>
<thead>
<tr>
<th>System Properties</th>
<th>MCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Period</td>
<td>$T_M = 3.1$ s</td>
</tr>
<tr>
<td>Effective Damping</td>
<td>$\beta_M = 15%$</td>
</tr>
<tr>
<td>Isolator Displacement</td>
<td>$D_M = 633.7$ mm (24.95 in)</td>
</tr>
<tr>
<td>Total Displacement</td>
<td>$D_{TM} = 762$ mm (30.0 in)</td>
</tr>
</tbody>
</table>

Figure 4-1. (a) Building floor plan showing penthouse location, braced bays, diaphragm constraints and shell element discretization, and (b) braced bay elevation.
4.3 Modeling Assumptions and Details

3D models of both buildings were developed in SAP2000® (Figure 4-2). As an overview, the models were developed considering linear behavior of the superstructure (secondary effects, e.g. P-Delta, were neglected) and nonlinear link elements for the isolators. Fast nonlinear analysis (FNA), which can be effectively utilized when nonlinear behavior is limited to a small number of link elements (CSI, 2014), was selected to increase computational efficiency. All beams and columns were modeled with frame elements connected at end joints, and slabs were explicitly modeled as shell elements. Panel zones or gusset plate regions at the beam-column interface were not modeled, and end (length) offsets in the connection region were not considered. Steel and concrete materials were defined for the frame and shell elements, respectively. The compressive strength of concrete, which determined its elastic modulus, was specified as 24 MPa (3.5 ksi).

Figure 4-2. Analytical 3D models for the (a) conventional and (b) isolated buildings

For both building models, all gravity beams were modeled with moment releases at
both ends. Beam-column connections on braced bays were modeled as fixed connections to represent the stiffness of the gusset plates. In the conventional building, gravity columns were pinned at the base, with the exception of braced bays, where the base was fixed. In the isolated building, fixed connections were assumed at all base level beam-column joints, to provide the adequate rigidity required for the isolation system. The shell elements were defined with a thickness equal to the concrete thickness above the ribs of the deck (82.55 mm or 3.25 in) plus half the thickness of the concrete inside the ribs (25.4 mm or 1 in), totaling 107.95 mm (4.25 in) (See Figure 4-3). The insertion point and end joint offset method was selected to best represent composite floor behavior in the models. The insertion point defines the local axis along the length of frame members through a specified point on the cross section identified as a cardinal point in SAP2000®. The top center point was selected as the insertion point for all gravity beams (Figure 4-3(b)). In addition, an end joint offset on the frame members equal to half the thickness of the shell element or 53.98 mm (2.13 in) was applied, to align the bottom of the shell element with the insertion point location.

Mass in the numerical model was calculated from element self-weight and loads applied to elements. Namely, area loads and frame loads with associated mass were directly applied to shell elements and perimeter frame elements, respectively, to account for superimposed dead loads and the exterior cladding. Shell elements were discretized as shown in Figure 4-1(a), to distribute floor masses over the floors more realistically. In addition, area loads with associated mass were directly applied to shell elements to investigate the effect of additional superimposed dead loads on the coupling behavior of
Isolators were modeled as independent link elements, one beneath each column, with properties obtained from a generic constitutive behavior (i.e. bilinear force-deformation relation) in each horizontal direction. Either elastomeric or friction-based isolation bearings could be designed to provide the desired behavior; however, since axial load dependence is neglected, the assumed characteristics most closely reflect lead-rubber bearings (LRBs). The characteristic yield strength $Q_d$, post yield stiffness $K_d$, and yield displacement $u_y$ of the isolators (Figure 4-4(a)) determine the lateral force-deformation relation. Assuming $u_y = 1$ cm, $Q_d$ and $K_d$ were determined by matching the effective stiffness $K_{\text{eff}}$ and hysteretic energy dissipated to the equivalent period $T_M = 3.1$ s and damping ratio $\beta_M = 15\%$ at the MCE level displacement $D_M = 633.7$ mm (24.95 in), according to:

$$Q_d = \frac{\pi K_{\text{eff}} \beta_M D_M^2}{2(D_M - u_y)} \quad (4.2)$$

$$K_d = \frac{K_{\text{eff}} D_M - Q_d}{D_M} \quad (4.3)$$

The compressive stiffness $K_v$ of the isolators was computed assuming a linear force-
deformation corresponding to a vertical frequency of 10 Hz (Figure 4-4(b)), and viscous damping in the vertical direction corresponding to a damping coefficient of 5% at that frequency was applied. Damping was applied to the conventional and the isolated superstructure using 2.5% constant modal damping for all modes.

![Figure 4-4](image)

**Figure 4-4.** (a) horizontal and (b) vertical force-deformation relationship of isolators for numerical simulation

The in-plane stiffness of the floor slabs was accounted for through the application of diaphragm constraints. In theory, all nodes at a given floor should be “slaved” to each other in plane to achieve the desired behavior; however, this constraint also affects the out-of-plane behavior, and makes the floor system too stiff. Thus, the diaphragm constraint was applied to the primary beams only, as shown in Figure 4-1(a). This floor in-plane stiffness was calibrated based on experimentally validated models described in Guzman and Ryan (2017), used to replicate the seismic response of the previously discussed specimen tested at E-Defense, where the same modeling approach was used.

### 4.3.1 Base Model and Mass-Eccentric Variations

Mass eccentricities imposed on structures, such as permanent or future roof-mounted
equipment, superimposed dead loads (e.g. penthouse loads), or similar loading with a significant mass concentration, are believed to lead to an H-V coupling phenomenon. Moreover, the slab response depends on its vibration properties, which is greatly affected by mass concentrations (i.e. increased flexibility). Additional mass may amplify slab vibrations from the vertical component of the ground motion, and further drive the horizontal response of the structure by way of coupling. Hence, slab vibration amplifications and the coupled response of the structure are of greatest interest in this investigation.

First, a baseline model was defined – applicable to both conventional and isolated structures – that ignores the penthouse superimposed loads and is perfectly symmetric. These penthouse loads have an associated mass that would be characterized as eccentric due to its location (Figure 4-1(a)), where if absent no coupling effect would be expected.

Additional model variations that include eccentric superimposed mass concentrations in addition to the penthouse loads were also defined. For both conventional and isolated models, superimposed mass concentrations of 2%, 5%, or 10% of the floor effective seismic weight were applied directly as area loads to shell elements at specific locations on each floor, which led to a corresponding increase in the total seismic weight of the building. All superimposed masses were applied either on the right or left half of the building in the longest plan dimension (see Figure 4-5), to induce eccentricity. Reflecting design strategies for non-uniform loading, mass locations were selected that spanned tributary areas of interior frame elements to maximize support and mid-span moments under gravity loading. Variations were considered with superimposed masses placed at alternating locations relative to the floor above and below, and at the same location in all floors. For instance,
Figure 4-5(a) shows additional mass locations applied on the left half of the conventional structure and alternating locations from floor-to-floor, with a span-to-span loading sequence that maximizes support moments. Similarly, Figure 4-5(b) shows mass locations applied on the right half of the isolated structure and same locations from floor-to-floor, with loading that maximizes mid-span moments. These case variations are coded “10%LA-” and “10%RS+”, respectively, where the mass percentage increment is indicated first, followed by ‘L’ or ‘R’ for left or right, ‘A’ or ‘S’ for alternating or same floor-to-floor location, and ‘-’ or ‘+’ for negative (support) or positive (mid-span) loading sequence. The same convention is used for all mass-eccentric cases presented hereafter, which all include the penthouse loads in addition to the superimposed masses.

4.4 Ground Motion Selection and Scaling

Target spectra for the Maximum Considered Earthquake (MCE) were developed following the provisions of ASCE 7-16 (ASCE 2016) for the horizontal direction and the 2009 NEHRP recommendations (BSSC 2009) for the vertical direction (Figure 4-6). A suite of seven ground motions was selected, and components were scaled to best match the horizontal and vertical design response spectra. Little guidance is available on scaling
procedures that simultaneously consider horizontal and vertical motions; however, new commentary in ASCE 7-16 Chapter 17 provisions for isolated buildings recommends that an identical scale factor be applied to all three ground motion components when vertical excitation is included in response history analysis. For this purpose, the square root of the sum of the squares (SRSS) spectrum of the horizontal components and the vertical spectrum over the desired period ranges were combined into a single data set, and normalized by their target spectra to reduce the bias toward the high amplitude region of the spectrum.

The earthquake records selected for scaling are listed in Table 4-3, and they represent a variety of motions with different spectral content, durations and intensities, and some with very large vertical input. Records were amplitude scaled using a Least Squares fit that minimizes the sum of squared residuals for both components over a specified period range. Different period ranges were selected for the conventional and isolated buildings to reflect code provisions. For the conventional structure, the horizontal period ranged from $0.2T$ (0.096 s) to $1.5T$ (0.72 s), where $T = 0.48$ sec is the natural period of the structure (Figure 4-6(a)), obtained from modal analysis (see next section). The period range for an isolated structure was based on periods $T_{M,\text{upp}}$ and $T_{M,\text{low}}$ associated with upper and lower-bound force-deflection characteristics at the MCE displacement that account for variability in the isolation system properties (ASCE 2016). According to the provisions, the period range for the isolated building was $0.75T_{M,\text{upp}} = 1.9$ s to $1.25T_{M,\text{low}} = 4.4$ s, where $T_{M,\text{upp}} = 2.55$ s and $T_{M,\text{low}} = 3.54$ s. The vertical component for both buildings was scaled following recommendations for seismically isolated structures. The period range was selected as $0.2T_v = 0.02$ s to $1.5T_v = 0.15$ s, where $T_v = 0.1$ s is the primary vertical period of vibration
corresponding to an assumed compressive stiffness of the isolators. The same $T_v$ and scaling range was applied to the conventional building, since corresponding provisions are lacking and a fundamental vibration mode for the conventional building is difficult to define (modal analysis suggests largely a collection of localized modes). Table 4-3 shows each ground motion record with corresponding record sequence number (RSN) and station, peak ground accelerations ($PGA$) and applied scale factors.

**Table 4-3. Selected ground motion suite and characteristics**

<table>
<thead>
<tr>
<th>RSN</th>
<th>Earthquake</th>
<th>Station</th>
<th>Peak Ground Acceleration (g)</th>
<th>Scale Factor (Conv.)</th>
<th>Scale Factor (Isolated)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>n/a</td>
<td>Simulation</td>
<td>Diablo Canyon</td>
<td>X 0.988 0.699 0.564</td>
<td>Y 1.12 1.15</td>
</tr>
<tr>
<td>6</td>
<td>Imperial Valley-02, USA, 1940</td>
<td>El Centro Array #9</td>
<td>0.313 0.215 0.205</td>
<td>Y 3.41 3.49</td>
<td></td>
</tr>
<tr>
<td>1158</td>
<td>Kocaeli, Turkey, 1999</td>
<td>Duzce</td>
<td>0.312 0.358 0.228</td>
<td>Z 2.77 1.91</td>
<td></td>
</tr>
<tr>
<td>779</td>
<td>Loma Prieta, USA, 1989</td>
<td>LGPC</td>
<td>0.804 0.598 0.823</td>
<td>X 1.00 0.78</td>
<td></td>
</tr>
<tr>
<td>1063</td>
<td>Northridge, USA, 1994</td>
<td>RRS, 228</td>
<td>0.825 0.490 0.838</td>
<td>Z 1.11 1.09</td>
<td></td>
</tr>
<tr>
<td>1086</td>
<td>Northridge, USA, 1994</td>
<td>SYLMAR - Olive View Med FF</td>
<td>0.843 0.605 0.536</td>
<td>X 1.42 1.21</td>
<td></td>
</tr>
<tr>
<td>143</td>
<td>Tabas, Iran, 1978</td>
<td>Tabas</td>
<td>0.837 0.853 0.689</td>
<td>Y 0.93 1.07</td>
<td></td>
</tr>
</tbody>
</table>

Figures 4-6 and 4-7 compare the spectra for individual records, scaled according to the procedure described above, the average response spectrum, and the target MCE spectrum in the horizontal and vertical directions, for the conventional and isolated structures, respectively. As shown in Figure 4-6(a), the average horizontal spectrum falls below the
corresponding ordinates of the target MCE spectrum in the period range between 0.09 s and 0.2 s for the conventional building, and the average vertical spectrum is well below the target for both buildings. This does not meet the ASCE code requirement that the average of the SRSS spectra from the horizontal components does not fall below the corresponding ordinate of the target (ASCE 2016). To remediate this misfit, a factor corresponding to a $\text{PGA}$ increase of about 0.5g would have to be applied to the horizontal components, and an even larger factor would be required if the vertical component was held to the same scale factor and fit criteria. Nevertheless, the fit is still adequate for the horizontal direction over most of the period range, especially for the isolated building (Figure 4-7(a)). Hence, the authors have opted for presenting the results herein using the scale factors obtained from the amplitude fit shown in Figures 4-6 and 4-7, which highlight the limitation of scaling the vertical component relative to the horizontal.
Figure 4-6. Ground motion response spectra and average vs target for (a) SRSS horizontal and (b) vertical directions and respective period range for scaling for the conventional building.
Figure 4-7. Ground motion response spectra and average vs target for (a) SRSS horizontal and (b) vertical directions and respective period range for scaling for the isolated building.

4.5 Fundamental Properties of Buildings

Modal analysis of the models was applied to determine the elastic dynamic properties of the buildings. An analysis of the mode shapes and frequencies of the numerical model is useful for interpreting the floor spectra, and understanding the potential for H-V
coupling. For the analysis, equivalent linear parameters were assigned to the isolator elements in the horizontal and vertical directions corresponding to $T_M = 3.1$ s and $T_v = 0.1$ s. Figure 4-8 illustrates the deflected shape of the building in 3D space of X-direction modes 1, 4, 16 and 33 for the conventional building. The undeflected shape of the system is shown in light gray lines for reference. Comparable modes were also found in the Y-direction. Mode 1 ($T = 0.48$ s) is the 1\textsuperscript{st} horizontal structural mode, characterized by linear deflection across the structural system. Mode 4 ($T = 0.22$ s) corresponds to the longest period vertical mode, characterized by vertical slab vibration primarily at the roof level, directly beneath the penthouse superimposed loads. Mode 16 ($T = 0.19$ s) represents the 2\textsuperscript{nd} horizontal structural mode, characterized by deflection with a node between the 3\textsuperscript{rd} floor and roof. Lastly, Mode 33 ($T = 0.12$ s) represents the 3\textsuperscript{rd} horizontal structural mode, which appears to be a coupled H-V mode that could be driven by vertical shaking due to its combination of vertical slab vibration at the roof level and horizontal deflection of the system at the 2\textsuperscript{nd} and 3\textsuperscript{rd} floors in particular.

Similarly, Figure 4-9 illustrates X-direction modes 1, 5, 7 and 39 for the isolated building. Mode 1 ($T = 3.15$ s) represents the isolation mode, characterized by shear deformation in the isolators, and no significant deflection in the superstructure. Mode 5 ($T = 0.41$ s) corresponds to the 1\textsuperscript{st} horizontal structural mode, characterized by linear deflection across the system, with a node at the 3\textsuperscript{rd} floor. Hence, the rigid structure assumption is upheld in Mode 1, as the isolation period is much longer than the first structural mode period. Mode 7 ($T = 0.24$ s) represents the longest period vertical mode, analogous to Mode 4 in the conventional building. Mode 39 ($T = 0.11$ s) represents the 2\textsuperscript{nd} horizontal structural mode, characterized by considerable slab vibration across all floors.
and horizontal deflection of the system (i.e. also could be an H-V coupled mode driven by vertical shaking).

4.6 Data Processing

Response history analysis was applied to the 3D models for each of the ground motion records selected, where a low-pass filter with a cut-off frequency of 50 Hz (0.02 s) was applied to each record. Response parameters of interest include peak floor (horizontal) and slab (vertical) accelerations, as well as floor and slab spectral accelerations. A single horizontal floor acceleration history for each floor level was obtained by averaging the accelerations observed at nine column nodes at the floor level (N, S, E, W, C, NE, NW, SE, and SW) in each direction of the motion. These floor acceleration histories were then processed to obtain peak floor accelerations (PFA), and floor spectral accelerations in each direction. In the vertical direction, vertical acceleration histories were observed at the center node of each of 24 possible slab locations bounded by the diaphragm constraints shown in Figure 4-1(a) (i.e. column nodes). The peak slab acceleration (PSA) as well as the slab spectral accelerations out of 6 possible slab locations within the quadrants (NE, NW, SE and SW) were determined for each quadrant on every floor. One location was identified within each quadrant, identical over all floors, that reflects the location where the most peaks were observed. Results are presented for this quadrant, which varied from record to record.
Figure 4-8. Representative modal deflected shapes of the conventional model in 3D, along with their associated periods and descriptions. The undeflected shape of the system is shown in gray lines.
Figure 4-9. Representative modal deflected shapes of the isolated model in 3D, along with their associated periods and descriptions. The undeflected shape of the system is shown in gray lines.
4.7 Base Model Response

Conventional and isolated building responses in the horizontal and vertical direction are compared next for the base model, which, as discussed previously, represents a symmetric system without H-V coupling. The El Centro ground motion was selected as a representative input excitation for all the results presented herein. Figure 4-10(a)-(b) shows $PFA$ and (c)-(d) $PFA$ normalized by the $PGA$, compared for each building in each horizontal direction. These plots include 2D vs 3D excitation for the El Centro record and the median computed over the ground motion suite. As seen in the figure, with the exception of the conventional building in the Y-direction, the floor acceleration profiles for El Centro are very similar to the median profiles. $PFA$ normalized by the $PGA$ can be interpreted as the amplification of the horizontal floor acceleration relative to ground, and will be hereafter referred to as “floor amplification factor”. The amplification trends shown in the profiles in Figure 4-10(c)-(d) reflect on the efficacy of base isolation against conventional systems in reducing floor accelerations to less than the $PGA$. In addition, these profiles suggest that in either building, the base model does not experience floor amplifications due to H-V coupling, as significant differences are not observed for 2D versus 3D input. Under the applied ground motions, yielding in the conventional building would be expected, resulting in reduced floor acceleration demands. While the amplification factors discussed herein would be lower if yielding occurred, nonlinear effects are neglected to allow direct response comparisons to the isolated building.

Figure 4-11 shows $PSA$ normalized by the vertical direction $PGA$, for every floor in both buildings at (a), (c) the NW quadrant for El Centro, and (b), (d) median values for the rest of the locations. $PSA$ normalized by the vertical $PGA$ will be hereafter referred to as
“slab amplification factor”. As seen in Figure 4-11, profiles of slab amplification factors are shown for the base model, and summarized for all the superimposed mass cases discussed previously. These profiles are referred to as ‘Other Cases’, and are presented here to illustrate the variability in peak slab vibration response against the base model. As shown in Figure 4-11(b) and (d), the slab amplification trends suggest that the response is similar for every quadrant, and any quadrant can be selected as a representative location. In addition, both buildings show median slab amplification factors between about 2.5 to 6.5.

Figure 4-10. Base model (a)-(b) peak horizontal floor acceleration and (c)-(d) horizontal amplification profiles for each building and each direction for 2D and 3D El Centro and median values.
Figure 4-11. (a), (c) NW vertical amplification profiles for El Centro, and (b), (d) median values for every quadrant, for all added mass cases vs base model.

Figure 4-12 presents some representative floor and slab response spectra for El Centro for the base models. Figure 4-12(a)-(b) shows 5% damped horizontal spectra in the X-direction at the 3rd floor and roof, compared for both 2D and 3D input in the conventional and isolated buildings, respectively. Two distinct peaks appear in the conventional building spectra in both floors (Figure 4-12(a)). The first one at a period of 0.48 s (2.1 Hz) represents the 1st structural mode in the X-direction (Mode 1), and the second at a period of 0.19 s (5.3 Hz) represents the 2nd structural mode (Mode 16), as previously identified in modal analysis. Figure 4-12(b) shows a small peak for the first or isolation mode at a period of 3 s (0.33 Hz), which can vary from record to record due to the nonlinearity of the isolation system. A second peak for the 1st structural mode appears at a period of 0.41 s (2.5 Hz) in
the roof only, since the 3rd floor is a node (Mode 5). Figure 4-12(a)-(b) shows no difference between 2D and 3D input, aside from the slight amplitude increase at a period of about 0.12 s (8.3 Hz) for 3D input relative to 2D on the 3rd floor of the isolated building. This also confirms that in both buildings there is no floor acceleration amplification due to H-V coupling.

Figure 4-12(c)-(d) shows 5% damped vertical slab spectra at representative NW and SE locations in both buildings. These slab spectra indicate a multi-mode response behavior of the slabs in the period range between 0.07 s and 0.3 s (3.3 to 14.3 Hz). This range is consistent with reported slab vibration frequencies of composite floor systems, which range between 3 and 13 Hz (Allen and Pernica 1998, Boice 2003, Hicks 2004). Note that unlike the horizontal response spectra, spectral acceleration amplitudes in the vertical direction are comparable in both buildings.
Figure 4-12. Base model sample 5% damped (a)-(b) 3rd floor and roof spectra in the X-direction, and (c)-(d) roof slab spectra at NW and SE locations, in each building configuration.

4.8 Horizontal Floor Accelerations and H-V Coupling

4.8.1 Influence of Mass Eccentricity

The influence of imposed mass eccentricities on the horizontal response of the structure is evaluated next. Figure 4-13 shows floor amplification factors for the base model (‘BM’), the model with a penthouse (‘PH’), and all the mass eccentricity variations considered (20 total), for every floor and each horizontal direction for El Centro ground motion in the conventional building. As seen in Figure 4-13, floor amplification factors are essentially the same for 2D and 3D shaking, indicating H-V coupling is absent and that Mode 33, identified in modal analysis as a potential coupled mode (Figure 4-8), is not a coupled mode. This suggests that the conventional building is not sensitive to eccentric mass
variations.

![Figure 4-13](image)

Figure 4-13. Conventional building floor amplification factors at each floor for all the mass eccentricities considered, and compared for 2D and 3D El Centro ground motion.

Figure 4-14 shows comparable floor amplification factors for the isolated building. In most cases shown, the amplification factor increases at the 2\textsuperscript{nd} and 3\textsuperscript{rd} floor locations for 3D shaking vs 2D (without vertical). However, base and roof locations showed no significant additional amplification due to vertical excitation, and nearly no sensitivity to mass variability. These response trends strongly suggest that for many mass-eccentric cases, a coupled H-V mode is contributing to the response during 3D shaking that is absent during 2D. To validate this theory, Figure 4-15 compares X-direction 5% damped floor spectral accelerations for El Centro PH, 5%LS- and 10%RA+ cases for 2D and 3D input, as well as the PFA for each. Note that the PFA (an extension of the spectrum to the zero period) is plotted on the y-axis for reference. The 1\textsuperscript{st} structural mode of the isolated building
(Figure 4-9) appears as a peak at about 0.4 s at the base and the roof for these sample simulations (Figure 4-15), such that on average, $PFA$ and floor amplification factors are higher at the base and roof than the other floors (Figure 4-14). In addition, the 3D simulations exhibit an additional peak at 0.12 s (not present in 2D) that reflects participation of the H-V coupled Mode 39 (Figure 4-9). This mode has higher displacements at the 2nd and 3rd floor than the base and roof level, and thus shows up more strongly in these floors. From Figure 4-15, the coupled mode participates weakly in PH and 5%LS-, but strongly in 10%RA+, which in turn is reflected in the floor amplification factors in Figure 4-14.

This evidence suggests that the variation in the 2D and 3D floor amplification factors across all the mass-eccentric cases considered is really an indication of the extent to which the coupled modes participate in the response. The greater the discrepancy in floor amplification factors between 3D and 2D input, the higher the participation of the coupled mode, meaning the added mass excited that mode in the presence of vertical excitation. Conversely, 2D amplification factors can be used to interpret horizontal structural modes that are not coupled. For both horizontal directions shown in Figure 4-14, the amplifications tend to increase as the mass is increased. Amplification also tends to be larger when the mass is placed on the right half of the building, which is further away from the penthouse location in the long direction.
Figure 4-14. Isolated building floor amplification factors at each floor for all the mass eccentricities considered, and compared for 2D and 3D El Centro ground motion.
Figure 4-15. Comparison of X-direction 5% damped spectral accelerations within the period range considered for average SA, showing base through roof for PH, 5%LS-, and 10%RA+ subjected to El Centro.

Figures 4-16 and 4-17 show 2D and 3D spectral acceleration (SA) normalized by PFA for El Centro in each direction at every floor, and for all the cases considered in the conventional and isolated buildings, respectively. SA was computed by averaging the spectral acceleration observed over periods ranging from 0.1 s to 0.8 s (1.25 Hz to 10 Hz). This period range was selected in reference to the constant amplitude spectral region bounded between 1.3 Hz and 8.3 Hz on the target horizontal and vertical response spectra for shake-table testing of NCSs (ICC, 2010). By definition, an NCS is considered flexible for fundamental frequencies between 1.3 Hz and 16.7 Hz, thus the period range selected
applies to a wide range of flexible components. The normalized spectral acceleration (i.e. $SA/PFA$) will be hereafter referred to as the “average floor spectral amplification factor”. For the conventional building, the average floor spectral amplification factors are nearly the same for 2D and 3D (1.9 to 2.7 on average in both directions), and they are not influenced by mass variability (Figure 4-16). As previously discussed, only the 1st and 2nd structural modes participate in the conventional building. For the isolated building (Figure 4-17), the average spectral amplification factors appear to decrease with increasing mass on the 2nd, 3rd and roof floors, particularly in the Y-direction, for some of the cases considered. The 1st ($T = 0.41$ s) and 2nd ($T = 0.11$ s) structural modes (5 and 39 respectively in Figure 4-9) are the only modes that would appear in horizontal floor spectra in the period range considered for the average $SA$, and thus the only modes that would influence the spectral amplification factors. To illustrate the reason behind the decreasing trends, Figure 4-15 also shows the average $SA$ over the period range for the sample simulations considered. If the coupled mode is excited, the $PFA$ at the 2nd and 3rd floor is larger for 3D input vs 2D, as this mode is driven by vertical excitation. The base level and roof $PFA$ on the other hand, are virtually the same, with a few exceptions (e.g. see Y-dir. case 10RS- or 10RA+ in Figure 4-14). Likewise, as the coupled mode is excited, the average $SA$ increases mainly at the 2nd and 3rd floors. Figure 4-15 indicates that although the $PFA$ also increases, the increase in the average $SA$ relative to the $PFA$ is less than the increase in $PFA$ relative to the $PGA$ for 3D input, resulting in lower spectral amplifications than for 2D. This response suggests that the increase of floor amplification factors observed in Figure 4-14 is counteracted by a decrease in average floor spectral amplification factors seen in Figure 4-17.
Figure 4-16. Conventional building floor average spectral amplification factors at each floor for all the mass eccentricities considered, and compared for 2D and 3D El Centro ground motion.
4.8.2 Influence of Vertical Acceleration Intensity

Previous discussion of floor amplification factors as well as average floor spectral amplification factors presented evidence that the conventional building is not sensitive to eccentric mass variations, indicating absence of a H-V coupled mode in the response. In contrast, the isolated building amplification factors had evidence of H-V coupling when the response of 2D and 3D input were compared. Hence, the effect of vertical acceleration intensity on the H-V coupled response in the isolated building is evaluated next. Figure 4-18 shows El Centro floor spectra from base to roof with increasing vertical acceleration intensity (0 to 200% of target) for each direction considered in the isolated building ‘PH’
model. The horizontal input was constant at 100% of the target for all the simulations, and the reference 2D (vertical = 0%) and 3D (vertical = 100%) responses are identified. Peaks representing the 1\textsuperscript{st} and 2\textsuperscript{nd} structural modes appear in Figure 4-18 (the isolation mode peak corresponding to \( T > 1 \text{ s} \) is not shown). The first structural mode is insensitive to vertical acceleration intensity, but the 2\textsuperscript{nd} structural mode (\( T = 0.12 \text{ s}, \) or 8.3 Hz) peak amplitude increases with increasing vertical excitation. This confirms that the 2\textsuperscript{nd} structural mode (Mode 39 in Figure 4-9) is an H-V coupled mode and the amplification of this mode in the horizontal floor spectra is being driven by the vertical excitation.

Figure 4-18. Comparison of 5\% damped spectral accelerations, base through roof for PH model, subjected to El Centro with increasing vertical component intensity.
4.9 Vertical Slab Accelerations

4.9.1 Influence of Mass Eccentricity

In this section, the influence of mass eccentricities on the vertical response of the floor system is evaluated. Figure 4-19 shows (a), (c) NW median slab amplification factors, and (b), (d) median slab amplification factors averaged over all quadrants, for the BM, PH model, and all other mass-eccentric cases in both buildings. The amplification trends observed in Figure 4-19 (a) and (c) suggest the vertical response is highly sensitive to mass eccentricities, meaning added floor mass does not necessarily lead to increased amplification factors. Roof slab amplification factors are largest at the NW and SW (not shown) locations, due to close proximity to the PH superimposed loads. Higher amplifications at the roof can also be attributed to higher superimposed dead loads compared to lower floors (i.e. 25 psf vs 23 psf, without self-weight).

Figure 4-20 shows median vertical $SA$ normalized by the $PSA$ for all the cases considered at the (a), (c) NW quadrant and (b), (d) averaged over all quadrants in both buildings. As before, the $SA$ was computed by averaging the spectral acceleration over the range of 0.1 s to 0.8 s (1.25 Hz to 10 Hz). The normalized vertical spectral acceleration (i.e. $SA/PSA$) will be hereafter referred to as the “average slab spectral amplification factor”. Average slab spectral amplification factors in both buildings range from about 1 to 2. Systematic trends for each floor, reflecting little dependence on the mass eccentricity, are noticeable in Figure 4-20(b) and (d), when the median factors are averaged over all quadrants.

For the isolated building (Figure 4-20(d)), $SA$ is not amplified at the base level compared to the $PSA$ (average slab spectral amplification ~ 1), because the base slab is
stiffer than the floor slabs at upper levels. To demonstrate, Figure 4-21 plots median and individual ground motion 5% damped slab spectra at every floor and every quadrant, for the isolated building PH model. At the base level, dominant slab vibration frequencies (denoted by spectral peaks) are higher than upper levels, which can be attributed to the fixed connection assumption at all base level beam-column joints designed to intentionally stiffen the base level. The stiffening effect at the base reduces the period of the dominant spectral peak close to or below the average SA cutoff period, $T = 0.1$ s (Figure 4-21). As a result, the average SA at the base is comparable to the PSA or lower, which in turn decreases the slab spectral amplification factors seen in Figure 4-20(d).

Figure 4-19. (a), (c) NW median slab amplification factors and (b), (d) median averaged over all quadrants, at each floor and for all the mass eccentricities considered in both buildings.
Figure 4.20. (a), (c) NW median slab spectral amplification factors and (b), (d) median averaged over all quadrants, at each floor and for all the mass eccentricities considered in both buildings.
4.9.2 Influence of Vertical Isolation Period

In this section, the influence of the assumed vertical frequency (i.e. 10 Hz) of the isolation system on the vertical response of the slabs is evaluated. Figure 4-22 plots (a) NW slab amplification factors for El Centro and (b) median slab amplification factors averaged over all quadrants, for vertical isolation frequencies of 8, 10, and 15 Hz. Frequencies outside of this range were considered to be uncharacteristic of LRBs. Figure 4-22(a) at the base level appears to indicate that slab amplification factors increases with increasing vertical isolation frequency. However, the same effect does not translate to the upper floors with more flexible floor slabs; rather the effect of the isolation frequency on these floors appears to be random. From evaluation of median slab amplification factors in individual
quadrants (not shown in the figures), similar trends were observed in the NW and SW quadrants, and separately in the NE and SE quadrants. The trends observed suggest the effect of the assumed vertical isolation frequency on the vertical slab response is not systematic. Figure 4-22(b) indicates that the effect of the vertical isolation frequency on the slab response is negligible when median slab amplification factors were averaged over all quadrants.

Figure 4-22. (a) NW slab amplification factors for El Centro, and (b) median slab amplification factors averaged over all quadrants, compared for different isolation system vertical frequencies.

4.9.3 Influence of Slab Flexibility

In this section, the influence of different slab design assumptions, which lead to changes in slab vibration properties, on the vertical response of the floor system is evaluated. The BM, PH, and other mass-eccentric cases considered so far assumed 107.95 mm (4.25”) thick lightweight concrete (LWC) slab. Figure 4-23 shows slab amplification
factors for (a), (c) El Centro NW quadrant and (b), (d) median values averaged over all quadrants, in both buildings. As seen in Figure 4-23, the responses are presented for the PH model (LWC), as well as modifications to this model that include normal weight concrete (NWC), a thinner 76.2 mm or 3” slab (LWC) and a slab with double the stiffness (LWC, stiff. (x2)). The modifications correspond to increased mass and stiffness for NWC, decreased mass and stiffness for 3” slab (LWC), and increased stiffness for (LWC, stiff. (x2)). In addition, the BM (LWC) is also shown for reference. Results for El Centro NW quadrant (Figure 4-23(a), (c)) indicate that the effects are very inconsistent at different floors. For instance, an increase or decrease in the slab amplification factor at lower floors, in comparison to the PH model, is often matched with the opposite effect at higher floors. Evaluation of the slab vertical response at other quadrants (not shown) indicates that the effect of variable slab properties is inconclusive.

To provide further insight on the effect of slab property changes, Tables 4-4 and 4-5 show modal periods for selected vertical modes for the PH model and variations considered herein, for the conventional and isolated buildings, respectively. In addition, the modal mass participation of the selected modes is shown for the PH model in both tables for reference. The vertical modes were selected based on the PH model, considering modal mass participation in the vertical direction greater than 2%, and significant slab vibration observed on the mode shapes, along with negligible mass participation in the horizontal direction. As seen in these tables, applying NWC (increased mass and stiffness) or using a thinner 3” slab (decreased mass and stiffness) both lead to lengthening of the natural period of these vertical modes in both buildings, and an increase in the overall vertical response flexibility. The only exception in this case is Mode 32 (Table 4-5) for the thinner slab case,
where the period shortens. In contrast, doubling the slab stiffness shortens the natural period of the vertical response in both buildings, as expected. Overall, the changes in modal periods are very small, which is reflected in the random influence on slab amplification factors at each floor shown in Figure 4-23. In comparison to the PH model, median slab amplification factors averaged over all quadrants (Figure 4-23 (b), (d)) indicated that decreased slab flexibility led to decreased amplification factors, while increased slab flexibility appeared to increase amplification factors in the isolated building (Figure 4-23(d)), but was proven inconclusive by the effect observed in the conventional building (Figure 4-23(b)). The median amplification factors ranged from about 3 to 6, when averaged over all quadrants, excluding the BM.

![Graph showing slab vibration amplifications at each floor for the slab flexibility changes considered.](image)

**Figure 4-23.** NW El Centro and median slab vibration amplifications at each floor for the slab flexibility changes considered.
Table 4-4. Effect of slab flexibility on the conventional building vertical modes

| Vertical Mode | PH, 4.25"
Slab (LWC) | PH, 4.25"
Slab (NWC) | PH, 3"
Slab (LWC) | PH, 4.25"
Slab Stiff. (x2) (LWC) |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Mode 4</td>
<td>0.2183</td>
<td>0.2326</td>
<td>0.2273</td>
<td>0.2093</td>
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<tr>
<td>Mode 6</td>
<td>0.2122</td>
<td>0.2294</td>
<td>0.2175</td>
<td>0.2033</td>
</tr>
<tr>
<td>Mode 8</td>
<td>0.2070</td>
<td>0.2235</td>
<td>0.2116</td>
<td>0.1982</td>
</tr>
<tr>
<td>Mode 11</td>
<td>0.1981</td>
<td>0.2138</td>
<td>0.2025</td>
<td>0.1897</td>
</tr>
<tr>
<td>Mode 20</td>
<td>0.1744</td>
<td>0.1897</td>
<td>0.1745</td>
<td>0.1666</td>
</tr>
<tr>
<td>Mode 26</td>
<td>0.1561</td>
<td>0.1691</td>
<td>0.1632</td>
<td>0.1500</td>
</tr>
<tr>
<td>Mode 28</td>
<td>0.1489</td>
<td>0.1607</td>
<td>0.1531</td>
<td>0.1420</td>
</tr>
</tbody>
</table>

Table 4-5. Effect of slab flexibility on the isolated building vertical modes

| Vertical Mode | PH, 4.25"
Slab (LWC) | PH, 4.25"
Slab (NWC) | PH, 3"
Slab (LWC) | PH, 4.25"
Slab Stiff. (x2) (LWC) |
<table>
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4.10 Statistical Evaluation of Amplification Factors

Equation 13.3-1 in ASCE7-16 (2016) can be used to estimate the horizontal seismic design force \( (F_p) \) of NCSs, and it is presented below as Eq. (4.4):

\[
F_p = \frac{0.4a_pS_{DS}}{R_p/I_p} \left(1 + 2\frac{z}{h}\right)W_p \tag{4.4}
\]

where \( W_p \) is the component weight, \( R_p \) is a response modification factor, \( I_p \) the component importance factor \( (R_p/I_p = 1 \) for unreduced inelastic response as it will naturally occur on
the component), 0.4SDS = PGA, \((1 + 2\frac{z}{h})\) represents a linear amplification factor over height (i.e. \(\frac{PFA}{PGA}\)), and \(a_p\) is the component amplification factor. Then, the spectral acceleration of the component, \(S_{a,comp} = 0.4a_pSDS\left(1 + 2\frac{z}{h}\right)\) where \(a_p\) is taken as 1 (min.) for rigid components, and 2.5 (max.) for flexible. In this study, \(\left(1 + 2\frac{z}{h}\right)\) and \(a_p\) have been referred to, in the horizontal direction, as the floor amplification factor, and the average floor spectral amplification factor, respectively. In the vertical direction, the same factors apply and have been referred to in this study as the slab amplification factor, and the average slab spectral amplification factor. However, the code takes \(z = 0\) in the vertical direction, which implies that the code value for the slab amplification factor = 1.

Both of these amplification factors in each direction are discussed next for the conventional and isolated buildings in turn. Figure 4-24 plots minimum and maximum (a), (c) median (‘\(\theta\’)’), and median plus one standard deviation (‘\(\theta + 1\sigma\)’) values of floor amplification factors for 2D and 3D input, and (b), (d) ‘\(\theta\)’ and ‘\(\theta + 1\sigma\)’ values of slab amplification factors for 3D input at each floor level, where subfigures (a), (b) are for the conventional building and (c), (d) are for the isolated building. The statistics were computed over the ground motion suite, considering the BM, PH model, and all other mass-eccentric cases, and are compared against \(\left(1 + 2\frac{z}{h}\right)\) where applicable. The \(\theta\) values reported represent the absolute minimum and maximum of the medians over all mass eccentric models. As observed in Figure 4-24(a) the numerical predictions for the minimum 2D and 3D median floor amplification factors in the conventional building are in agreement with the code recommended factor. As mentioned, yielding within the structure would lower the predictions shown in Figure 4-24(a) making the code values
more conservative. H-V coupling was not identified in the conventional building, and thus no difference is observed for 3D amplification factors relative to 2D. Nevertheless, the possibility of this effect should not be neglected, and the code should allow for some conservatism. For the isolated building (Figure 4-24(c)), which lacks code provisions, an average floor ‘reduction’ factor of 0.5 would appear to be an adequate design recommendation. For the slab amplification factors (Figure 4-24(b) and (d)), both buildings indicate that the code recommendation is very unconservative in comparison to the predicted values. A more conservative amplification factor should be applied to the vertical direction, perhaps on the order of 4-5.

Figure 4-24. Floor and slab median (‘0’) and ‘0 + 1σ’ amplification factors for (a), (b) conventional and (c), (d) isolated buildings, respectively, at each floor level and compared against the code where applicable.
Figure 4-25. Floor and slab median (‘\(\theta\)’) and ‘\(\theta + 1\sigma\)’ average spectral amplification factors for (a), (b) conventional and (c), (d) isolated buildings, respectively, at each floor level and compared against the code where applicable.

Figure 4–25 plots minimum and maximum median (‘\(\theta\)’), and median plus one standard deviation (‘\(\theta + 1\sigma\)’) values for (a), (c) average floor spectral amplification factors and (b), (d) average slab spectral amplification factors, computed over the ground motion suite at each floor level, for each building, and compared against the code \(a_p = 2.5\) for flexible components where applicable. As observed in Figure 4-25(a) and (c), all the numerical predictions in the horizontal direction in both conventional and isolated buildings appear to be in agreement with the code recommended \(a_p\) factor limits (i.e. 1 – 2.5), while in the vertical direction, an \(a_p\) factor limit of 2 might be more adequate.
4.11 Conclusions

This paper has focused on investigating the factors that may influence slab vibration and/or induce a horizontal-vertical (H-V) coupled response of base-isolated and conventional buildings with irregular configurations. The work has been motivated by findings on the seismic response of a full-scale building with strong asymmetry subjected to 3D (combined horizontal and vertical) excitation at E-Defense. Parameters that influence the vertical response of the floor system and subsequent H-V coupling effect were investigated through computational simulations of a 3D numerical model of a hypothetical 3-story building base isolated with lead-rubber bearings (LRBs) and with conventional configuration. The numerical model used standard modeling techniques that were shown in Guzman and Ryan (2017) to predict with sufficient accuracy slab vibration and H-V coupling. The parameters investigated included superimposed mass induced eccentricities, vertical acceleration intensity, vertical stiffness of the isolators, and slab design assumptions (i.e. slab stiffness/mass variations). Mass eccentricities were observed to considerably alter the frequency of vertical modes, having the most influence on the vertical and coupled response. In addition, the direct implications of vertical slab vibrations and the H-V coupling behavior on the design forces of nonstructural components and systems (NCSs) were evaluated. The numerical simulations supported the following conclusions in alignment with the objectives of this paper:

1. A base model (‘BM’) representing a symmetric system did not lead to amplification of horizontal acceleration demands in the presence of vertical excitation (i.e. no H-V coupling) in both conventional and isolated buildings. However, the BM median
slab acceleration amplifications were considerable in both buildings. This model was used as a baseline comparison to the response of parameter variations.

2. Horizontal structural modes and relevant vertical modes were identified through modal analysis of both buildings, including higher modes that indicated possible H-V coupling behavior. These modes were effectively matched to horizontal floor spectral peaks, allowing identification of participating higher H-V coupled modes in the presence of vertical excitation.

3. The influence of imposed mass eccentricities, evaluated by comparing horizontal floor acceleration amplifications for 2D (horizontal only) and 3D excitation, indicated absence of H-V coupling in the conventional building, and low sensitivity to eccentric mass variations. In contrast, certain mass eccentricities applied to the isolated building induced participation of a higher structural H-V coupled mode.

4. Variation in the 3D relative to 2D horizontal floor amplification factors across all the mass-eccentric cases considered was shown to indicate the extent to which the coupled mode participated in the response. The greater the discrepancy in floor amplification factors between 3D and 2D input, the higher the participation of the coupled mode, meaning the added mass excited that mode in the presence of vertical excitation. Induced mass eccentricity by a singular superimposed load of about 5% of the floor effective seismic weight, which is representative of a small penthouse on the roof, was enough to excite the H-V coupled mode in the isolated building.

5. Averaged spectral accelerations observed over periods ranging from 0.1 s to 0.8 s (1.25 Hz to 10 Hz), applicable to a wide range of flexible NCSs, were normalized
by peak floor accelerations to evaluate floor spectral amplification factors. For the conventional building, these factors were nearly the same for 2D and 3D excitation, and were not influenced by mass variability. In the isolated building however, the average spectral amplification factors appeared to decrease with increasing mass on the 2nd and 3rd floors, but were negligibly affected by mass variations at the base and roof. The effect was attributed to the structural modes that engaged in the response over the period range considered.

6. The floor acceleration amplifications in response to vertical excitation in the isolated building were confirmed by evaluation of floor spectra with increasing vertical acceleration intensity, while keeping the horizontal input at 100% of the target. A structural mode with a frequency of 9 Hz or $T = 0.11$ sec was positively identified as an H-V coupled mode (i.e. driven by vertical excitation) in the isolated building response.

7. Slab accelerations were generally amplified by a factor of 2.5 to 6.5 relative to the vertical ground acceleration. Evaluation of slab accelerations in response to induced mass eccentricities, and other parameters considered, indicated that all variations led to slightly altered vertical modal frequencies. The modified frequencies had an unpredictable influence on slab amplification factors. Thus, vertical design forces of NCSs should conservatively account for a range of possible slab amplifications, which are not reflected in current code provisions.

8. Statistical evaluation of amplification factors in the conventional building indicated greater unconservatism in the code recommended amplification factors in the vertical direction in comparison to the horizontal. A more conservative
amplification factor should be applied to the vertical direction, perhaps on the order of 4-5. In the horizontal direction code recommended values appear to be adequate. For the isolated building which lacks corresponding code provisions, an average floor ‘reduction’ factor of 0.5 would appear to be an adequate design recommendation. In the vertical direction, similar amplification factors to the conventional building would be needed.

9. Predicted component amplification factors evaluated in both buildings that account for probable spectral amplification of the floor or slab acceleration (i.e. $a_p$ factor) were found to be in agreement with the code recommended limits (i.e. $1 – 2.5$), while in the vertical direction, an $a_p$ factor limit of 2 might be more adequate.
4.12 References


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Spectra, 32(2), 771-794.


Chapter 5. Summary and Conclusions

5.1 Summary

Base isolation is one of the most effective and well-known mitigation strategies implemented to protect both building structures and their nonstructural components and systems (NCSs) from seismic demands. This technology has slowly been adopted by the engineering community since the 1980s, to achieve improved building performance objectives. Most system level testing of isolated buildings has been performed on reduced-scale bare-frame models that cannot simulate the effects of realistic floor system response or NCSs/contents. In addition, testing of isolated building models subjected to combined horizontal and vertical motions has been limited.

Recent developments in modern seismic performance objectives that allow superior structural designs have highlighted the importance of providing continued functionality to the NCSs. These elements, such as piping, partition walls, suspended ceilings, electrical systems, equipment, etc., are not part of the load carrying system, yet they provide the necessary operational and functional capabilities of the facility. Ever since the 1971 San Fernando earthquake, field observations have reported extensive damage to NCSs, resulting in threats to life safety, economic losses and functionality losses.

For this study, experiments were conducted on a full-scale, five-story, steel moment-frame building subjected to a number of 2D (horizontal only) and 3D (combined horizontal and vertical) strong earthquake records using the world’s largest shake table at E-Defense. The test bed specimen was shaken with two seismic isolation systems and in the fixed-base configuration. In total, the building structure was subjected to 41 sinusoidal and earthquake simulations including 13 distinct earthquake records. The NCSs included an integrated
system of interior walls, suspended ceilings, fire-sprinkler piping, and a precast concrete cladding column cover, all of which were assembled using U.S. construction techniques. The two isolation systems were: 1) triple friction pendulum bearings (TPB), and 2) a hybrid combination of lead-rubber and cross-linear bearings (LRB/CLB). The project was a collaboration between U.S. researchers of the Network for Earthquake Engineering Simulation (NEES) Tools to facilitate the widespread use of Isolation and Protective Systems (TIPS) and Grand Challenge projects.

The datasets corresponding to each building configuration have been permanently archived as separate standalone experiments under NEES TIPS/Project No. 571, and are currently publicly accessible through the DesignSafe-CI (cyberinfrastructure) as part of the Natural Hazards Engineering Research Infrastructure (NHERI). The archiving, organization and documentation of this comprehensive dataset (about 211 GB) is described in detail in Chapter 2. Each dataset includes unprocessed and derived data, metadata pertinent to the experiment, and supporting documentation to assist with interpretation of the data. This open-access resource has high potential for data re-use, as researchers have direct access to extensive system-level dynamic data with which to develop and calibrate new component models. Examples include models for bearing response (force versus displacement), seismic-induced floor vibration, and various parts of the NCSs.

Chapters 3 and 4 describe studies that directly stem from the use of this unique dataset resource. In Chapter 3, the seismic response of the LRB/CLB and fixed-base configurations is investigated, with particular focus on slab vibration amplifications in response to vertical excitation and a horizontal-vertical (H-V) coupling effect observed in both buildings. The H-V coupling appeared as a significant amplification of horizontal floor accelerations.
observed during 3D shaking compared to 2D. A 3D numerical model using industry standard software and modeling assumptions was developed to replicate the vertical response and H-V coupling effect in detail in both building configurations. The coupling behavior was partially attributed to the strong asymmetry of the building (i.e. unequal bay widths, staircase located in a corner) and enhanced by asymmetrically configured supplemental mass at the roof to represent equipment or a roof penthouse.

Motivated by the findings in Chapter 3, the study described in Chapter 4 investigates the factors that may influence slab vibration and/or induce an H-V coupled response of base-isolated and conventional buildings with irregular configurations. Parameters that influence the vertical response of the floor system and subsequent H-V coupling effect were investigated through computational simulations of a 3D numerical model of a hypothetical 3-story building in two configurations: base isolated with lead-rubber bearings (LRBs) and conventionally supported (i.e. fixed at the base). The parameters investigated included superimposed mass induced eccentricities, vertical acceleration intensity, vertical stiffness of the isolators, and slab design assumptions. In both hypothetical isolated and conventional buildings, a base model (‘BM’) representing a symmetric system did not lead to amplification of horizontal acceleration demands in the presence of vertical excitation (i.e. no H-V coupling). However, the BM median slab acceleration amplifications were considerable in both buildings. This model was used as a baseline comparison to the response of parameter variations. Both hypothetical building models had superimposed mass concentrations of 2%, 5%, or 10% of the floor effective seismic weight, applied as area loads to shell elements at specific locations on each floor. Mass eccentricities were observed to considerably alter the frequency of vertical modes, and thus had the most
influence on the vertical and coupled response relative to the other parameter variations considered. In addition, the direct implications of vertical slab vibrations and the H-V coupling behavior on the design forces of nonstructural components and systems (NCSs) were evaluated.

5.2 Conclusions

The numerical simulations in Chapters 3 and 4 supported the following observations and conclusions:

1. Slab acceleration amplification factors (i.e. vertical accelerations over height relative to peak ground acceleration) observed in the E-Defense experiments ranged from 2 at the 2nd floor, to 7 at the roof. Results from all the superimposed mass induced eccentricities considered on the hypothetical 3-story isolated and fixed buildings indicated median (over the ground motion suite) slab amplification factors ranging from 2.5 to 6.5. In the E-Defense experiments, the slab amplifications were observed in both building configurations at slabs located directly beneath the supplementary roof mass, which raises the question of the direct influence of significant mass eccentricities on the vertical response and subsequent H-V coupling effect. Thus, increased slab amplifications over building height may be particular to the atypical configuration of this test bed frame. In comparison, slab vibration amplifications were always larger at slab locations located near/directly beneath the supplemental roof mass at E-Defense, and the superimposed penthouse loads in the hypothetical buildings. Both of these masses were asymmetrically configured.

2. Slab spectra obtained from the E-Defense experimental results and the hypothetical
3-story buildings indicated most slab vibrations were dominated by a single-mode response. In the E-Defense building, the observed periods of slab vibration modes ranged from about 0.08 sec at the 2nd floor to 0.13 sec at the roof (7.7 to 12.5 Hz). Localized vertical modes dominated by vibration of individual slabs, identified in modal analysis, were in agreement with the dominant frequencies observed in the spectral response. In the hypothetical buildings, the periods of slab vibration modes ranged between 0.07 sec and 0.3 sec (3.3 to 14.3 Hz). These ranges are consistent with slab vibration frequencies of composite floor systems reported in the literature, which range between 3 and 13 Hz.

3. Slab acceleration amplifications and slab spectra in the E-Defense buildings were predicted by the numerical models with sufficient accuracy, which validated the experimental results. Slab spectral peaks were accurately predicted at each floor, where evaluations favored the accuracy of predictions of floor slabs with dynamic properties that led to increased vibration (i.e. period increasing with height and damping ratio decreasing with height). In the LRB/CLB configuration, relevant modal damping ratios were fine-tuned to match the experimental results at the 5th floor and roof slabs in particular. The level of accuracy of the numerical predictions indicates simple modeling assumptions are sufficient to replicate the vertical response of composite floor slabs.

4. Peak horizontal floor accelerations in both E-Defense buildings were captured well by the numerical simulations, with predictions generally within 20% of the experimental results for all the simulations in the test program. Reduced accuracy of the model was observed for motions with large vertical excitation. Horizontal
floor spectra in the E-Defense buildings were also well matched by the numerical
model, with differences mostly in amplitude of spectral peaks representing higher
modes, primarily due to lack of precision in modeling damping within the
superstructure.

5. Horizontal structural modes identified through modal analysis of both E-Defense
buildings were effectively matched to horizontal floor spectral peaks, including
higher modes that appeared in the response only during 3D excitations, which
suggested an H-V coupling behavior. The presence of each mode identified in the
simulation validated the experimental response of both buildings. The same
approach allowed for floor spectral peaks in the hypothetical buildings to be
effectively matched to structural modes.

6. In both E-Defense and hypothetical buildings, the contribution of higher modes
driven by vertical excitation (i.e. H-V coupled) was confirmed by comparatively
evaluating the floor spectra as the vertical acceleration intensity was increased
while keeping the horizontal input at 100% of the target. A structural mode with a
frequency of 9.45 Hz or $T = 0.11$ sec was positively identified as an H-V coupled
mode in the E-Defense LRB/CLB configuration. In the fixed-base building, two
structural modes with frequencies of 8.67 Hz ($T = 0.12$ sec) and 12.61 Hz ($T = 0.08$
sec) were also identified as H-V coupled. In the hypothetical isolated building, a
structural mode with a frequency of 9 Hz or $T = 0.11$ sec was positively identified
as an H-V coupled mode, while a comparable mode in the fixed-base building could
not be confirmed.
7. Peak slab accelerations and H-V coupling predicted by the E-Defense numerical models provided good estimates of the experimental response. Simple modeling assumptions used to effectively replicate the vertical and H-V coupled response of the superstructure included a composite floor system model consisting of frame elements for beams/girders and shell elements for floor slabs, and the use of the insertion point method with end joint offsets to represent composite floor bending stiffness. In addition, adequate discretization of the shell element mesh was necessary to distribute floor masses over the floors more realistically, and floor diaphragm constraints were limited to a few nodes. A constant modal damping of 2.5% was found to give a good match between experimental and numerical results. The assumptions demonstrated that slab vibration and H-V coupling can be predicted with standard modeling techniques without intensive refinement of details. The same modeling assumptions were applied to the hypothetical building models.

8. The influence of mass eccentric variations on the hypothetical buildings was evaluated by comparing horizontal floor acceleration amplifications for 2D (horizontal only) and 3D excitation. The evaluation indicated absence of H-V coupling in the fixed building, and low sensitivity to eccentric mass variations. In contrast, certain mass eccentricities applied to the isolated building induced participation of a higher structural H-V coupled mode.

9. Increase in 3D relative to 2D horizontal floor amplification factors across all the mass-eccentric cases considered in the hypothetical isolated building, was shown to indicate the extent to which the coupled mode participated in the response. The
greater the discrepancy in floor amplification factors between 3D and 2D input, the higher the participation of the coupled mode, meaning that the coupled mode was excited in the presence of vertical excitation. Induced mass eccentricity by a singular superimposed load of about 5% of the floor effective seismic weight, which is representative of a small penthouse on the roof, was enough to excite the H-V coupled mode in the isolated building. This confirms higher mode participation induced by vertical excitation is possible even in moderately asymmetric isolated buildings, but less likely in fixed-base buildings.

10. Spectral accelerations averaged over periods ranging from 0.1 s to 0.8 s (1.25 Hz to 10 Hz), applicable to a wide range of flexible NCSs, were normalized by peak floor accelerations to evaluate floor spectral amplification factors. For the conventional building, these factors were nearly the same for 2D and 3D excitation, and were not influenced by mass variability. In the isolated building however, increasing levels of eccentric added mass caused a decrease in the average spectral amplification factors on the 2nd and 3rd floors, but had negligible effect at the base and roof. The effect was attributed to the structural modes that engaged in the response over the period range considered.

11. Evaluation of slab accelerations in response to induced mass eccentricities, assumed vertical isolation properties, and slab design strategies (i.e. slab stiffness/mass variations) indicated that all variations led to slightly altered vertical modal frequencies. The modified frequencies had an unpredictable influence on slab amplification factors. Thus, vertical design forces of NCSs should conservatively account for a range of possible slab amplifications, which are not
reflected in current code provisions.

12. Statistical evaluation of amplification factors in the conventional building indicated greater unconservatism in the code recommended amplification factors in the vertical direction in comparison to the horizontal. A more conservative amplification factor should be applied to the vertical direction, perhaps on the order of 4-5. In the horizontal direction code recommended values appear to be adequate. For the isolated building which lacks corresponding code provisions, an average floor ‘reduction’ factor of 0.5 would appear to be an adequate design recommendation. In the vertical direction, similar amplification factors to the conventional building would be needed.

13. Predicted component amplification factors evaluated in both buildings that account for probable spectral amplification of the floor or slab acceleration (i.e. $a_p$ factor) were found to be in agreement with the code recommended limits (i.e. 1 – 2.5), while in the vertical direction, an $a_p$ factor limit of 2 might be more adequate.