ABSTRACT

Title of Thesis: AN EVALUATION OF FLOOR RESPONSE SPECTRA FOR ACCELERATION-SENSITIVE NONSTRUCTURAL COMPONENTS SUPPORTED ON REGULAR FRAME STRUCTURES

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This study evaluates the acceleration response of elastic nonstructural components (NSCs) subjected to earthquake-induced supporting structure motions. The objective is to provide insight into the development of the floor response spectrum (FRS) and its dependence on critical ground motion and structural system parameters such as the ground motion intensity, modal periods of the supporting structure, fundamental period of the NSC, strength of the structure, and location of the NSC with respect to the height of the supporting structure. The focus is on NSCs supported on regular moment-resisting frames. Results indicate that the FRS is highly dependent on the ratio of the period of the NSC to the modal periods of the supporting frame, the level of inelastic behavior of the frames and the location of the NSC. This study demonstrates that in several cases these effects are not adequately represented in floor design spectra recommended by current building codes.

AN EVALUATION OF FLOOR RESPONSE SPECTRA FOR ACCELERATION-SENSITIVE NONSTRUCTURAL COMPONENTS SUPPORTED ON REGULAR FRAME STRUCTURES

by

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

This list of abbreviations also applies to the title blocks of all of the figures in this paper.

a_p	Component amplification factor
BH	Beam hinge failure mechanism
ϕ	Mode shape ordinate
FRS	Floor Response Spectrum
γ	Base shear strength
i	Floor Level
K_1	Stiffness pattern (straight line first mode)
LMSR-N	Ground motion record set
MDOF	Multiple-degree-of-freedom
NSC	Nonstructural component
N	Number of stories, N=3, 9, and 18
PCA	Peak component acceleration
PFA	Peak floor acceleration, maximum absolute floor acceleration at a particular level
PGA	Peak ground acceleration
POM	Peak oriented hysteretic model
S_1	Strength design load pattern (parabolic)
$S_{aComponent}$	Spectral acceleration of the nonstructural component
$S_a(T_{B1})$	Spectral acceleration of the supporting structure at its fundamental period
SDOF	Single-degree-of-freedom
T_B	Period of the supporting structure (building)
T_{B1}	Fundamental period (first mode) of the supporting structure (building), T_{B1}=0.1N
T_{B2}	Second Mode Period of the supporting structure (building)
T_C	Period of the nonstructural component
$T_{Component}$	Period of the nonstructural component
ω_n	Natural frequency of the supporting structure
ξ	Percent of critical damping

Naming convention for plots and output data points (representative sample):

0311	03 = 3-story building, 1 = 0.25 Relative intensity, 1 = Node 1
18217	18 = 18-story building, 2 = 4.00 Relative intensity, 17 = Node 17

CHAPTER I: INTRODUCTION

Nonstructural components (NSCs) are objects in a building that are supported by the structure, but do not form part of the main gravity or lateral load resisting systems. Nonstructural components may consist of furniture, equipment, partitions, curtain wall systems, piping, venting systems, electrical equipment, bookcases, and many other items. Villaverde (1997b) provides an extensive list of nonstructural component types and groups them into three main categories: architectural components, mechanical and electrical equipment, and building contents.

Nonstructural components are sensitive to large floor accelerations, velocities, and displacements. When a building is subjected to an earthquake ground motion, the building can amplify this motion, resulting in floor accelerations higher than the peak ground acceleration (PGA). NSCs are subjected to these amplified accelerations, and if the natural periods of the NSC are close to those of the structure, the component can experience a peak component acceleration (PCA) that is much higher than the peak floor acceleration (PFA), which is defined as the maximum absolute floor acceleration at a particular level. Therefore, the PCA can be much greater than the PGA and cause severe damage to nonstructural components and their attachments to a structure.

The survival of NSCs during an earthquake event is important for maintaining the continuity of emergency services, for the safety of the public, and for mitigating the financial impact of the resulting damage. During the 1994 Northridge Earthquake, several hospitals were closed due to severe NSC damage (water damage, power systems, heating, lighting, etc.) (Hall, 1994, 1995). NSC damage can also represent a threat to life-safety, as falling and overturning NSCs can injure or fatally harm building occupants nearby. Data from past earthquakes in the United States (San Fernando 1971, Loma Prieta 1989, and Northridge 1994) shows that the direct and indirect costs associated with NSC damage can easily exceed the replacement cost of the structure (Scholl, 1984). Taghavi and Miranda (2003b) state that NSCs typically represent 65%-85% of the total construction cost of commercial buildings. They also note that NSCs typically fail at much lower deformation and acceleration demands than the supporting structure. Since the majority of occurring earthquake events are of small to moderate magnitudes, NSCs have a greater probability of experiencing a harmful ground motion.

In view of the importance of protecting the integrity of NSCs during seismic events, there is a need to carry out additional research studies to develop reliable performance-based design criteria for NSCs. This study contributes to the aforementioned purpose by evaluating the acceleration response of elastic nonstructural components as a function of ground motion and structural system parameters. The focus is on nonstructural components supported on regular moment-resisting frame structures.

CHAPTER II: OBJECTIVE OF THIS STUDY

This study focuses on the effect of building floor accelerations on the response of nonstructural components. Relationships are investigated between the floor response spectrum (FRS) and input variables, such as the ground motion intensity level, the strength of the structure, the fundamental period of the structure, and the location of the NSC with respect to the height of the supporting structure. The applicability of current building code methods for determining the acceleration response of NSCs is also addressed. The results of this study are intended to support the current efforts in performance-based earthquake engineering to create simple and transparent design methodologies for NSCs that correspond to various performance objectives.

CHAPTER III: PREVIOUS AND CURRENT STUDIES IN THE RESPONSE OF NSCs

Early research on the response of NSCs focused on safety-critical components in nuclear power plants. The study by Biggs and Roesset (1970) for the nuclear power plant industry was one of the first studies to provide a solution for the peak NSC acceleration response from the ground response spectrum. Meant as an aid in the design and analysis of plant equipment and piping, their study serves as the baseline comparison for many of the FRS studies that followed. Biggs and Roesset used the ground response spectrum to obtain the maximum modal accelerations and solve for the FRS for an elastic component mounted on an elastic structure. While assuming a series of damped harmonic inputs, Biggs and Roesset arrive at a theoretical equation that is modified based on the results of one ground motion comparison. Their proposed methods are approximate and empirical, but have been shown to produce conservative results in calculating the component amplification factor (PCA/PFA) when compared to time history analysis methods (Atalik, 1978).

Although the FRS method has some limitations, the resulting acceleration values outside of these limitations are conservative, as indicated by Villaverde (1997b) and Sing, et al. (1974). The United States Nuclear Regulatory Commission (NRC) has accepted the use of the FRS method, and has published guidelines for the development and use of floor design response spectra for the design of nuclear facilities (U.S. NRC, 1978). One of the methods implemented by the NRC to account for modeling uncertainties is to broaden the response spectrum around the peaks of all fundamental modes. This FRS peak-broadening is meant to account for uncertainties related to structural frequencies, damping, material properties of the structure, soil properties, soil structure interaction, and modeling techniques.

Other "simplified" methods to determine the floor response spectrum directly from the ground response spectrum require random vibration theory and power spectral density functions (Singh, 1974). Most of the methods proposed as a result of the nuclear industry's attention to the response of NSCs involve significant computation using time-history analysis, modal combinations, or statistical methods, all of which are too time-consuming for their implementation in the seismic design of ordinary structures. Realizing this limitation, current research focuses on eliminating the computational efforts and developing relationships for the NSC response that can be easily applied in the current building codes.

Much of the current research for NSCs is dedicated to the study of the PFA response of structures. The PCA has a strong dependence on the PFA, as the PFA represents the acceleration for low periods on the floor response spectrum. Understanding that the PFA is the starting point for the development of the FRS, researchers are currently trying to obtain more accurate estimates of the PFA.

Rodriguez, et al. (2002) performed an analytical study for regular buildings with rigid diaphragms, and propose a new method for obtaining floor accelerations. This method, based on modal superposition, is modified to account for the inelastic nature of the supporting structure. Their method also takes into account the higher mode effects and assumes elastic NSCs (Rodriguez et al., 2002).

Taghavi and Miranda (2003a, 2003b) implement a continuum model in their study of PFAs using a time-history analysis. The fundamental period, the type of structural system, the lateral stiffness variations with height, and the assumed damping for the first three modes of the structure define their analytical building model. Their work is primarily in the area of PFAs for linearly elastic structures, although a recent paper (Miranda et al., 2003b) has included a comparison of their analytical estimate of a FRS and the FRS from a recorded floor acceleration time history.

These previous studies on the acceleration response of NSCs serve to highlight the major characteristics and considerations that significantly affect the response of a NSC. The most important of these considerations, the dynamic interactions between the NSC and the building, the fundamental periods of the structure and the NSC, the damping in the system, the type of structural support system, and the location of the NSC within the structure, will be discussed in the following sections.

Recent research into the area of NSCs is a result of the increased focus on performance-based earthquake engineering. Performance-based earthquake engineering allows building owners to go above and beyond the life-safety intent of current building codes, and assign even greater levels of protection for their structures and NSCs in the event of an earthquake. Performance-based earthquake engineering attempts to quantify the impacts of various structural damage types, and assign parameters that relate the earthquake motion and the building to the amount of resulting damage. These parameters (accelerations, velocities, and displacements) are currently being studied to obtain more accurate predictions and prevent NSC damage.

This study provides quantitative information on the dependence of the response of NSCs on ground motion characteristics and the structural properties of regular frames. This information is deemed necessary for the development of performance-based design criteria for acceleration-sensitive nonstructural components that are expected to behave linearly elastic when exposed to floor accelerations.

CHAPTER IV: FACTORS INFLUENCING THE ACCELERATION RESPONSE OF NSCs

Dynamic Interaction Between NSC and Primary Structure

Analyzing the NSC and the supporting structure in one coupled model accounts for the dynamic interaction of the NSC and the supporting structure, and can provide a more accurate result for the NSC response. This method of including the NSC in the main structural model is impractical (Biggs, 1970b) and rarely used due to the large computational effort involved, software modeling limitations, and the inefficiencies that may occur in the design process (Villaverde, 1997b).

The response of NSCs is primarily a function of the ratio of the period of the component to the modal periods of the supporting structure. As noted by Biggs (1970a), there are three types of NSC response to floor response motions. If the NSC and its attachment to the structure are rigid, the maximum acceleration for the NSC is equal to the PFA. If the NSC is relatively flexible (longer period) compared to the supporting structure, then the component responds as if supported directly on the ground. When the periods of the structure and the NSC are close, resonance can occur, resulting in significant amplifications of the floor accelerations. This explains how the acceleration response of nonstructural components can be much higher than the PGA. This tuning of frequencies is often possible because of the low weight and stiffness values of the NSC, which can shift the fundamental frequency of the component closer to that of the supporting structure (Villaverde, 1997b).

The FRS method of NSC analysis, as implemented in this study, assumes a decoupled dynamic model for the NSC and supporting structure. Villaverde (1997b) indicates that the FRS method of NSC analysis is accurate for NSCs with small relative masses and frequencies away from the fundamental frequency of the supporting structure. Singh and Ang (1974) conclude that when analyzing the supporting structure, decoupling the system (no dynamic interaction considered) is acceptable for mass ratios up to 0.10. For the analysis of the NSC response, they recommend that the mass ratios not exceed 0.01 for a decoupled dynamic analysis. Igusa and Der Kiureghian (1985) claim that more economical results can be obtained for the NSC if the interaction effects with the structure are considered.

The NRC (U.S. NRC, 1978) recommends that the interaction between component and the supporting structure be considered when the component is a major equipment system whose stiffness, mass, and resulting frequency range make dynamic interaction possible. When the components are light, the interaction need not be considered, but the mass should be added to the mass distribution of the structural model (U.S. NRC, 1978). The American Society of Civil Engineers' (ASCE) standard for the seismic analysis of nuclear facilities (ASCE, 1987) indicates that a coupled analysis is required in cases such as flexible walls and floors that support equipment, where the interaction effects can be significant. This ASCE standard indicates that a coupled analysis is not necessary if the NSC's mass as a percentage of the supporting structure mass is less than 1%. Amin, et al. also indicate that for mass ratios less than 1% the conservatism that results is not appreciable, and therefore the interaction of the primary and secondary systems can be ignored for mass ratios less than 1% (Amin et al., 1971). For NSCs supported at only one point, the ASCE document (ASCE, 1987) provides a relationship between the modal mass ratio of the NSC and the supporting structure, and the frequency ratio of the NSC and the supporting structure. As the mass ratio increases, and the frequency ratio nears a tuned condition, a coupled model that captures the interaction of the secondary and primary system becomes increasingly necessary. When the frequencies of the supporting structure and the NSC are not close, the mass ratio can be much higher before the interaction of both systems must be considered to reduce the conservatism.

These results match the criteria established in a study by Igusa and Der Kiureghian (1985b) for two-degree of freedom equipment-structure systems. Their decision to consider the interaction effects of the system can be based on the following formula:

$$\gamma_i < 4 \cdot e \cdot \zeta_i \cdot \zeta_e \cdot \left[1 + \frac{\beta_i^2}{\left(\zeta_i + \zeta_e\right)^2} \right] \qquad \textbf{(Equation 1)}$$

where γ_i mass ratio (mass of NSC / mass of supporting structure)

β_i tuning parameter that relates the NSC and supporting structure frequencies

ζ_i and ζ_e critical damping percentages for the structure and the NSC

This formula shows that when the system is away from resonance, large values of $\beta_{i,}$ then large values of the mass ratio are necessary before consideration of the dynamic interaction becomes necessary. When the system is near resonance, values for β_i approach zero, and extremely small mass ratios are necessary before the interaction can be neglected (Igusa and Der Kiureghian, 1985a).

The above equation by Igusa and Der Kiureghian is developed for simple two-degree of freedom systems. Singh and Ang (1974) have shown that the over-estimation of the amplification for the two-degree of freedom system is greater than that for a multi-degree of freedom system. Realizing that most buildings are multi-degree of freedom structures, and following the recommendations of Singh and Ang (1974), Amin et al. (1971), and the NRC (U.S. NRC, 1978), it seems appropriate to disregard the component and structure interaction when mass ratios are less than 1%. Even if this were incorrect, the results obtained when ignoring the interaction effects would be conservative as indicated above.

Current building codes such as the 1997 Uniform Building Code (*UBC*, 1997) and the 2003 International Building Code (*IBC,* 2003) only require consideration of the supporting structure and NSC interaction effects in high seismic areas (Zones 3 and 4 – UBC 1997; Seismic Design Categories D, E, or F – IBC, 2003), when the structures support flexible NSCs with a weight greater than 25% of the structure's weight.

In this study, the dynamic interaction between the supporting structure and the elastic NSC is not considered; therefore, results apply to NSCs with small masses relative to the total mass of the frame structure.

Percent of Critical Damping

Damping has a considerable effect on maximum floor accelerations. If the damping ratio of the supporting structure or the NSC were over-estimated, this would create unconservative acceleration results for the NSC. The choice of a proper damping ratio is therefore critical to the analysis of NSC response.

Current building codes (*IBC*, 2003) suggest that 5% of critical damping be used for the supporting structure. Villaverde (1997a, 1997b) consistently assumes very low damping ratios (0% and 0.1%) for nonstructural components and 5% for the supporting structure. Other studies by Miranda and Taghavi (2003b) assume NSC damping of 5%.

Biggs and Roesset (1970b) assign damping ratios of 5% and 2% for the supporting structure and NSC, respectively. The analysis implemented in this study assumes a damping value of 5% of critical damping for both the structure and the NSC.

If the dynamic interaction of the NSC and the supporting structure is deemed significant, then the damping of the combined system will be in between that of the NSC and the supporting structure. Villaverde (1997a) suggests that the damping value for the combined system is most likely around the average of the two damping values. Not only will the value be in between the two bounds, but also the combined system can exhibit non-classical damping. Non-classical damping gives rise to "complex-valued mode shapes" (Villaverde, 1997a). If the vibration modes of the NSC and the structure are not in resonance and the fundamental frequency of the NSC is sufficiently away from that of the supporting structure, then the system is classically damped. If the system is tuned, and the interaction must be considered, then the system is non-classically damped (Igusa and Der Kiureghian, 1985a).

Although studies by Miranda, et al. (2003a) assume a classical damping model, alternate studies by Singh and Suarez (1987) indicate that the effect of non-classical damping can be important for light equipment. If this light NSC is tuned with a dominant mode of the supporting structure and has damping values much lower than those of the supporting structure, neglecting the non-classical damping effect would result in unconservative results. For the highest modes of vibration for the supporting structure, consideration of the non-classical damping yields very little difference in the acceleration results when compared to an analysis where the complex damping is not considered (Singh and Suarez, 1987).

Through the use of the Rayleigh damping matrix, damping is accounted for in the time-history analysis of this study. Rodriguez, et al. (2002) also assume a Rayleigh damping formulation for their study. Villaverde (1997a) on the other hand, has indicated that Rayleigh damping can cause significant error in the calculation of the damping matrix if the damping ratios of the structure and the NSC differ by orders of magnitude. As this work assumes that the dynamic interaction of the NSC and the supporting structure is not significant, and the damping ratios for both the NSC and the supporting structure are 5%, then it is acceptable to use the Rayleigh damping matrix.

Structural System of the Building

The structural system of the supporting structure affects the acceleration response of the floors, and thus the NSC. In general, upon going inelastic, flexural frames (braced frames) yield at the base, while shear frames (moment frames) yield over the height of the building. This gives rise to different mode shapes and thus different acceleration distributions for flexural and shear frames. For flexural beam type structures, the relative contribution of higher modes to the response would be more significant in the elastic range than in shear beam buildings. However, for the same building period, moment-resisting frames exhibit larger first-mode accelerations as compared to flexural frames (Miranda and Taghavi, 2003b). This study uses regular moment-resisting frame structures, which are designed as stiff frames with building periods equal to T = 0.1N (N = number of stories).

Miranda and Taghavi's (2003b) investigation of the PFA includes a parameter that varies the lateral stiffness along the height of the building. Their results show that reducing the lateral stiffness along the height of the supporting structure has a negligible effect on the dynamic characteristics (mode shapes, participation factors, and period ratios) of the supporting structure for flexural beam type buildings. The variation of lateral stiffness over the building height has a greater, but still not significant, effect on the dynamic characteristics for moment frame structures (Miranda et al., 2003b). This study assumes a uniform stiffness distribution over the height for all frames since the stiffness distribution along the height is not considered a critical parameter for the estimation of floor accelerations.

Non-linear Behavior and Overstrength of the Supporting Structure

Nonlinear Behavior

For the non-linear seismic design of NSC supporting structures, the spectral acceleration values are reduced by the response modification factor, i.e., R-factor. Current building codes implement a separate response modification factor for NSCs, which reduces the acceleration response of the NSC, while realizing that the same modification factors cannot be used for the structure and the NSC. When accounting for inelastic action of the supporting structure, the PFAs are reduced. Likewise, inelastic action of the supporting structure also reduces the PCA. This reduction is greatest at the upper most floors of the supporting structure (Rodriguez et al., 2002). A number of other studies have looked at the effect of non-linearity in the supporting structure using a variety of analysis methods (Villaverde 1987, Lin and Mahin 1985).

Rodriguez et al. (2002), in a study similar to the one presented in this paper, implement a non-linear time-history analysis using scale factors to vary the inelastic nature of the structure. By scaling the ground motion inputs and using two different hysteresis rules, the inelastic response of the structure is analyzed. When the scale factor is very low, the magnification of the ground acceleration is constant, because the building is responding elastically. As the scale factor increases, the normalized floor accelerations (PFA/PGA) decrease and have a smaller dependence on the scale factor as the scale factor becomes large. Therefore, their research concludes that the maximum floor acceleration magnifications occur when the structure responds elastically and this magnification decreases as the inelastic response (ductility demand) increases. As the number of stories increases, the ductility demand of the structure has less effect on the floor acceleration magnification of the PGA at the roof of the structure (Rodriguez et al. 2002).

According to their study, the reduction in the acceleration response of nonstructural components due to the inelastic action of the supporting structural system is the greatest for the period of the first mode of the supporting structure (Rodriguez et al. 2002). Due to these results, their approximate solution for floor accelerations assumes that the first mode of the structure is the only mode affected by the inelastic action (ductility). It will be shown in a later section that the data of this study matches these results obtained by Rodriguez, et al. (2002).

Miranda, et al. (2003c) and Villaverde (1997b) suggest that more research is needed to quantify the proper response modification factor to account for the inelastic action of the NSC and its attachments to the structure. Taking credit for the inelastic action of the NSC seems contrary to the efforts intended to protect NSCs from damage and loss of functionality. Allowing the NSC or its attachment(s) to plastically deform would cause permanent damage. Therefore, the inelastic effects of the NSC should only be considered if the component is designed to yield (or plastify) in localized areas, and still function in the aftermath of the earthquake, or require little repair effort in the post-earthquake reconstruction period.

Overstrength

When design codes recommend response modification factors for the determination of the PFA and the subsequent PCA, the effect of overstrength in the supporting structure must be considered. The yield strength of designed structures is often greater than the design yield strength level due to conservative material strengths, safety factors, and subjective design decisions. The NSC is therefore exposed to higher accelerations because the supporting structure would have increased elastic action before entering the non-linear range. When providing design recommendations based on their approximate solution for peak floor accelerations, Rodriguez, et al. (2002) suggest a response modification factor of $\mu/2$, where μ represents the ductility of the system, to reduce the elastic seismic forces and account for ductility in the structural supporting system (inelastic response). The selection of this value includes the consideration that this modification factor must be less than the one used in the design of the building due to the effects of overstrength in the structure. Villaverde (1997a) also notes the importance of overstrength in the supporting structure, and suggests a modification factor equal to one-half the one used in the design of the structure. Villaverde believes that the response modification factor for NSCs is the most critical factor leading to over or under conservative designs (Villaverde, 1997a).

CHAPTER V: OVERVIEW AND ANALYSIS METHODOLOGY

The main factors that influence the acceleration response of NSCs that are addressed in this study are the relationship between the period of the NSC and the modal periods of the supporting structure, the nonlinear response of the supporting structure, and the height of the component in the structure. The desired output of the analysis is a floor response spectrum, representing the response of an elastic, SDOF nonstructural component. A method of analysis is developed to understand the relationship between the floor response spectrum and relevant ground motion and structural characteristics. These characteristics include the ground motion frequency content, intensity level, fundamental period of the structure, and location of the NSC with respect to the height of the supporting structure.

The ground motion frequency content is controlled by the selection of ground motion records with similar spectral shapes. Therefore, one set of ground motion records is used consistently throughout this study. For each ground motion record and structural model, two time history analyses are performed to study the dependence of the behavior of nonstructural components on the structure's relative intensity level. The relative intensities of interest are 0.25 and 4.0, which correspond to elastic and moderately inelastic behavior, respectively. The relative intensity parameter is defined as $[S_a(T_{B1})/g]/\gamma$ (Medina and Krawinkler, 2003), and it is a measure of the ground motion intensity level relative to the base shear strength of the frame structure. This relationship is defined using the 5% damped spectral acceleration at the first mode period of the frame, $S_a(T_{B1})$, and the base shear coefficient. The base shear coefficient, γ, relates the yield base shear to the weight of the structure, $V_y = \gamma W$, as seen in current seismic codes

(*IBC*, 2003). The relative intensity measure is equal to the ductility dependent strength reduction factor, R_μ. This is the ratio of the elastic strength demand to the yield strength of an inelastic system. R_μ is equal to the response modification factor, R, of the current building codes if there is no overstrength in the structure.

The dependence of the NSC response on height in the building and number of floors is studied using three different regular-frame building models of varying heights, i.e., 3, 9 and 18 stories. The relationship between the NSC response and the modal periods of the structure can be seen due to the fact that the building periods change for each structure.

The method of analysis allows for an assessment of the applicability of current code provisions based on this limited study. The analysis provides insight into the component amplification and the effect of building non-linearity, for both of which a limited amount of research is currently present. The linear and non-linear time history analyses help address the effect of structural non-linearity on the response of an elastic nonstructural component. In order to study the component amplification factor (a_p), it is necessary to obtain the peak floor accelerations and the peak component accelerations, such that a comparison can be made. The PFA and the PCA are recorded for each analysis and the results are presented in the sections below.

Methodology

The analysis used in this study is a computer-based method. Simplified building structural models (regular frame models) are subjected to ordinary ground motions (Medina and Krawinkler, 2003) to obtain the acceleration response of selected building floors. Three buildings with strengths corresponding to the two aforementioned relative intensities are subjected to forty ground motions each. Each building model, relative intensity and ground motion combination is input into the non-linear time history analysis software, DRAIN-2DX (Prakash et al. 1993). The resulting floor acceleration time history outputs (see Figure 1) are then used as input into a single degree of freedom (SDOF) analysis program to obtain an elastic response spectrum for each floor time history input. Figure 2 depicts a map of the analysis process used in this study, showing the input choices and outputs for each stage of the analysis. Table 1 displays all of the possible combinations of structural input variables (number of stories, relative intensity, floor level) for this analysis. Each structural model combination indicated in Table 1 is analyzed for the entire set of 40 ground motions.

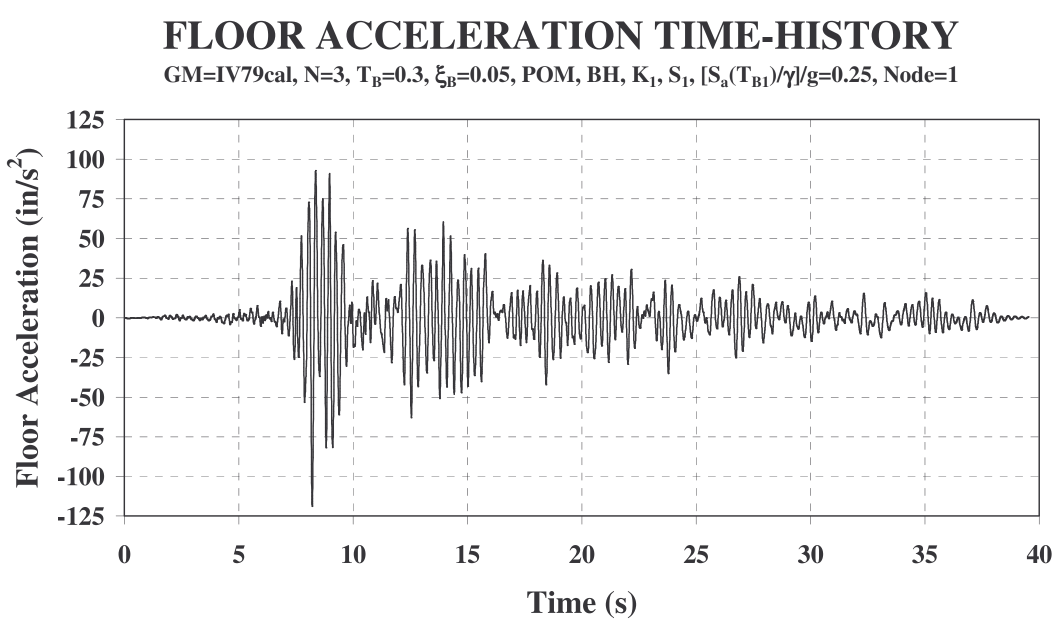

Figure 1 – Sample DRAIN-2DX Floor Acceleration Time-History Output

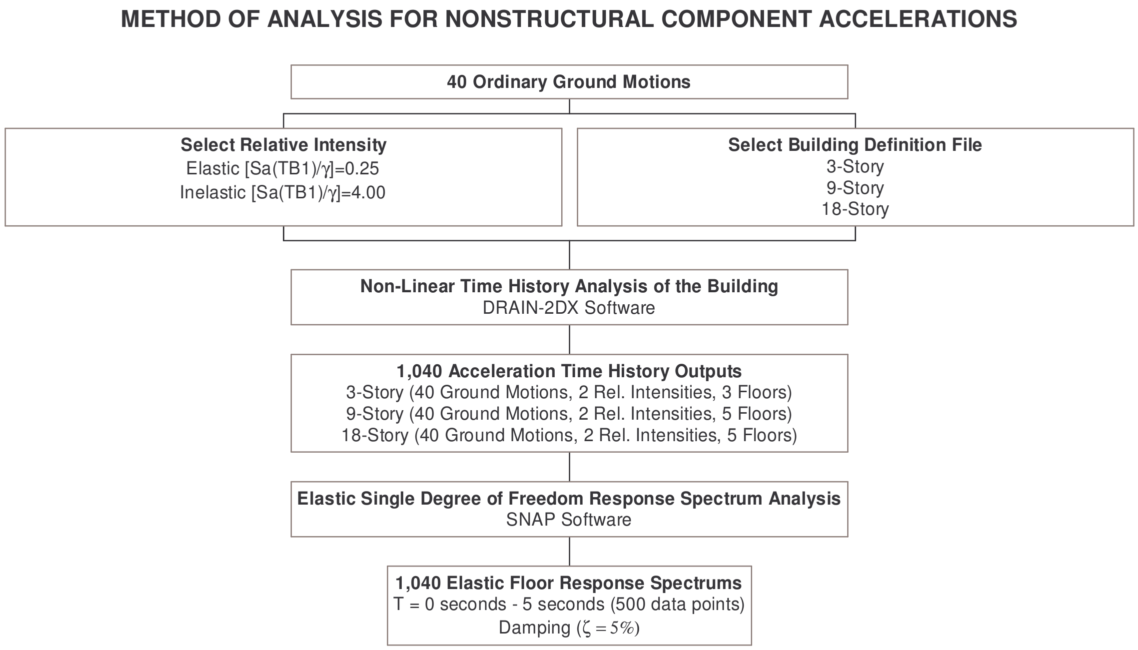

Figure 2 – Method of Analysis for Nonstructural Component Accelerations

The 3, 9, and 18-story structures are meant to provide a representative set of building heights for systems whose lateral load resisting system is composed of isolated moment-resisting frames. The results of this study provide ample data for small height, medium height, and tall frames.

The time-history analysis implemented in this study only reflects the results of seismic loading in one direction. The effect of other building vertical or lateral loads, and their influence on the dynamic response of the structure is beyond the scope of this study. The current building codes require that the horizontal and vertical seismic forces be combined in addition with other load cases, using required loading combinations. This study is only concerned with the response of frames in one direction, which is a reasonable approach for regular frames, i.e., frame structures that do not exhibit significant vertical or plan irregularities.

The multi-degree of freedom (MDOF) analyses and SDOF analyses completed as part of this study do not account for the dynamic interaction between the frame and SDOF NSC. As seen earlier in this report, there are guidelines by which this interaction must be considered. The effect of multiple NSC support excitations is also neglected in this analysis.

Table 1 – 26 Combinations of Structural Input Variables Analyzed for 40 Ground Motions Each

Number of Stories N	Relative Intensity $[S_a(T_{B1})/g]/\gamma$	Floor Level i
3	0.25	2
		3
		4
3	4.00	2
		3
		4
9	0.25	2
		4
		6
		8
		10
9	4.00	2
		4
		6
		8
		10
18	0.25	3
		7
		11
		15
		19
18	4.00	3
		7
		11
		15
		19

Nonstructural Component SDOF Analysis

Representative floor levels are selected to study the variation of floor response spectra with height. While the accelerations on each level of the 3-story structure are recorded, only 5 floors from the 9 and 18-story frames are studied. The selected floors from the 9 and 18-story frames are evenly distributed throughout the height of the building, therefore still providing a near-complete picture of the distribution of acceleration over the height of the building. Each building model and the floors for which data is recorded can be seen in Figure 5.

The floor acceleration time history outputs from DRAIN-2DX for these selected floors are input into a single-degree of freedom analysis software entitled SNAP, which is an in-house SDOF analysis program developed at Stanford University. The model analysis in SNAP represents the calculation of the elastic response of a range of component fundamental periods for the floor acceleration time history. The percent of critical damping specified for all analyses is 5%.

The plot of this acceleration response spectrum output can be viewed in Figure 3.

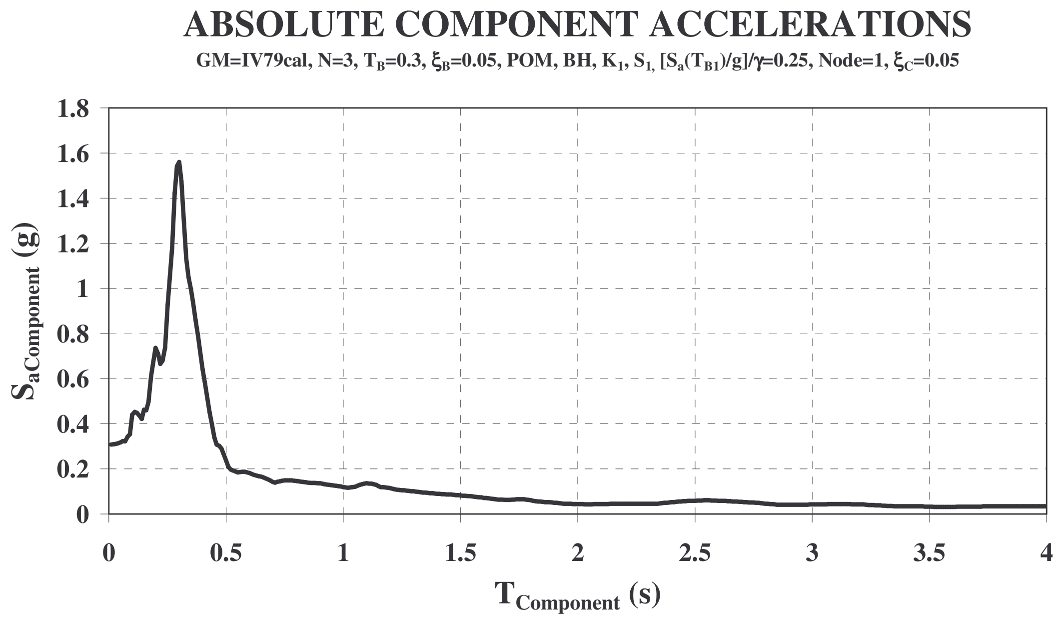

Figure 3 – Sample SNAP Floor Response Spectrum Output

Modeling Uncertainties

All analyses based on simulations are subject to inherent uncertainties in the modeling process. There are uncertainties in the structural frequencies, material properties of the structure, and the modeling techniques (i.e. connections and floor slabs). Building models also differ from reality because the damping mechanisms in real buildings are not well understood, and the soil-structure interaction is not usually represented in the models. As previously discussed, the Nuclear Regulatory Commission acknowledges these modeling uncertainties and adjusts for some of them by simply broadening the peak of the floor response spectrum.

CHAPTER VI: INPUT GROUND MOTIONS

This study is carried out using the LMSR-N set of 40 ordinary ground motions selected by Medina and Krawinkler (2003). The ground motions in the LMSR-N database were recorded on NEHRP site class D soils (stiff soils), between 13 km and 40 km from the fault rupture area, and have moment magnitudes between 6.5 and 6.9 (Medina and Krawinkler, 2003). Qualitatively, it is expected that the results from this study apply to stiffer soils and rock. These ground motions do not consider soft-soil, near-fault, or long-duration characteristics.

The input ground motions of the LMSR-N set are comparable in shape to the design response spectrum (*IBC*, 2003), although with much lower magnitude values. Since this study implements the relative intensity measure $[(S_a(T_{B1})/g)/\gamma]$ to carry out the elastic and inelastic analyses, the absolute intensity of the ground motion is not a critical parameter. This latter statement is true as long as the frequency content of the ground motions is an adequate representation of the ground motion hazard represented by the IBC 2003 design response spectrum. Medina and Krawinkler (2003) demonstrated that this is a reasonable assumption for this ground motion set.

Figure 4 displays a sample ground motion input time history plot for one of the ground motions used in this study.

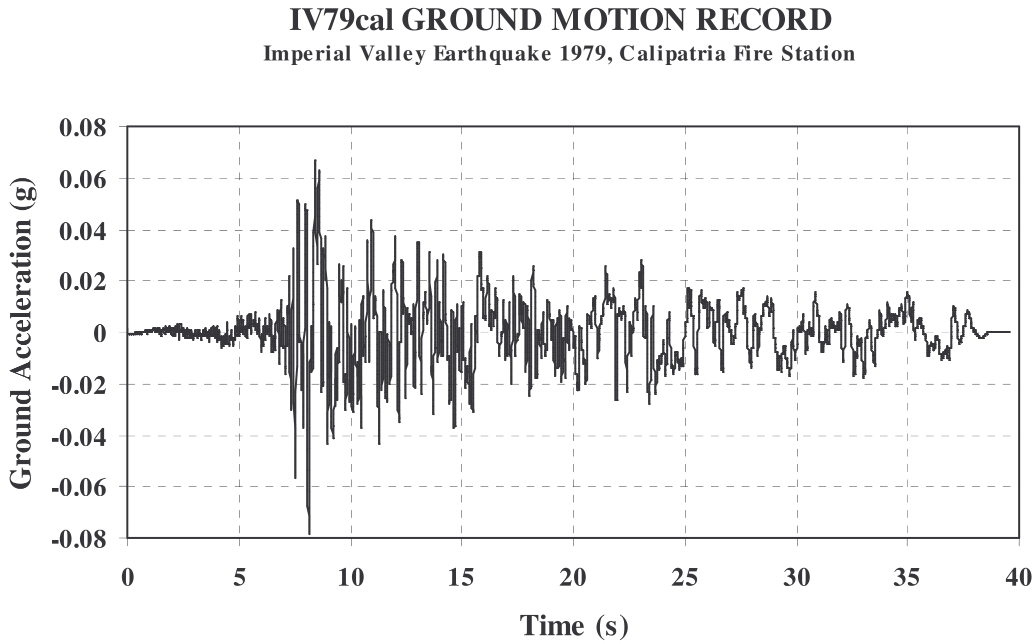

Figure 4 – IV79cal Ground Motion Record Input to DRAIN-2DX

CHAPTER VII: BUILDING MODELS

Three building models (3, 9 and 18-story frames) developed by Medina and Krawinkler (2003) are used in this study. The fixed base moment frame buildings have a uniform mass distribution over their height. Each building is a two-dimensional, single-bay, nondeteriorating, regular moment frame, measuring 24 feet wide, with 12-foot high stories. Figure 5 displays the three structural models used throughout this study, and indicates the selected floors for which data has been recorded. A DRAIN-2DX input file is created for each frame, defining the structural properties and dimensions of the model.

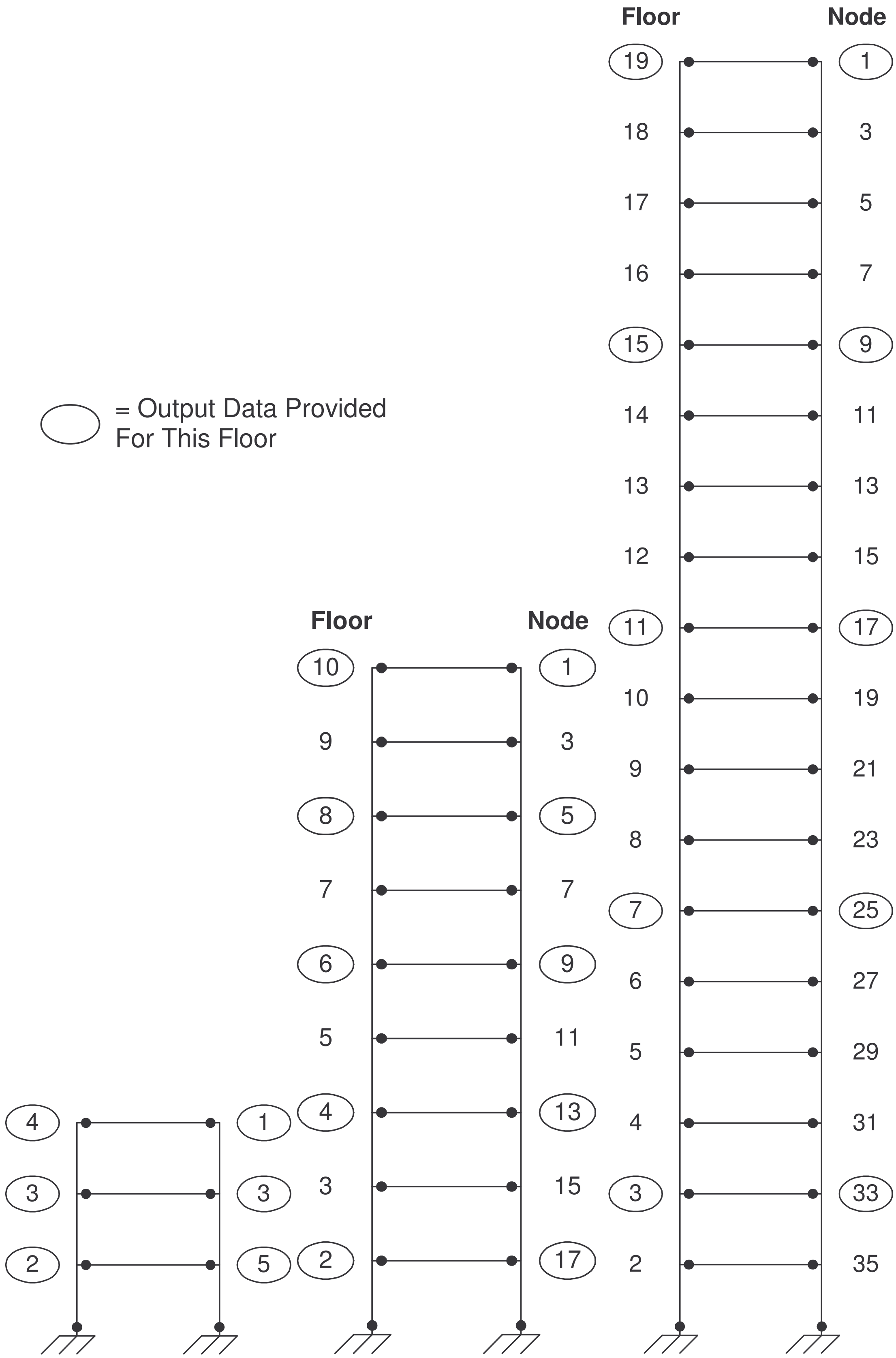

Figure 5 – 3, 9, and 18-Story Regular Frame Structural Models

The frames are designed based on the strong column-weak girder philosophy and infinitely strong columns are used. A beam-hinge mechanism develops when the building is subjected to a parabolic load pattern, which corresponds to a $k = 2$, NEHRP load pattern (*NEHRP*, 2000). In this study the same load pattern is used in all buildings, regardless of height, to have a consistent strength distribution over the height.

The frames are designed to exhibit a bilinear pushover curve when subjected to the parabolic load pattern. This bilinear pushover curve indicates that each of the hinges yield simultaneously. The location of plastic hinges can be seen as • symbols in Figure 5 at the base of the first story columns and the ends of the floor beams. It is through the use of these hinges that the nonlinear behavior of the structure is modeled. These hinges are rotational springs defined by a peak-oriented, moment-rotation relationship. This peak-oriented model includes a 3% strain-hardening region in the moment-rotation curve.

Each of the structures has a first mode fundamental period of $T_{B1} = 0.1N$, where N is the number of stories in the structure. $T_{B1} = 0.1N$ is considered a lower bound for fundamental periods of moment-resisting frame buildings when compared to results obtained by Goel and Chopra (1997) for real buildings.

The beam to column stiffness ratio can affect the fundamental period, the separation of the natural periods, and the mode shapes for a given supporting structure (Chopra, 2000). With low stiffness ratios, the building acts as a flexural beam, and as the stiffness ratio increases, the frame behaves more like a shear beam. The beam to column stiffness ratio (ρ) is set to maintain a linear first mode shape for each frame, as seen in Figure 6. Additional mode shapes for each building model can be seen in Appendix A.

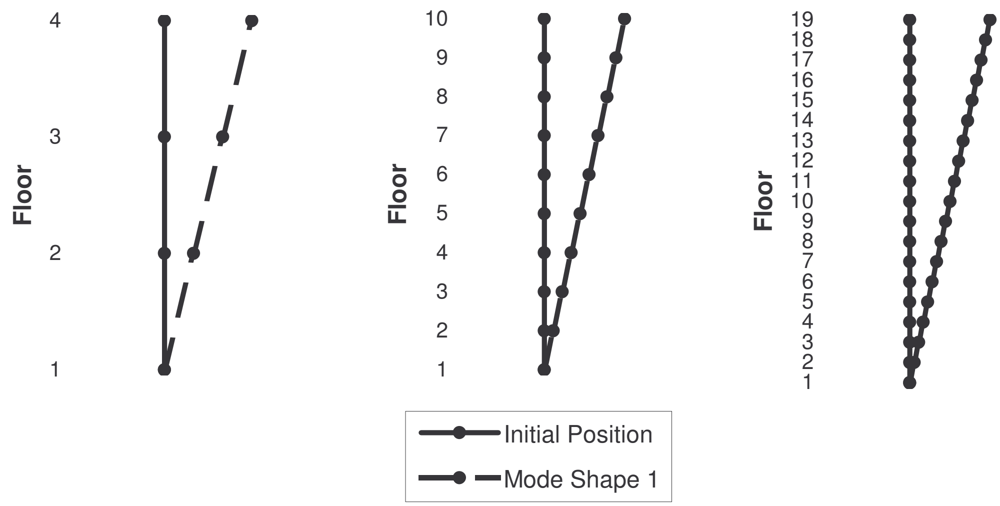

Figure 6 – 3, 9, and 18-Story First Mode Shapes

Medina and Krawinkler (2003) show that single-bay frame models are adequate to obtain an understanding of the response of multi-bay regular frame structural models of varying degrees of inelasticity. However, three-dimensional effects (i.e. torsional effects), which become significant when structural irregularities are present, have not been studied in this analysis. As suggested by Villaverde (1997b) torsional effects of the building could increase the acceleration response of nonstructural components.

CHAPTER VIII: DATA ANALYSIS

Throughout this study, a lognormal probability distribution is assumed for the floor acceleration response of NSCs. This probability distribution is verified using the Kolmogorov-Smirnov Test (K-S Test). The K-S Test is performed on random sets of 40 acceleration data points from various period and relative intensity combinations for the 3-story and 18-story structures. The K-S test, as shown in Figure 7, for the 11th floor of the inelastic 18-story structure, is representative of all the K-S tests performed on the randomly selected output samples. Figure 7 indicates that the probability distribution of the acceleration data can be represented by a normal or a lognormal distribution. It can also be observed that the data more closely follows a lognormal probability distribution than a normal distribution. The lognormal probability distribution is clearly a better distribution hypothesis near the fundamental frequency of the supporting structure. Similar results are obtained from other randomly selected output data sets.

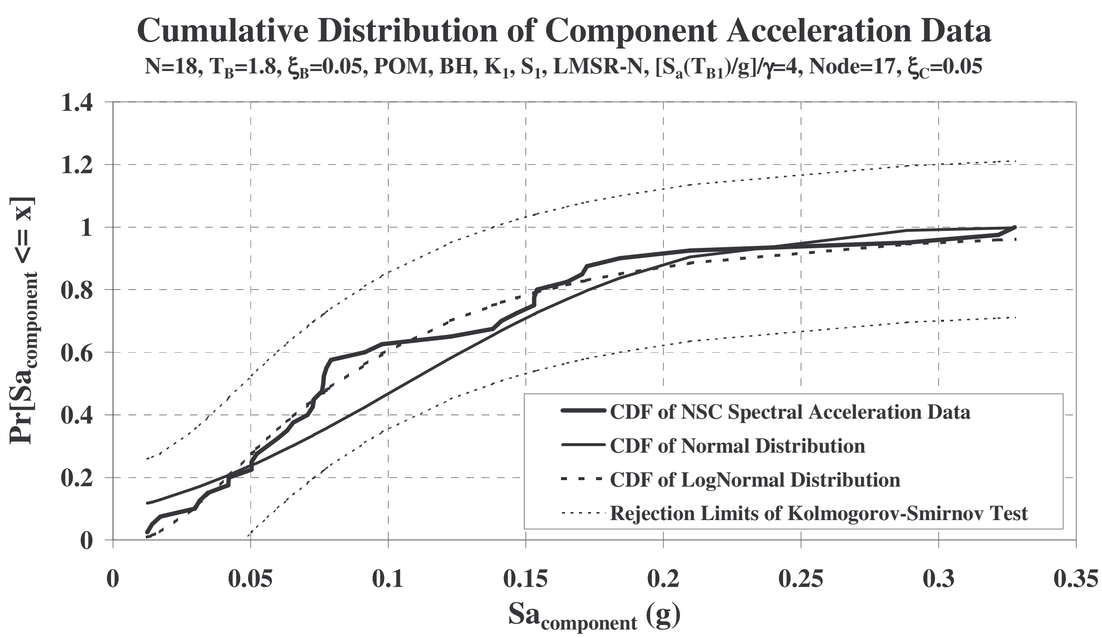

Figure 7 – Cumulative Distribution and Kolmogorov-Smirnov Test for NSC Spectral Acceleration Data

For a lognormal distribution, the median of the data is used as the measure of central tendency. The scatter of the data around the median is reflected in the standard deviation. The 16th and 84th percentiles of the data indirectly represent the standard deviation. Using a set of 40 sorted output data points, the average between the 20th and 21st values is the median, while the average between the 6th and 7th values is the 16th percentile, and the average between the 33rd and 34th values is the 84th percentile. Figure 8 displays the elastic floor response spectra output for the roof of the 18-story elastic frame. Figure 9 shows the elastic floor response spectra output for the roof of the 18-story inelastic frame. These plots clearly indicate the median as well as the 16th and 84th percentiles for the resulting output data from the use of 40 ground motion inputs. These three values are recorded for each of 26 analyses, for the absolute component accelerations and the normalized component accelerations ($S_{aComponent}$/PFA), and will be used to discuss and compare the output data trends in the following sections.

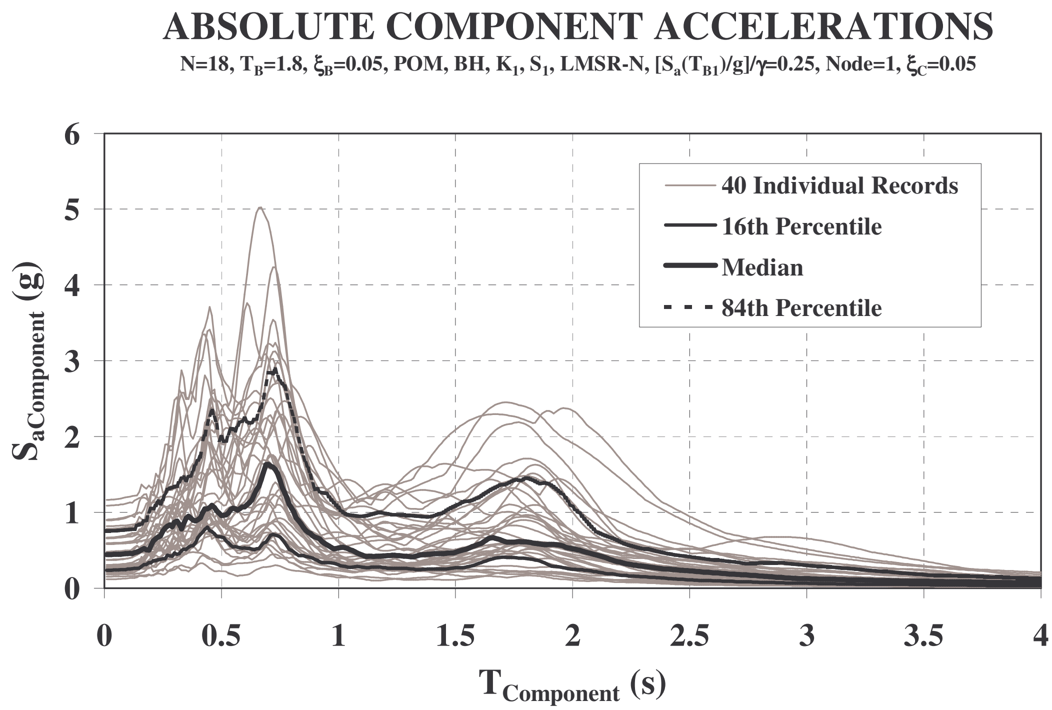

Figure 8 – Sample Elastic Floor Response Spectrum

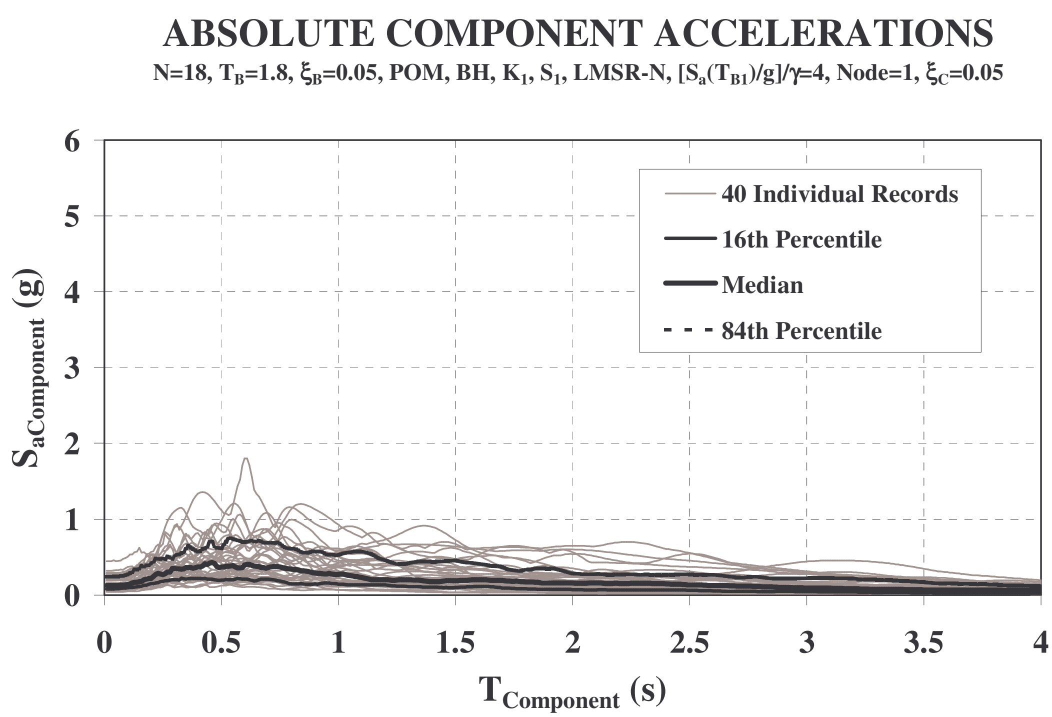

Figure 9 – Sample Inelastic Floor Response Spectrum

CHAPTER IX: RESULTS / DISCUSSION

A comparison of the floor response spectra for all of the 1,040 combinations indicated in Table 1 shows that the maximum acceleration response of the elastic SDOF nonstructural components ranges between 0.8 and 28.7 times the PGA. Due to the wide range of amplification values, a focus on the key drivers of NSC acceleration response is needed. It is important to remember that most of the data presented in this section is in terms of median values, which represent central tendencies in the data. Therefore, there exists response values that both exceed and are below the median values presented in the above plots.

Effect of the Fundamental Period on the Response of NSCs

Figures 8 and 9 indicate that the NSC acceleration response experiences significant amplification near component periods that are in tune with the modal periods of the supporting structure. The FRS peaks in Figures 8 and 9 correspond with the periods for the first three modes of the 18-story structure (1.8 s, 0.73 s, and 0.45 s). Therefore the fundamental period ratio between the NSC and the supporting structure is a critical parameter that defines the shape of the FRS. Implementing this ratio also enables comparisons between all of the frames by eliminating the period dependence so that the dependence on height (i.e., number of stories) can be evaluated.

As indicated earlier in this report, the NSC acceleration is closely related to the peak floor acceleration through the component amplification factor. Therefore, an understanding of the PFA distributions for various frames can lead to better predictions of NSC acceleration. The results from this study support the conclusions drawn by

Medina and Krawinkler (2003) and Miranda and Taghavi (2003b) that the PFA is highly dependent on the period of the supporting structure (Figure 10).

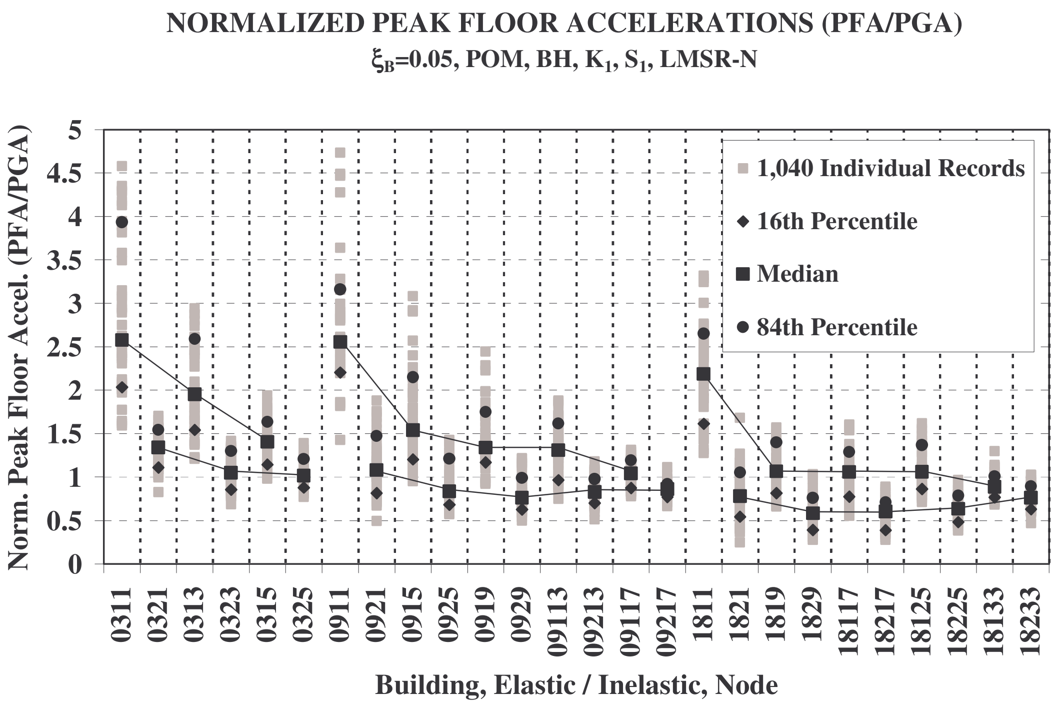

Figure 10 – Normalized Peak Floor Accelerations for All Records

Figure 10 indicates that higher normalized PFAs (PFA/PGA) occur for shorter period frames. As the frame period increases, the normalized PFAs decrease and become more uniform over the height of the building. The exception is the top floor (roof) of the frame because of the effect of higher supporting structure modes and the building stiffness distribution. Figure 10 also shows that for longer period elastic systems, the maximum floor acceleration is at the roof. Medina and Krawinkler (2003) obtained similar results for PFAs, and also concluded that with an increase in inelastic behavior and structural period, the peak values move to the lower floors.

Supporting Structure Higher Mode Effects

This study demonstrates that the maximum FRS acceleration values do not always occur at the fundamental period of the supporting structure. This behavior highlights the importance of a NSC with a period in tune with the higher mode periods of the supporting structure (Figure 11). These results are consistent with those obtained by Bachman (2003), Miranda and Taghavi (2003b), and Rodriguez et al. (2002) for different systems and ground motion characteristics. This behavior can be seen in all of the frames, especially those with longer periods and larger levels of inelastic behavior (Figure 12).

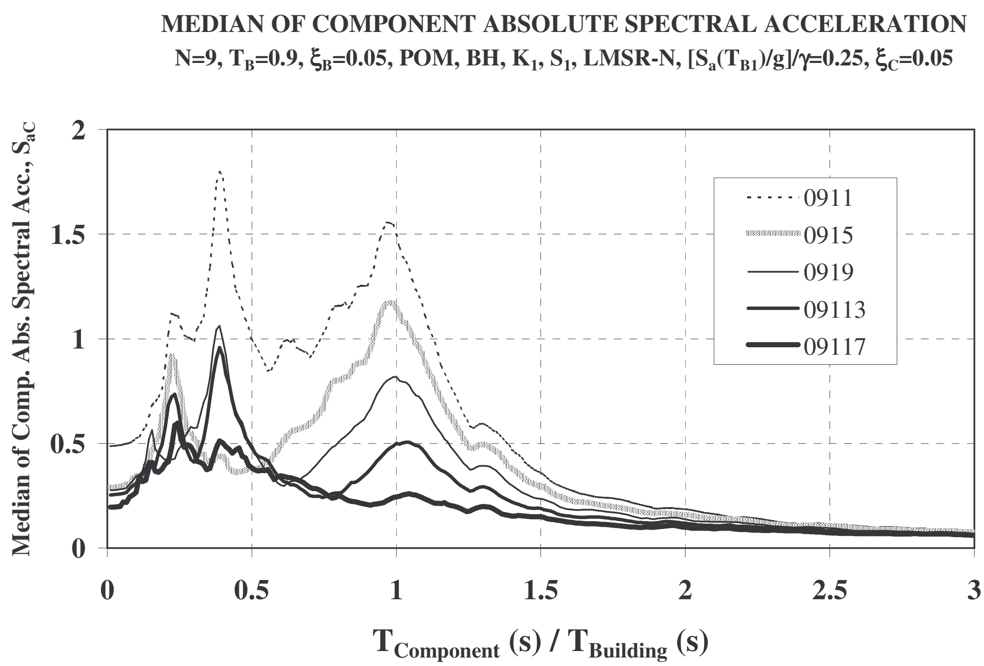

Figure 11 – Median of Component Absolute Spectral Acceleration for the 9-story Elastic Frame

Figure 12 represents the ratio of the spectral component acceleration at the fundamental period of the supporting structure, to the spectral component acceleration at the second mode period of the supporting structure. Values on this plot smaller than 1.0 indicate a larger FRS acceleration response when the NSC is in tune with the second mode period of the supporting structure. The relative contribution of the second mode to the response of NSCs becomes more critical as both the period and the level of inelastic behavior of the frame increase. Moreover, this effect is more pronounced as the height at which the NSC is located in the building decreases. The only instances where a decrease is not observed in Figure 12 are at the top 1/3rd of the 9 and 18-story structures. This is due to the second mode shape for those frames. The second mode shapes indicate that node 5 (floor 8) of the 9-story building and node 9 (floor 15) of the 18-story building have a relatively small second mode acceleration contribution to the overall response. This causes the ratio of spectral NSC accelerations for the first to the second mode of the structure to be very high. This can be seen in the following mode shape plots (Figure 13), and in the previous FRS plots (Figure 11), where the respective FRS (0915) skips the second mode peaks exhibited for the other floors of the 9-story frame.

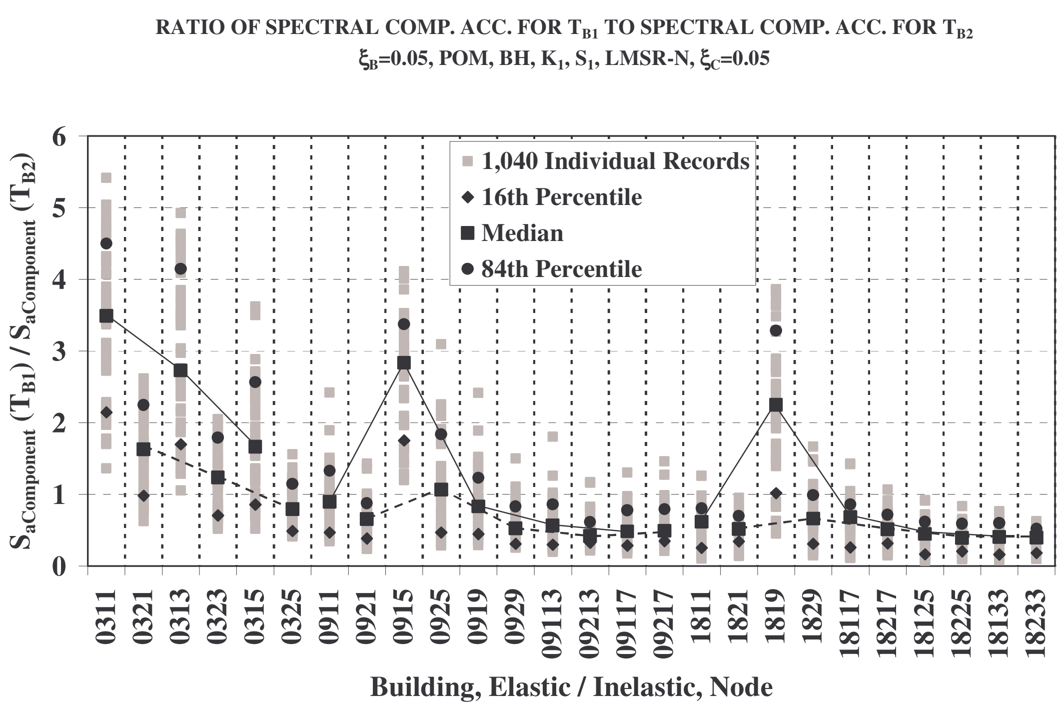

Figure 12 – Ratio of Absolute Spectral Component Acceleration for T_{B1} to the Absolute Spectral Component Acceleration for T_{B2}

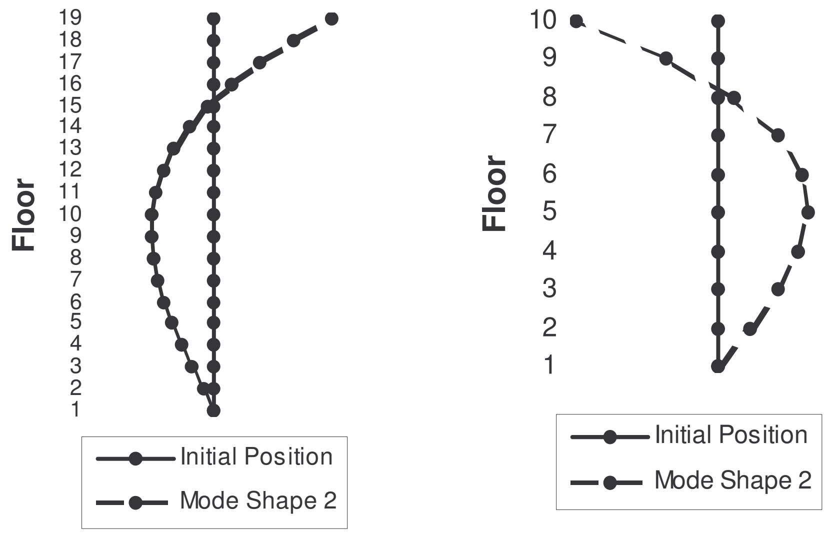

Figure 13 – Second Mode Shapes for the 9 and 18-story Frames

<u>Nonlinear Behavior of the Supporting Structure</u>

Figure 10 shows PFAs values greater than the PGA (or ground acceleration amplifications greater than 1.0) along most of the height of all elastic frames. The elastic frames also exhibit higher ground acceleration amplifications than the inelastic frames. Except for the stiff, short period 3-story structure, most of the inelastic frames have amplifications below 1.0 and maintain relatively constant floor accelerations over the height. Figure 10 shows that inelastic behavior of the supporting structure results in a more significant reduction in PFA for the upper 1/3rd of the building.

Figures 8 and 9 demonstrate that the median inelastic floor response spectrum (Figure 9) does not exhibit sharp acceleration peaks, as seen in the elastic median plots (Figure 8). With short period structures, such as the 3-story structure of this study, the floor acceleration response spectrum peaks are evident for both the elastic and inelastic cases. Another point to note in Figures 8 and 9 is the severe reduction in the acceleration response of nonstructural components due to the inelastic action of the supporting structure near the first mode period of the supporting structure. Figure 14 confirms this reduction near the first mode period of the supporting structure and indicates an increasing ratio of elastic spectral component acceleration to inelastic spectral component acceleration as the period ratio nears one. This demonstrates that the level of inelastic behavior in the frame primarily affects the response of the first mode only. The response near periods corresponding to the higher modes of the supporting structure is also reduced, as indicated also in Figure 14, but not as significantly as the response near the first mode period of the supporting structure.

Studies by Rodriguez, et al. (2002) and Lin and Mahin (1985) also indicate that the inelastic action for other types of supporting structures, e.g., structural walls, significantly reduces the acceleration near the first mode period of the supporting structure. The data from this study is consistent with the results of their previous studies.

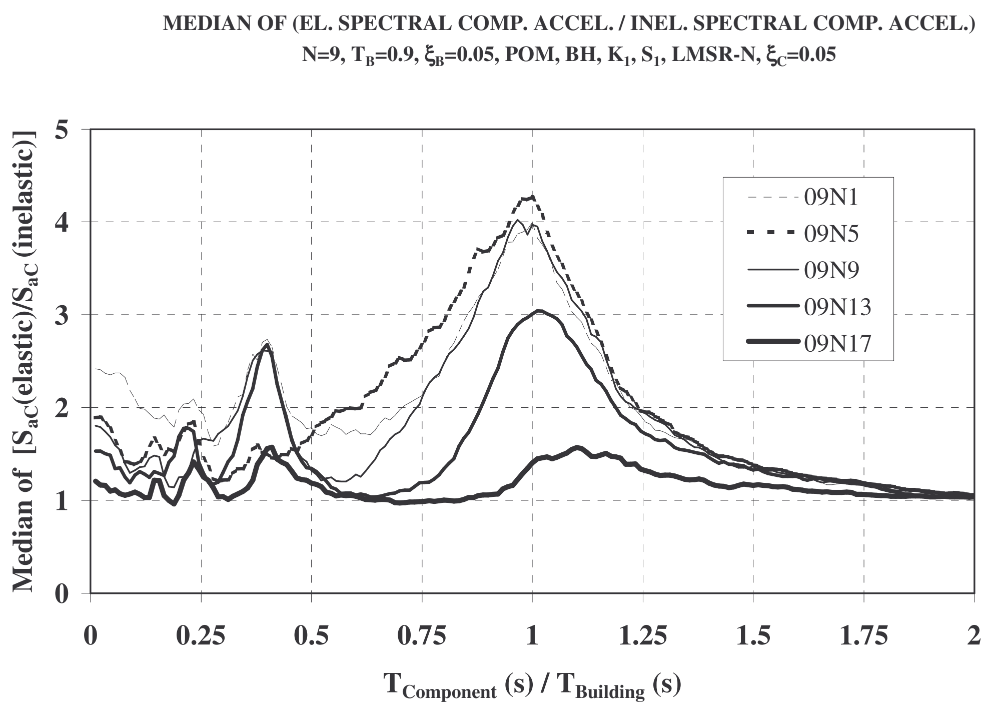

Figure 14 – Median of Ratio of Elastic Spectral Component Acceleration to Inelastic Spectral Component Acceleration (Inelastic Supporting Frame), 9-Story Frame

Figure 14, which depicts the de-amplification of peak floor accelerations for the NSCs due to ductility demands, shows that for period ratios greater than 2.0, the elastic and inelastic responses are roughly equal. This behavior is attributed to the fact that the NSC accelerations are small due to the flexible nature of the component as compared to the primary structure. These results indicate that maximum accelerations experienced by NSCs whose periods are greater than or equal to the fundamental period of the supporting structure are weakly sensitive to the level of inelastic behavior in the supporting structure. Figure 14 indicates no significant de-amplification effects in the inelastic spectra with respect to the elastic one for the lower floors. Figure 12 also shows that the variation along the height of the ratio of 1st mode to 2nd mode maximum acceleration response of NSCs attached to inelastic frames is more uniform than that of NSCs attached to elastic frames. This pattern becomes clearer with an increase in the fundamental period of the frame. Overall, Figure 12 indicates smaller ratios for inelastic structures, although the ratios for inelastic and elastic frames approach the same value as the period of the structure increases.

This study investigates only two relative intensity levels for each frame. To obtain a better understanding of the variation of NSC acceleration response do to the supporting structures' ductility demands, a range of relative intensities should be tested.

Effects of Frame Height, Number of Stories, and the Location of NSCs in the Supporting Structure

As shown earlier, damage to NSCs can be severe due to the amplification of the ground motion by the primary structure. The variation of PFA with height is strongly dependent on the dynamic characteristics of the building. For moment-resisting frames with longer periods, the normalized floor accelerations (PFA/PGA) decrease with building height. Figure 10 shows that larger normalized PFAs occur at higher floor levels, especially for shorter period structures dominated by the first mode (i.e. 3-story structure)

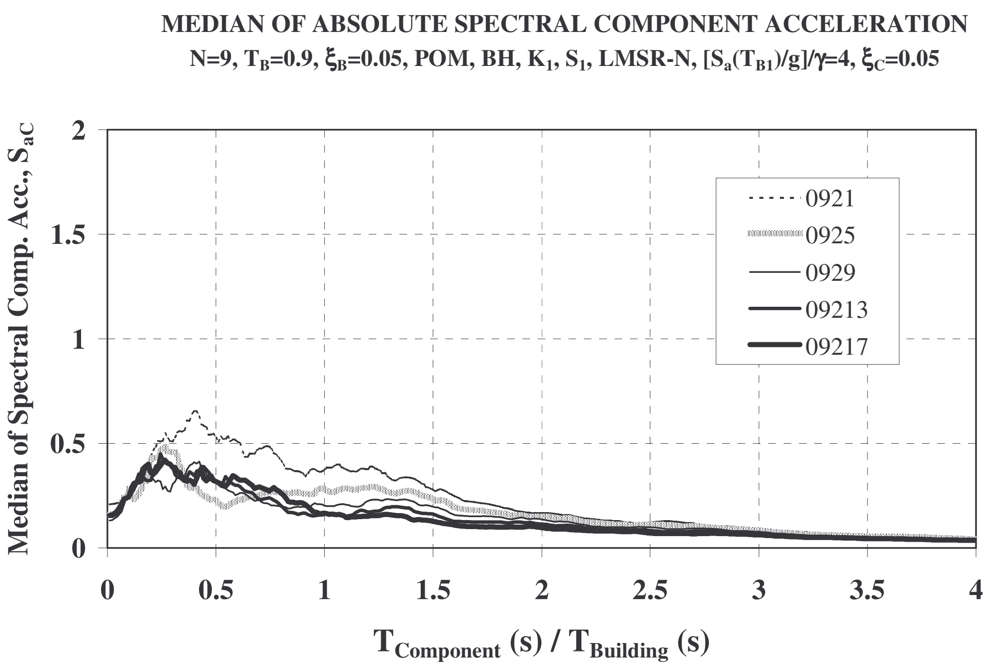

Figure 15 – Medians of the Absolute Spectral Component Accelerations for the 9-Story, Inelastic Frame

Figures 11 shows for the first and second modes of the supporting structure, that as the height in the structure increases for the elastic frames, so does the absolute spectral component acceleration. The 3-story and 18-story elastic frame plots follow the same pattern. The only two instances where this does not hold true are for node 5 (floor 8) of the 9-story building and node 9 (floor 15) of the 18-story building, where the relative contribution of the second mode acceleration to the overall acceleration response is small.

Comparing Figures 11 and 15 clearly shows the de-amplification in the spectral response of the NSC due to increased inelastic behavior of the supporting structure. The reduction near the first mode of the supporting structure is the most significant reduction. The spectral acceleration for first floor of the 9-story structure is approximately the same for the elastic (Figure 11) and inelastic (Figure 15) frames. As the height increases, there is a greater reduction in spectral acceleration response due to the inelastic frame action.

Assessment of Current Seismic Design Provisions for NSCs

Current U.S. building code requirements for the response of NSCs are based on the National Earthquake Hazard Reduction Program (NEHRP) seismic provisions. The most recent NEHRP requirements (*NEHRP*, 2000) for the design of NSCs subjected to seismic motions are based on the calculation of the peak floor acceleration. This PFA is then scaled according to a component amplification factor, and a response modification factor to obtain the NSC design force. The following NEHRP 2000 equations determine the force transferred to a component and/or its attachment to the supporting structure.

$$F_p := 0.3 \cdot S_{DS} \cdot I_p \cdot W_p \quad < \quad F_p := \frac{0.4 \cdot a_p \cdot S_{DS} \cdot W_p}{\dfrac{R_p}{I_p}} \cdot \left(1 + 2 \cdot \frac{z}{h}\right) \quad < \quad F_p := 1.6\, S_{DS} \cdot I_p \cdot W_p$$

(Equation 2)

The above equations take into account the component's weight (W_p), the PGA including site soil effects and seismic design category (0.4 x S_{DS}), the amplification of the PGA with the height of the component in the structure (z/h), the importance of the component (I_p), and the relative ductility expected in the component and its connections to the supporting structure (R_p). The component amplification factor (a_p) accounts for the dynamic amplification of the NSC response, especially near the point of resonance with the supporting structure.

Component amplification factor, a_p

The component amplification factor is equal to the maximum spectral NSC acceleration normalized by the PFA. Current seismic design provisions (e.g., *NEHRP*, 2000, *IBC*, 2003) assume a maximum component amplification factor of 2.5 around the period ratio (T_C/T_{B1}) of 1.0. Upon reviewing all of the maximum component amplification factors that resulted from the 1,040 analyses of this study, it is evident that the results of this study consistently exceed the maximum code provided a_p (see Figure 16).

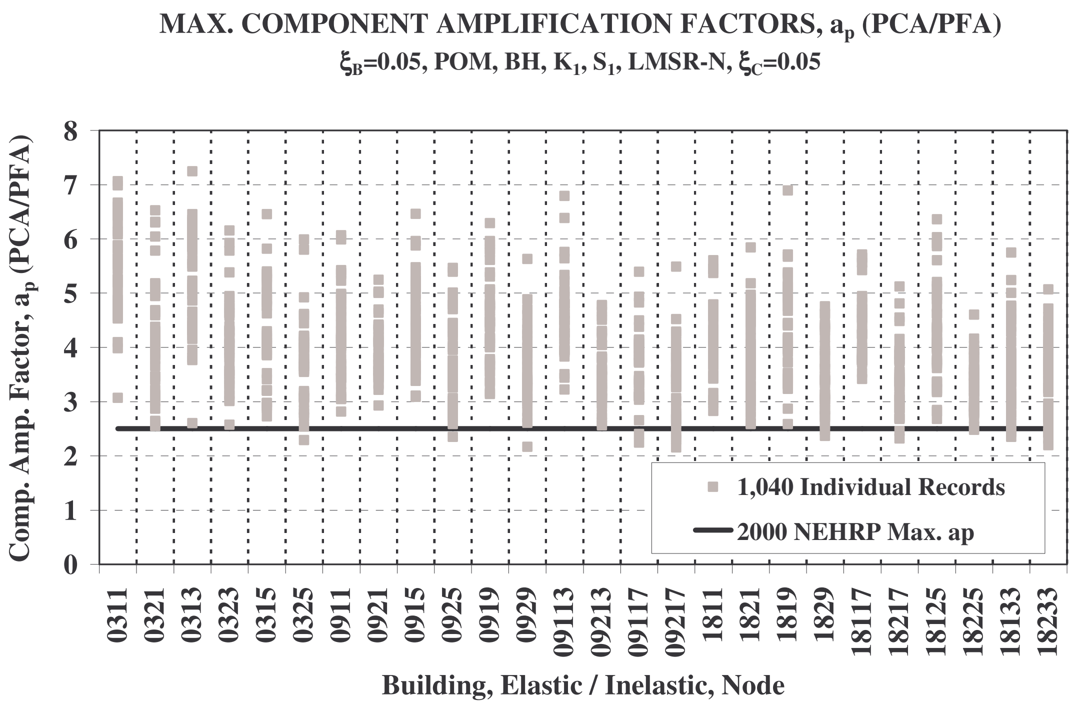

Figure 16 – Maximum Component Amplification Factors, a_p, for All Records

This study does not include the dynamic interaction between the NSC and the supporting structure. Some researchers suggest considering the interaction would reduce the NSC acceleration response, although it has been assumed in this study that this interaction is negligible since the focus is on NSC with masses that are small as compared to the total mass of the supporting structure. Moreover, 5% of critical damping is used in this study to generate the floor response spectra; therefore, NSCs with smaller damping values (as suggested by some researchers) would exhibit much larger component amplifications. Figure 16 typically shows higher component amplification factors for the elastic frames.

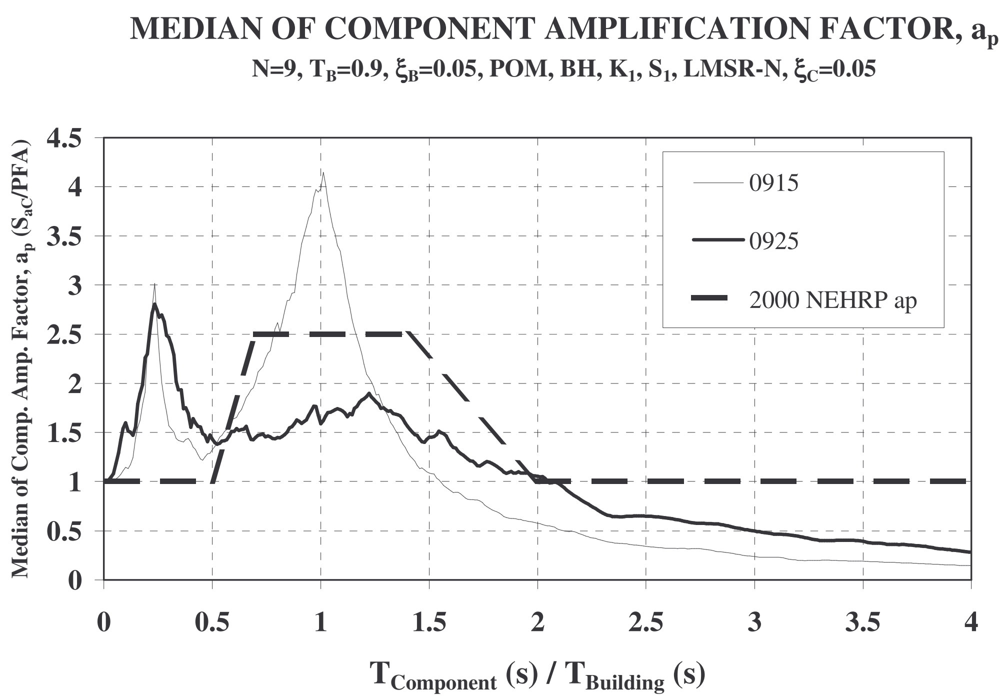

Figure 17 – Median of Component Amplification Factor, a_p, for the 8th floor of the Elastic (0915) and Inelastic (0925) 9-Story Frames

Figure 17 is a plot of the component amplification factor for the 8th floor of the 9-story frame used in this study. The plot shows the amplification factor for an elastic and inelastic frame. It is clear from this picture that the median component amplification factor is far above the maximum of 2.5 specified by *NEHRP* (2000). In the region of 0 to 0.5, NEHRP assumes a value of 1.0, and the results for both the elastic and inelastic frames exceed this value. This result in the low period ratio range is due to the NSC being in tune with a higher mode of the supporting structure. When the NSC is in tune with a higher mode of the supporting structure, this effect is almost always greater than 1.0, and often greater than 2.5. This plot represents the trend of the entire set of frames.

For some inelastic frames, as the stiffness degradation becomes large in the frame due to the inelastic action, the period elongation effect causes FRS values to exceed the code provided amplification factor of 1.0 for large period ratios. This condition is particularly important for the 3-story structure, for which the ductility demand is larger due to the frame's short period. Figure 17 shows that for period ratios greater than 1.5, the inelastic frame results in larger amplification as verified by the plots for the 3, 9, and 18-story structures.

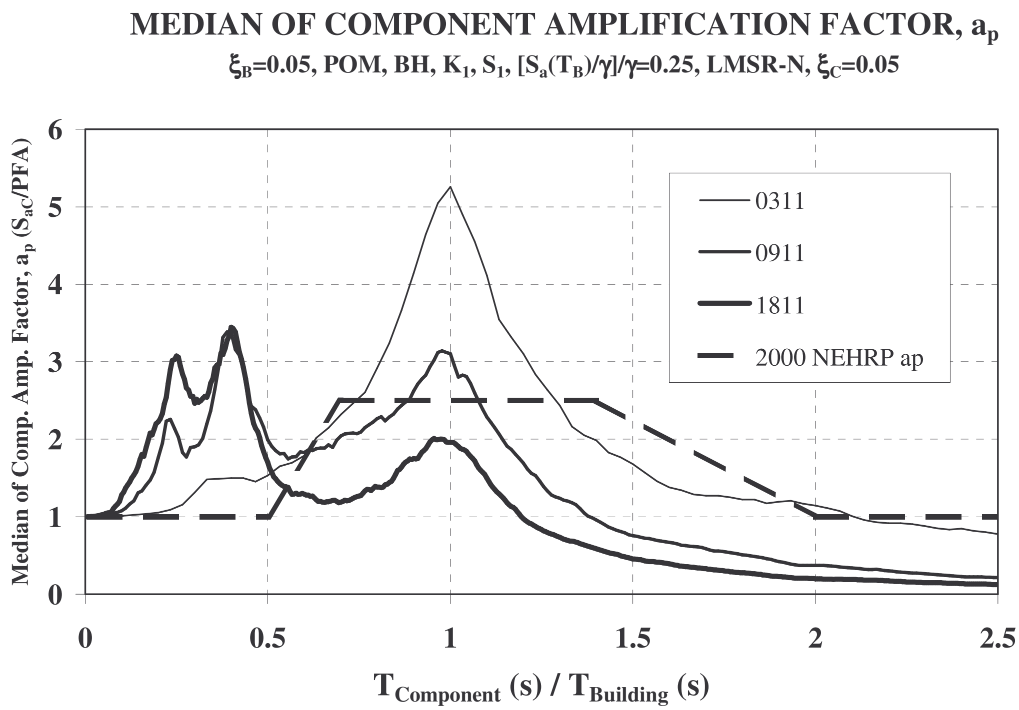

Figure 18 – Median of Component Amplification Factor, a_p, for the roof of the Elastic 3, 9, and 18-Story Frames

Figure 18, which depicts FRS for the roof level of the 3, 9 and 18-story frames, shows that taller, more flexible structures experience more severe component amplification factors for NSCs with periods close to the higher mode periods of the supporting structure. However, the opposite behavior is observed for the maximum acceleration response of NSC around a period ratio of 1.0. The behavior trends observed in Figure 18 are also shown in Figure 12.

The aforementioned results suggest that second mode effects are more critical for the design of acceleration sensitive NSCs for supporting structures with longer periods of vibration. This effect is not adequately represented in the current *NEHRP* (2000) provisions, which only recommends an amplification of the component acceleration near the fundamental period of the supporting structure. In the development of FRS, PFAs should be amplified to account for tuning of the NSC with the higher modes of the supporting structure, especially for taller, more flexible frames.

CHAPTER X: CONCLUSIONS

Earthquake ground motions can be severely amplified due to the dynamic characteristics of the supporting structures and nonstructural components supported on these structures. The amplified accelerations are a major threat to the survival of NSCs in the event of an earthquake. NSC damage has proven to be costly and dangerous; thus, there is a need to increase our understanding of the behavior of NSCs attached to buildings and develop transparent design methodologies to minimize and prevent damage to NSCs.

Floor response spectra are developed in this study to evaluate the maximum acceleration response of NSCs and provide significant insight into their dynamic behavior. Moreover, an assessment of the adequacy of current seismic design provisions for NSCs is carried out based on the results obtained in this work. The primary structures under consideration are stiff regular frame structures exposed to ordinary ground motions. The analysis controls variable system inputs such as the ground motion frequency content, the ground motion intensity level, the strength of the structure, the fundamental period of the structure, and the location of the NSC with respect to the height of the supporting structure. The most significant results obtained in this study are summarized as follows:

- The component amplification factor, a_p, is a function of the ratio T_C/T_{B1}, the inelastic behavior of the supporting structure, and the height of the component in the supporting structure.
- The acceleration response of a NSC is strongly dependent on how its period compares to the modal periods of the supporting structure. Therefore, the ratio of

the component period to the building period, T_C/T_{B1} is a critical parameter for the design of NSCs.

- An increase in the inelastic behavior of the supporting structure significantly reduces the component amplification factor, a_p, when the NSC period is near the fundamental period of the supporting structure. This reduction is not as significant for NSCs with periods that are tuned with the higher modes of the supporting structure.
- The absolute values of the FRS are strongly influenced by the location of the NSC along the height of the building. For elastic frames, especially short period frames, the higher the location of the NSC in the building, the larger the maximum accelerations it will experience. For inelastic frames, the higher the location of the NSC in the building, the smaller the maximum accelerations it will experience. This latter statement does not apply to the short period, $T_{B1} = 0.3$ s. frame for which the maximum NSC accelerations increase with height regardless of the level of inelastic behavior in the system.
- The effects of the higher modes of the frame on the acceleration response of NSCs with periods close to the higher mode periods of the frame is more critical for tall, flexible structures.
- In several cases, the component amplification factor, a_p, of current seismic design provisions severely underestimates the maximum acceleration response of NSCs, especially those with periods corresponding to $T_C/T_{B1} = 1.0$.
- Current seismic design provisions recommend a constant a_p for all floor levels. This study suggests that the component amplification factor should be a function

of the height of the building since the shape of the FRS is severely influenced by the location of the NSC along the height of the frame. This effect is more pronounced for NSCs attached to elastic frames.

- The effect that the higher modes of the supporting structure have on the maximum acceleration response of tuned NSCs is not considered in current seismic design provisions.

These observations and conclusions have to be interpreted within the limitations discussed in this paper. The results of this study are intended to support current efforts in performance-based earthquake engineering to create simple and transparent design methodologies for NSCs that correspond to various performance objectives. This work also provides much needed insight into the dynamic response of elastic NSCs supported on regular-frame structures.

Future research in this area should include the investigation of ground motions with different frequency content and longer duration. Moreover, frames with different stiffness distributions, fundamental periods (e.g. more flexible frames), structural systems, hysteresis models, and a range of relative intensity values need also be evaluated. Analyses should account for soil-structure interaction, multiple attachment locations, three-dimensional and torsional effects, and the influence of alternate damping mechanisms in real buildings. Finally, future efforts should also include the investigation of critical NSC design details, such as alternate damping values, the inelastic behavior of the NSC, and the overstrength characteristics of the NSC.

APPENDIX A

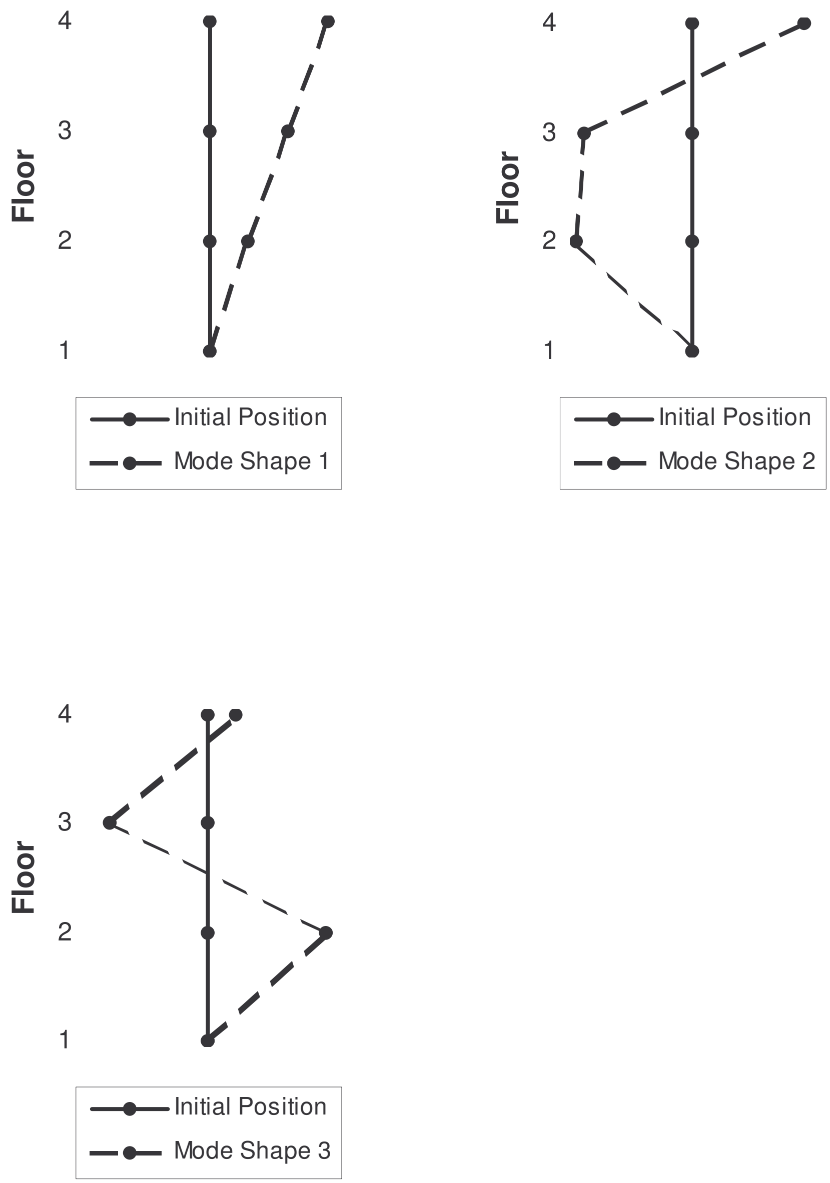

Figure A.1 – Mode Shapes for the 3-story Frame

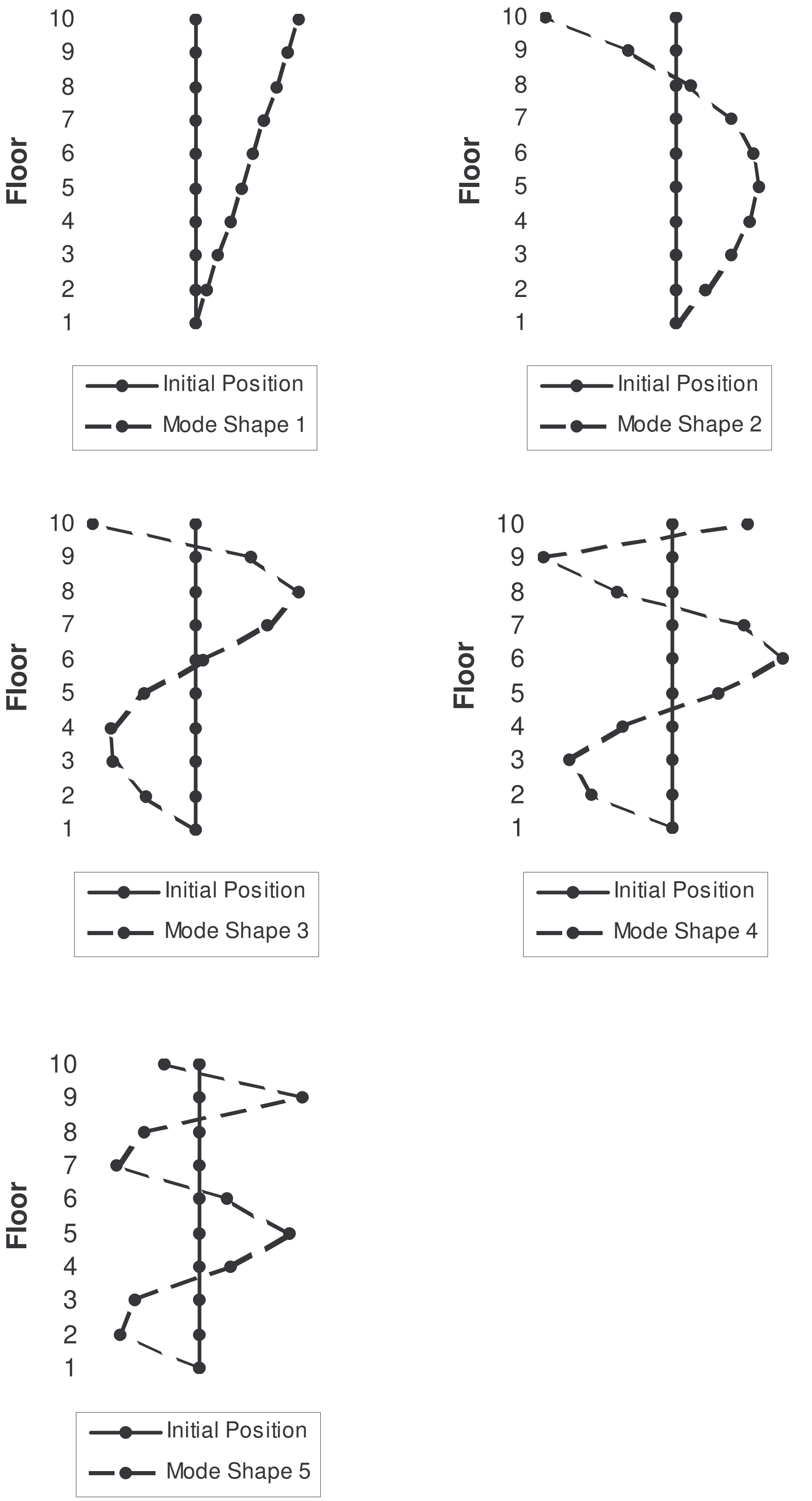

Figure A.2 – Mode Shapes for the 9-story Frame

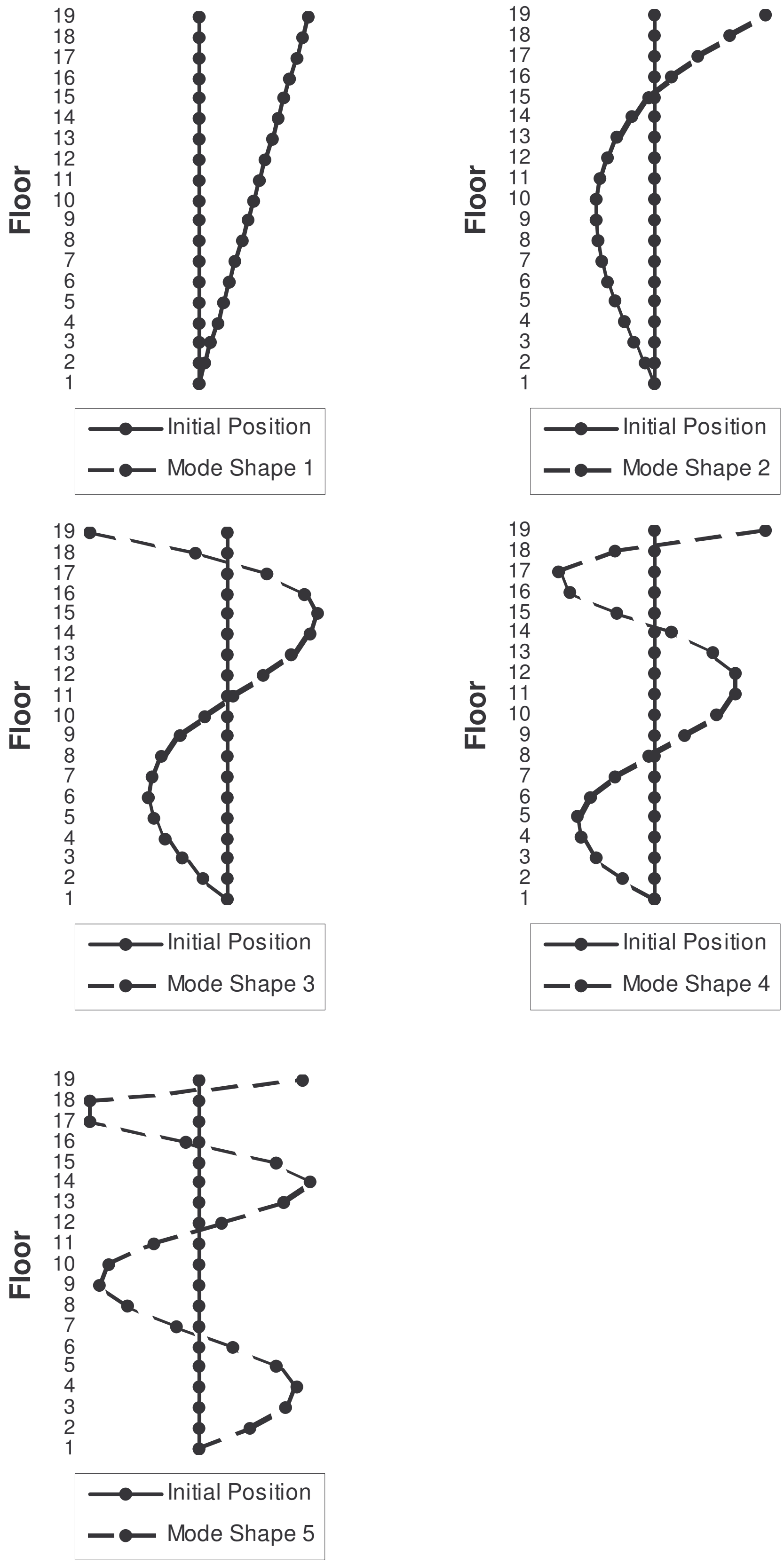

Figure A.3 – Mode Shapes for the 18-story Frame

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