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Improved Seismic Design of Non-structural Components (NSCs) and Development of Innovative Control Approaches to Enhance the Seismic Performance of Buildings and NSCs

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IMPROVED SEISMIC DESIGN OF NON-STRUCTURAL COMPONENTS (NSCs) AND DEVELOPMENT OF INNOVATIVE CONTROL APPROACHES TO ENHANCE THE SEISMIC PERFORMANCE OF BUILDINGS AND NSCs

BY

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DISSERTATION

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Improved Seismic Design of Non-structural Components (NSCs) and Development of Innovative Control Approaches to Enhance the Seismic Performance of Buildings and NSCs

By

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University of New Hampshire, September 2018

Abstract

Post-earthquake reconnaissance following past earthquakes in the US and other seismic-prone countries illustrates that the majority of building losses (injury, dollar loss and downtime) resulted from damage to nonstructural components (NSCs) and building contents. NSCs damages can severely compromise a building functionality, even if the building does not suffer significant structural damages. NSCs can be classified either as primarily displacement/deformation-sensitive or acceleration-sensitive. This study focuses on acceleration-sensitive components.

Previous studies on NSCs are mostly based on the responses of simplified models of primary systems and components. These models, while providing valuable insight into understanding the influential parameters on NSCs seismic demands and behavioral patterns, may not adequately represent the characteristics present in the response of actual buildings. In the first part of this dissertation, acceleration responses of a wide variety of instrumented buildings and code-based designed building models are evaluated to: (i) identify the most important limitations of using simplified numerical models, (ii) quantify the most influential parameters that control NSC responses, (iii) evaluate the design equivalent static equations of ASCE 7-16 for acceleration-sensitive NSCs, (iv) assess alternative design equivalent static equations proposed as part of a recent project sponsored by the Applied Technology Council (Project ATC-120), and (v) develop modifications and improvements to the proposed ATC-120 equations.

In the second part of this dissertation, modern seismic protection techniques are studied that can decrease seismic input demands to a building, as opposed to modifying the seismic resistance of a building, which is the approach taken in current US design seismic provisions. The conventional base-isolation and tuned-mass-damper concepts are utilized to develop an innovative seismic control system (i.e., partial mass isolation, PMI) that can reliably enhance the seismic performance of the structural elements and NSCs so that the building can be occupied and remain functional immediately after a design earthquake. The practicality, limitations, effectiveness, and robustness of the PMI system for protecting the structural and nonstructural components of building structures are discussed and evaluated.
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Chapter 1

Introduction

1.1 Summary

In part I of this dissertation, floor acceleration responses of a wide range of instrumented and code-based designed buildings are used to identify and quantify the most important parameters influential on horizontal seismic demands on acceleration-sensitive nonstructural components (NSCs). This discussion is followed by an evaluation of the current ASCE 7-16 equivalent static equation for designing NSCs. Recently proposed equations by ATC-120 Project are also evaluated and potential improvements to these equations are proposed. Part II of this dissertation studies passive control systems with a focus on a partial mass isolation (PMI) system. In the PMI system, different portions of story masses can be decoupled from the superstructure to perform as inherent dynamic vibration suppressors. The objective of this system is to protect a building and its content during severe earthquake excitations increasing the likelihood that the building can be immediately occupied after an earthquake. Part I and II of this dissertation merge in Chapter 10 wherein the PMI system is used to improve the seismic performance of NSCs. In addition to this merging chapter, it is useful to note that the two parts of this dissertation are similar from a global view of mechanics. In both parts, basically the responses of “system assemblies” are studied, and the following parameters/behaviors are addressed: (i) seismic system response; (ii) geometric linear systems; (iii) frequency tuning/detuning; (iv) damping of subsystems; (v) mass ratio of subsystems; (vi) strength of the subsystems; among others. Of course, depending on the specific topic studied in a chapter, some of the abovementioned parameters are more emphasized than others. Study presented in Part II, especially the tuning concept, can provide valuable insight into the behavior of NSCSs studied in Part I. The flowchart illustrated in Figure 1-1 summarizes the studies conducted in the two parts of this dissertation.
1.2 Introduction

Nonstructural components and systems (NSCs), often referred to as secondary systems, are those systems, parts, elements, or components that are not part of the primary structural load-resisting system but support the functionality of a building. In this context, the term NSC implies a system composed of the component, its support(s), and attachment(s). Examples of typical NSCs in building structures are partition walls, architectural facades, stairways, cladding systems, suspended ceilings, storage tanks, fire protection systems, cooling towers, generators, bookshelves, file cabinets, decorative items, and furniture. NSCs are generally classified, based on their seismic response sensitivity, into two broad categories: primarily deformation-sensitive components (e.g., partition walls) or primarily acceleration-sensitive components (e.g., suspended ceilings). The horizontal seismic responses of acceleration-sensitive components are the focus of the present study.
Post-earthquake reconnaissance following previous earthquakes in the US and other seismic-prone countries has consistently revealed that the majority of building losses (i.e., injury, death, dollar loss and downtime) are due to the direct or indirect consequences of damage to NSCs. These assessments have indicated that even if modern seismic design techniques may be able to successfully limit the damage to the main structural elements of a building during severe earthquakes, seismic damage to NSCs may be extensive, very costly and in some cases even life threatening (Sullivan et al. 2013). For example, an evaluation of various Veterans Administration hospitals following the 1971 San Fernando earthquake revealed that many facilities still structurally intact were no longer functional because of the loss of essential equipment and supplies (Whittaker and Soong 2003).

During the 1994 Northridge earthquake in Los Angeles, CA, several major hospitals were forced to curtail their operations and evacuate patients not because of structural damages but due to NSCs damages such as the failure of chilled water lines, water supply tanks and fire sprinklers; the failure of emergency power systems and heating, ventilation, and air-conditioning units; damage to suspended ceilings, light fixtures, elevators, and computer systems (Goltz 1994; Hall et al. 1994). In many past earthquakes, losses from damage to NSCs have exceeded losses from structural damages. For example, the cost related to NSCs represented more than 50% of the total damage cost of $18.5 billion stemming from the 1994 Northridge, CA earthquake (Kircher 2003). This is because NSCs account for most of the total monetary investment in typical buildings. Furthermore, damage to NSCs often occurs at seismic intensities significantly lower than those required to produce structural damages.

Examples of damage to NSCs during past earthquakes are shown in Figure 1-2.
Over the years, numerous studies have been prompted to improve the understanding of acceleration demands on NSCs. A wide variety of these research works can be found in the literature: from several studies that have focused on single-degree-of-freedom (SDOF) buildings (e.g., Lin and Mahin 1985; Chen and Soong 1989; Igusa 1990; Zhu et al. 1994; Sullivan et al. 2013) to works that have studied multistory three-dimensional building models (e.g., Wieser et al. 2013; Jiang et al. 2015). Numerical models used in these research works, while providing valuable insight into understanding the influential parameters on NSCs seismic demands, in many cases may not accurately represent important characteristics present in the response of real buildings. Some of these previous works are based on adopting the linear-elastic behavior assumption for
the supporting building. This assumption seems adequate for the design of essential facilities such as emergency centers and nuclear power plants, which are typically designed to remain elastic or nearly elastic during severe earthquake ground motions. However, this assumption is not directly applicable to most nonessential buildings that are designed to undergo inelastic deformations during the DE even when the presence of overstrength is accounted for. Many other research works, which have considered the supporting building inelasticity, have been mostly based on SDOF or generic building models that do not adequately represent relevant characteristics present in the response of actual multistory buildings. Few studies (e.g., Wieser et al. 2013; Petrone et al. 2016) that evaluated the ASCE 7 Fp equation using code-compliant buildings were based on a handful of buildings. For example, the study conducted by Wieser et al. 2013 used four special steel moment resisting frame buildings, designed based on ASCE 7 seismic provisions; in Petrone et al. 2016 five reinforced concrete frames, designed based on Eurocode 8, were utilized.

In addition to using inelastic generic building models or linear buildings, which may not adequately represent responses of actual buildings, many of the previously mentioned research works have been based on a linear NSC behavior and/or a 5% NSC viscous damping. To more reliably evaluate the accuracy of the current design equations, and propose improved equations if needed, code-based designed buildings with different characteristics (e.g., lateral load resisting system, strength, modal periods) and use NSC with different characteristics (i.e., tuning ratios, ductility demand and viscous damping ratio should be developed). Furthermore, results of these numerical building models should be verified using the responses of instrumented or tested buildings. These discussions are the basis for the study conducted in Part I of this dissertation.

Traditional seismic design methods (e.g., those presented in Part I of this dissertation) focus on designing buildings with the required strength to resist earthquake induced forces, whereas modern seismic protection techniques aim to reduce the input seismic forces transmitted to the building and NSCs (Constantinou et al. 1998). Conventional approaches are mostly based upon energy dissipation through structural damage to predetermined components, referred to as seismic fuses. Accepting seismic damage to the fuse elements, while protecting a structure from catastrophic failure, may be cumbersome or even impractical to repair without interrupting the building serviceability. To resolve the inherent deficiencies of conventional seismic design
methods, alternative design procedures involving seismic control systems have been proposed such as base isolation (BI) and tuned mass damper (TMD) systems.

The TMD and BI systems, as two of the well adopted modern seismic protection techniques, provide building designers with a means to adjust structural periods and damping to substantially mitigate the detrimental effects of earthquake ground motions. However, these techniques can suffer limitations that prevent their application, especially to high-rise buildings. For instance, the excessive superstructure flexibility and heavy loads experienced by high-rise buildings can prevent the effective implementation of BI systems, while the large auxiliary mass generally required in TMDs may present significant practical and architectural constraints. Part II of this dissertation utilizes BI and TMD systems’ principles to develop a partial mass isolation (PMI) system as an innovative seismic protection system that can effectively protect both a structure and its contents. This system can integrate benefits of these two effective systems while resolving their shortcomings. The proposed PMI approach, by isolating portions of masses at different stories, can provide a building with multiple inherent vibration suppressors partially resolving the abovementioned deficiencies associated with the application of conventional TMD and BI systems.

This dissertation consists of nine main chapters, each being a single article that has either appeared in a peer review-journal or has been submitted for publication or is in progress. In addition, a Conclusion chapter summarizes the most important findings from the dissertation and discusses further research directions.

1.3 Part I Improved seismic design of nonstructural components

This part, which includes Chapters 2 through 5, presents results of studies conducted on a variety of instrumented buildings and code-based designed buildings to identify and quantify the most influential parameters on the seismic response of NSCs. The results are used to evaluate the current ASCE 7-16 $F_p$ equivalent static equation for designing acceleration-sensitive NSCs. Recently through the ATC-120 Project on Seismic Analysis, Design and Installation of Nonstructural Components and Systems, an updated version of the ASCE 7-16 $F_p$ equation was proposed. The adequacy of this design equation is also evaluated, and potential improvements are proposed.


**Organization of Part I of this dissertation**

Chapter 2 uses the responses of a wide variety of instrumented buildings in the US to evaluate the observations made in previous studies that used simplified numerical building models when quantification of seismic demands on NSCs. This chapter also identifies and quantifies influential parameters on the NSCs seismic demands that are challenging to capture using simplified numerical models. These parameters include, but are not limited to, the in-plane diaphragm flexibility, the vertical mass or stiffness irregularity, torsional responses caused by the plan irregularities and/or asymmetric yielding of the lateral-load resisting elements, soil-structure-interaction effects, and the seismic base location.

The next step toward understanding the seismic responses of NSCs is to develop reliable numerical building models. In this study, the ground motion excitations used in the primary analyses are 20 spectrum-compatible (SC) records. A set of 44 magnitude-scaled historical ground motions are also used in some cases for the verification purposes. Because the main analyses in this dissertation are based on SC records, the challenges of using these records are addressed in Chapter 3. In the spectral matching approach, the frequency content of an input time series is manipulated in such a way that its elastic pseudo-acceleration responses in a pre-defined frequency range tightly match ordinates of a target (or a design) spectrum. The main reason for using the SC records is usually the lack of recorded ground motions that can adequately represent the seismic hazard for a specific site. In addition, using the SC records can mitigate the record-to-record variability present in responses of a set of actual ground motions. In the present study, another concern is stated regarding the use of recorded ground motions and amplitude-scaling them based on ACSE-7 provisions. It is shown that the widely used amplitude-scaling approach of ASCE 7-16 (i.e., multiplying the entire acceleration time history of records by a constant factor) can lead to a significant deviation of the mean scaled response spectrum from the target spectrum in the high-frequency region, especially for high-rise buildings.

Chapter 4 presents an evaluation of the current equivalent static equation of the ASCE 7-16 for designing acceleration-sensitive NSCs primarily using the responses of instrumented buildings. This chapter, as an initial step of the study presented in this dissertation, assumes a 5% viscous damping ratio and elastic behavior for NSCs. The evaluation of the responses of the instrumented buildings illustrate significant drawbacks associated with the ASCE 7-16 static equations for
designing acceleration-sensitive NSCs in their current format. The influence of ground motion intensity (or the supporting building inelasticity) on the seismic demands on NSCs, which has been extensively addressed using the numerical models in the past, is examined on the instrumented buildings. Evaluation of the results corroborate the significance of this issue. While using the responses of instrumented buildings can reveal the drawbacks of the current design equations, they cannot be solely used for updating the design equations for a variety of reasons mainly because many of these buildings responded in their elastic region, and more importantly many of them were built before the modern seismic provisions with significant irregularities. Hence, there is a need to develop code-based designed buildings for updating and improving the design equations for NSCs. In this chapter, representative steel moment resisting frame (SMRF) and reinforced concrete shear wall (RCSW) buildings are designed and simulated under the 20 SC records. The acceleration demands on NSCs are estimated when the buildings are exposed to SC records scaled to different intensity levels varying from 0.25 DE to 1.5 DE. An attempt has been made to identify and explain the similarities and differences between the results obtained from the archetype and instrumented buildings responses. The drawbacks of the ASCE 7-16 equations for estimating seismic demand on NSC are addressed.

Despite early studies on NSCs, particularly on equipment in nuclear power plants, (e.g., Kawakatsu et al. 1979; Viti et al. 1981; Igusa 1990), most recent works have assumed a linear-elastic behavior for NSC and/or supporting buildings when quantifying seismic demands on NSCs. For many NSCs, with proper details of attachment and supports, the component-support-attachment can tolerate inelastic deformations and dissipate the input seismic energy. In Chapter 5, the effect of NSCs inelastic behavior on their seismic demands is investigated. In addition, the effect of NSCs viscous damping on their seismic demands is quantified. Inelastic response spectra for different floor levels of several archetype buildings are developed assuming different NSC viscous damping ratios and target ductility values. This chapter evaluates seismic force demands on elastic and inelastic NSCs mounted on elastic and inelastic primary buildings. In other words, four different primary-secondary scenarios of elastic-NSC—elastic-building, elastic-NSC—inelastic-building, inelastic-NSC—elastic-building, and inelastic-NSC—inelastic-building are addressed. Results of numerical simulations illustrate that for NSCs tuned to the modal periods of a building (i.e., the most critical NSCs in a building), the NSC inelasticity can significantly reduce
force as well as displacement demands on NSCs. The beneficial effect of the NSC inelasticity is more pronounced in the case of low-damped NSC and an elastic primary building.

Recently in the ATC-120 Project, an attempt has been made to incorporate the most salient influential parameters on NSCs demand into NSCs design equations. The new parameters that have been incorporated are the effect of supporting building inelasticity and fundamental period, and NSC inelasticity. As part of Chapter 5, the proposed equivalent static equation by ATC-120 is evaluated. Additional potential modifications and improvements to this equation are proposed.

1.4 Part II Seismic control systems to improve the performance of building and NSCs

The second part of this dissertation includes Chapters 6 through 10. This section primarily presents the results of studying an innovative seismic control system that can improve the seismic performance of buildings and NSCs. In this system, which is denoted as multi-floor isolation system (MFI) in Chapter 9 and a partial mass isolation (PMI) is other chapters, different portions of story masses are isolated from the superstructure to act as inherent seismic energy suppressors. Similar versions of this system were proposed by some previous research works (e.g., Feng and Chai 1997; Ziyaeifar and Noguchi 1998; Villaverde 2002; Tsuneki et al. 2009; Sakr 2017).

Organization of Part II of this dissertation

As an initial step, an important issue in base-isolated buildings that can significantly influence the estimation of seismic demands on NSCs is addressed in Chapter 6. In BI buildings, most likely with the notion that modal mass participations of higher modes are insignificant, the importance of higher-mode dominated responses (e.g., floor spectral ordinates in short period NSCs) is less studied. This can be seen even in statements provided in some reference dynamics books. However, an evaluation of the responses of tested and instrumented BI buildings can readily reveal the presence of high frequency responses (e.g., see Dao and Ryan (2013) for a tested BI building, and Nagarajaiah and Xiaohong (2000); Nagarajaiah and Sun (2001) for instrumented BI buildings). Although the values of floor spectral ordinates in BI buildings are significantly lower than those in the corresponding non-isolated buildings, they are still large enough to be of concern in the design process for certain types of NSCs. In this chapter, through conducting parametric
studies, it is shown than the method of modeling the superstructure viscous damping, which is usually de-emphasized in modeling BI buildings, can substantially impact the BI responses, especially higher-mode dominated responses.

In Chapter 7, the most important drawbacks of using conventional TMD and BI systems are addressed. It is stated how the proposed PMI approach can be used to partially resolve these drawbacks. The seismic performance of the PMI configurations with two extreme isolated mass ratio (IMRs) of 5% and 90% are compared with that of an equivalent TMD and an ideal BI system, respectively. This comparison, which aims to illustrate the competence of this innovative system with respect to the conventional control systems, is conducted assuming different narrow-band and broad-band base excitations.

In Chapter 8, PMI configurations with many discretized IMRs ranging from 5% to 90% with increments of 2.5% are studied. PMI systems with identical and dissimilar isolated components (ICs) characteristics along the height are optimized using parametric study and Genetic Algorithm methods to minimize various objective functions (structural responses) under stochastic excitations. Additionally, partial PMI systems with ICs only at a subset of upper stories are studied and optimized. The practicality and limitations of the PMI system are discussed.

The structural seismic design process is associated with inherent uncertainties in the estimation of demand and capacity. These uncertainties are present due to differences between the assumed parameters in the design procedure, for the ground excitation and a structure, and the actual parameters, or due to the fluctuation of a structure’s parameters during its lifetime (i.e., aging). A system that exhibits a performance that is stable, i.e., not significantly affected by the mentioned uncertainties, is known as a robust system in the literature. In Chapter 9, the robustness feature of the PMI system is discussed. It is shown that this system can result in first-mode dominated seismic responses that are less sensitive to variations in the characteristics of the ground motion excitation and primary structure (i.e., aleatory variabilities and epistemic uncertainties, respectively).

Chapter 10 primarily discusses the application of the PMI technique for protecting NSCs. Firstly, the effectiveness and robustness of the convectional TMD system for mitigating high-frequency components of floor acceleration responses, which affect most typical NSCs. Simulation
results illustrate that a single mass damper tuned to a higher mode of a building can protect NSCs in the vicinity of that specific modal frequency. However, the operating range of this TMD is relatively narrow meaning that this system is less effective for NSCs beyond the tuning frequency. A PMI system is designed that can robustly protect high-frequency NSCs. In other words, the performance of this system is less sensitive to the change in the NSC frequency ratio (tuning ratio). This feature is of significance given the uncertainness present in estimating NSCs and supporting buildings dominant frequencies.

1.5 References


Chapter 2

Lessons Learned from Evaluating the Responses of Instrumented Buildings in the US: the Effect of Supporting Building Characteristics on Floor Response Spectra
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Chapter 2
Lessons Learned from Evaluating the Responses of Instrumented Buildings in the US: The Effect of Supporting Building Characteristics on Floor Response Spectra


2.1 Abstract

This chapter evaluates floor response spectra of a large number of instrumented buildings to identify and quantify influential parameters on nonstructural components (NSCs) seismic demands that are not taken into account in the ASCE 7-16 design equations and are also challenging to capture through numerical models. This evaluation reveals significant torsional responses, even for nominally regular buildings, that increase seismic demands on NSCs located at a floor periphery. It is illustrated that the in-plane diaphragm flexibility can amplify demands on NSCs located at the middle of an unsupported floor. Results show that these three-dimensional effects do not occur simultaneously. In other words, diaphragm flexibility can mitigate torsional responses at the expense of the mentioned adverse effect. An evaluation of floor acceleration responses at grade and foundation levels of buildings with basements reveals that in many cases, even with the presence of perimeter concrete basement walls, criteria to establish the seismic base at the grade level are not satisfied. This can significantly affect the estimated demands on NSCs based on the ASCE 7-16 design equations, especially for short-rise buildings.

Keywords: Nonstructural Components; Floor response spectra; Instrumented buildings; Strong motion sensors; In-plane diaphragm flexibility; Torsional responses; Seismic base location.
2.2 Introduction

Nonstructural components and systems (NSCs), often referred to as secondary systems, are those systems, parts, elements, or components that are not part of the primary structural load-resisting system but support the functionality of a building. Examples of typical NSCs in building structures are partition walls, facades, stairways, cladding systems, suspended ceilings, storage tanks, fire protection systems, cooling towers, generators, bookshelves, file cabinets, decorative items, and furniture. NSCs are generally classified, based on their seismic response sensitivity, into two broad categories. They are classified as either deformation-sensitive components (e.g., partition walls) or acceleration-sensitive components (e.g., suspended ceilings) (FEMA 356, 2000) with the latter classification being the focus of the present study.

Post-earthquake reconnaissance following previous earthquakes in the US has revealed that the majority of losses (i.e., injury, death, dollar loss and downtime) in building structures are due to the direct or indirect consequences of damage to NSCs. These assessments have indicated that even if modern seismic design techniques may be able to successfully limit the damage to the main structural elements of a building during severe earthquakes, seismic damage to NSCs may be extensive, very costly and in some cases even life threatening (Sullivan et al. 2013). For example, an evaluation of various Veterans Administration hospitals following the 1971 San Fernando earthquake revealed that many facilities still structurally intact were no longer functional because of the loss of essential equipment and supplies (Whittaker and Soong 2003). During the 1994 Northridge earthquake in the Los Angeles, CA, several major hospitals were forced to curtail their operations and evacuate patients not because of structural damages but due to NSCs damages such as the failure of chilled water lines, water supply tanks and fire sprinklers; the failure of emergency power systems and heating, ventilation, and air-conditioning units; damage to suspended ceilings, light fixtures, elevators, and computer systems (Goltz 1994; Hall et al. 1994). In many past earthquakes losses from damage to NSCs have exceeded losses from structural damages. For example, the cost related to NSCs represented more than 50% of the total damage cost of $18.5 billion stemming from the 1994 Northridge, CA earthquake (Kircher 2003). This is because NSCs account for most of the total monetary investment in typical buildings. Furthermore, damage to NSCs often occurs at seismic intensities significantly lower than those required to produce structural damages.
The preceding observations have highlighted the need to develop an adequate and practical methodology for determining the magnitude of seismic demands on NSCs. ASCE 7-16 basically provides four different methods for the estimation of seismic demands on NSCs (see section 13.3.1 of ASCE 7-16). Three of these methods require conducting a dynamic response analysis on the supporting building, and the remaining approach is based on an equivalent static approach. In accordance with the ASCE 7-16 Eq. 13.3-1, the horizontal seismic design force ($F_p$) applied at the NSC’s center of gravity, can be calculated based on a simplified equivalent static force given by Eq. 2.1

$$F_p = \frac{0.40S_{DS}W_p}{R_p/I_p}a_p[1+2(z/h)] \quad (2.1)$$

where $S_{DS}$ is the short-period pseudo-spectral acceleration response for the building site; $W_p, I_p$ and $R_p$ are the NSC operating weight, importance factor and response modification factor, respectively; $a_p$ is the NSC amplification factor that depends on the NSC period; $z$ is the height of the structure at the point of attachment of NSC with respect to the seismic base; $h$ is the average roof height of the structure with respect to the base. $F_p$ shall not be greater than $1.60S_{DS}W_p/I_p$ and shall not be less than $0.30S_{DS}W_p/I_p$. In Eq. 2.1, the term $0.40S_{DS}$ essentially represents the peak ground acceleration (PGA) at the base of the supporting building at the design earthquake (DE) level, denoted as the design PGA in this paper. The peak component acceleration (PCA) response normalized to the design PGA, can be written in the form of Eq. 2.2

$$\frac{PCA}{PGA} = \frac{F_p/W_p}{0.40S_{DS}} = a_p[1+2(z/h)]/(R_p/I_p) \quad (2.2)$$

In Eq. 2.1, the term $[1+2(z/h)]$ is essentially the ratio between the peak floor acceleration (PFA) and the design PGA, and the parameter $a_p$ is the ratio between the PCA and the PFA for an elastic NSC with an importance factor of unity. The lower and upper design limits for PCA/PGA are obtained to be 0.75 and 4.0, respectively. According to Eq. 2.1, the NSC design acceleration is to some degree a function of NSC characteristics (i.e., period, ductility demand, and vertical location within the structure) as well as the magnitude of the design PGA at the supporting building site. However, the effects of the supporting building characteristics such as modal periods, ductility demand (or the level of inelastic behavior) and lateral-load resisting system are not explicitly taken...
into account in the design equation. Section 13.3.1.4 of ASCE 7-16 provides alternative dynamic analysis approaches in lieu of Eq. 2.1. Two of these methods require that maximum floor acceleration responses be obtained from the dynamic analysis of the supporting building. In other dynamic methods floor response spectra can be used to determine NSC acceleration demands. Since the vast majority of seismic design efforts in practice are based on the equivalent static method, the dynamic analysis approaches are not discussed herein. From here on the term ASCE 7 $F_p$ equation in this paper refers to the equivalent static approach of ASCE 7-16.

Many previous research studies (Villaverde 1997a; Medina et al. 2006; Sullivan et al. 2013; Wieser et al. 2013; Wang et al. 2014; Anajafi and Medina 2018a) have illustrated potential shortcomings associated with the design equations and methodologies provided in the US design codes and standards, particularly the ASCE 7-16 $F_p$ equation, for the estimation of seismic demand forces on NSCs. For example, several previous numerical studies have highlighted the need to incorporate the effect of the modal periods, lateral-load resisting system and the level of inelastic behavior of the supporting building into the $F_p$ equation (Medina et al. 2006; Anajafi and Medina 2018a). These research works, although have provided valuable insight into understanding the most influential parameters on the estimation of NSCs seismic demands, have been mostly based on simplified two-dimensional (2D) numerical models that in many aspects may not adequately represent the characteristics present in the responses of actual buildings. These relevant characteristics include, but are not limited to: the in-plane flexibility of the floor diaphragm; torsional responses of the supporting building; vertical mass and stiffness irregularities; the real distribution of seismic damage; the contribution of infill and interior partition walls to the building lateral stiffness and strength; soil-foundation-structure interaction; the equivalent viscous damping of the supporting structure; as well as potential interactions between heavy NSCs and the supporting building; among others. Based on the authors’ experience working with engineering practitioners for several years, significant skepticism has always been present when interpreting research results that have used these simplified numerical models to the point that, in many instances, the reliability of these results has been strongly questioned.

This study presents the results of a comprehensive evaluation conducted on the floor response spectra of a variety of instrumented buildings (i.e., buildings whose responses have been recorded during past earthquakes) in the US. The primary objective of this evaluation is to identify and
quantify the most salient parameters than can significantly influence the magnitude of NSCs acceleration demands but are not explicitly considered in the simplified ASCE 7-16 $F_p$ equation 13.3-1 and are also challenging to capture using numerical models. A second objective is to evaluate and validate observations from numerical studies included in the literature that illustrate the need to incorporate the supporting building modal periods, level of inelastic behavior, and lateral-load resisting system into the ASCE 7-16 $F_p$ equation.

2.3 Instrumented buildings selected for this study

All steel and concrete multistory buildings that have recorded ground motions with a PGA$_{\text{Max}}$ (the maximum PGA in the two orthogonal horizontal directions) larger than 0.15 g are selected from the Center for Engineering Strong Motion Data website (CESMD www.strongmotioncenter.org). The lower limit applied on PGA is believed to represent a reasonable threshold of seismic damages and can render a large enough sample of strong ground motions. Because the ASCE 7 load standard is meant to provide design equations for common buildings in the US, in the selection of the multistory instrumented structures, masonry building configurations and buildings equipped with seismic control devices (e.g., viscous damper and base isolation) are excluded. This strategy results in 35 individual multistory instrumented buildings. For single-story structures, plywood structural systems, which are common in the US, are also considered. Furthermore, in the entire CESMD database only three individual single-story buildings meet the criterion PGA $\geq 0.15$ g. Therefore, in this case, a PGA threshold of 0.10 g is adopted to increase the number of samples to nine.

Adopting the abovementioned criteria provides a total of 44 individual instrumented buildings. Given that some of these buildings have recordings available from more than one earthquake event, 59 building-earthquake cases are identified overall. Since the characteristics of the individual buildings, as well as the recorded ground motions in the two orthogonal principal directions, are distinct, the compiled database has an overall size of 118 (i.e., 59 $\times$ 2) building-directions. All floor motions available in the CESMD for the 118 considered building-directions (approximately 600 individual floor motions) are used for the evaluations conducted in this study. The compiled database encompasses a wide range of structural characteristics, in terms of height, modal periods and lateral-load resisting system, and presents ground motions with different intensity levels and frequency contents. An evaluation of the characteristics of the selected instrumented buildings
demonstrates that 16 sample buildings (i.e., 32 individual principal directions) are single-story structures and 43 samples (i.e., 86 individual principal directions) are taller than a single-story. Table 2-1 presents the lateral-load resisting system type for different single-story and multistory buildings. As seen, the lateral-load resisting system in most multistory building-directions (i.e., 46 out of 86) is a moment resisting frame (MRF) system. Further evaluation of the selected buildings indicates that 75% of these buildings were designed before 1975 (pre-modern code design) and the rest were designed after 1975 (modern design). At least 12 individual buildings exhibit moderate to high degrees of irregularity (e.g., asymmetric geometry, asymmetric lateral-load resisting elements in plan, irregularity in floor mass or story stiffness, etc.). Ground motion records from 21 different earthquake events are present in the database. The 1994 Northridge and the 1989 Loma Prieta earthquakes with 21 and nine record pairs, respectively, are the largest represented events in the prepared database. The PGA of the recorded ground motions varies between 0.04 and 0.80 g with an average of 0.20 g. A detailed description of the characteristics of the selected instrumented buildings and the recorded ground motions is presented in the Appendix.

Table 2-1  Lateral-load resisting systems for the instrumented building-directions

<table>
<thead>
<tr>
<th>Single-story buildings</th>
<th>Multistory buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral-load resisting system</td>
<td>Sample size</td>
</tr>
<tr>
<td>Shear wall (SW) (^a) with plywood diaphragm</td>
<td>12</td>
</tr>
<tr>
<td>SW with plywood sheathing over steel joists</td>
<td>8</td>
</tr>
<tr>
<td>SW with metal/concrete diaphragm</td>
<td>6</td>
</tr>
<tr>
<td>Steel bracing with concrete diaphragm</td>
<td>4</td>
</tr>
<tr>
<td>Wood frame with composite wood diaphragm</td>
<td>2</td>
</tr>
</tbody>
</table>

\(a\) SW at upper stories and MRF at the ground level.

Figure 2-1a illustrates the distribution of the selected instrumented building-directions in terms of the number of above-grade stories. As seen, most of the samples (i.e., 88 out of 118) are 10 stories tall or shorter. The tallest building in the collected database is a 52-story steel structure. Figure 2-1b depicts the estimated fundamental periods (\(T_{1\text{bldg.}}\)) for all building-directions considered in this study (see the Appendix for the \(T_{1\text{bldg.}}\) of the individual cases). The ASCE 7-16 equations for computing the approximate fundamental period for steel MRF (i.e., \(T_{1\text{bldg.}} = 0.028 h^{0.8}\)) and concrete SW (i.e., \(T_{1\text{bldg.}} = 0.02 h^{0.75}\)) systems, which are representative of flexible and stiff systems, are also illustrated in this figure (note that a structural height of
where \( n \) is the number of stories, is assumed). It is worthwhile to note that the current ASCE 7-16 formulas for \( T_{\text{bldg.}} \) are based on lower-limit estimates from a study conducted by Goel and Chopra (1997) on instrumented buildings. It is shown in Figure 2-1a that the observed \( T_{\text{bldg.}} \) for most MRF systems (i.e., 89% of the MRF data points) lies above the ASCE 7-16 estimate, but a few SW configurations (i.e., 34% of the SW data points) \( T_{\text{bldg.}} \) are below the ASCE 7-16 estimate. This latter observation implies that in some cases real SW buildings are stiffer than the ASCE 7-16 estimate.

![Graph](image1)

**Figure 2-1** Characteristics of the selected instrumented building-directions

Figure 2-2a shows the distribution of the recoded motions at the base of instrumented buildings in terms of PGA. As seen, most of the recoded ground motions are in the range of low-to-medium intensity (e.g., 83% of the data points fall in the range 0.10-0.35 g).

![Graph](image2)

**Figure 2-2** (a) Distribution of recoded motions at the base of instrumented buildings in terms of PGA; (b) Probability of exceeding a given normalized ground motion intensity measure
Many previous numerical studies have illustrated that the ground motion intensity, in other words, the level of inelastic behavior in the supporting building can significantly influence the magnitude of the floor spectral acceleration responses. Therefore, any evaluation of the instrumented buildings should account for this important characteristic. In this paper, a simplified approach is adopted to approximately estimate whether an instrumented building behaved in the elastic or inelastic behavior ranges. Let $[S_a(T_{1bldg})]_{GM}$ be the elastic 0.05-damped pseudo-spectral acceleration response of the recorded ground motion at the base on a given instrumented building-direction at a period equal to $T_{1bldg}$. Let $[S_a(T_{1bldg})]_{Design}$ be the elastic pseudo spectral acceleration ordinate predicted by the 0.05-damped design spectrum for the building site at $T_{1bldg}$ (calculated based on USGS tool). A dimensionless parameter, namely normalized ground motion intensity measure (IM), defined as the ratio $[S_a(T_{1bldg})]_{GM}/ [S_a(T_{1bldg})]_{Design}$ is introduced to roughly predict the level of experienced inelastic behavior in the supporting building-direction of interest. For a code-based designed building exposed to a ground motion, if this parameter is larger than $1/R_\mu$, where $R_\mu$ is the portion of the Response Modification Factor related to the displacement ductility, inelastic actions are expected. For the typical MRF and SW systems, $R_\mu$ ranges between 2.0 and 3.0. Assuming an average value of 2.5, one can expect that if the normalized IM for an instrumented building-direction is larger than 0.4, it has most likely experienced inelastic actions. Figure 2-2b illustrates the percentage of the building directions for which the normalized IM exceeds a given value. As seen, less than 25% of the studied instrumented building-directions exceeded the IM threshold of 0.4, implying that many of these structures most likely responded elastically. This IM will be used for interpreting the results of the conducted evaluations on the floor response spectra.

### 2.4 Main assumptions for generating floor response spectrum (FRS)

The floor response spectrum (FRS) method is generally based on an uncoupled analysis of the supporting building and NSCs meaning that the component-building dynamic interaction is neglected. In this case, the NSC is treated as a single-degree-of-freedom (SDOF) system in a separate model excited by floor accelerations obtained from the dynamic analysis of the supporting structure. The FRS method is sufficiently accurate for NSCs whose masses are much smaller than
the masses of the supporting building (e.g., by a factor of at least 1000 as mentioned in Toro et al. 1989; Adam et al. 2013) and whose fundamental periods are not too close to the modal periods of the building. This approach can result in conservative demands on NSCs that do not meet the mentioned mass and period characteristics because it does not account for the fact that the response of NSCs may modify the response of the supporting building and vice versa (Villaverde 1997b). In the case of instrumented buildings, the dynamic interaction between a supporting building and its specific NSCs is built-in in the characteristics of the recorded floor motions. Although it is recognized that the period range considered in a typical FRS may differ from the periods of these specific NSCs, the FRS results obtained from the instrumented buildings can still be considered a better representation of acceleration demands on NSCs than the results obtained from simulations using uncoupled numerical models.

Consistent with many previous studies that have dealt with the quantification of seismic demands on NSCs, the floor spectra in this study are developed based on the rule of thumb approximation of 0.05 viscous damping ratio and an elastic NSC behavior. Note that the objective here is not to evaluate the effect of NSCs damping ratio or inelastic behavior on their seismic force demands but is to evaluate the supporting building parameters that are influential on NSCs seismic demands given a component damping ratio and elastic behavior. The 0.05-damped elastic FRS is developed for all existing floor motions for a NSC period range varying from 0 to $2.0T_{\text{bldg}}$. The maximum pseudo-spectral acceleration over the entire FRS is defined as the PCA response. The spectral acceleration value at $T_{\text{comp.}} = 0$ corresponds to the PFA response.

### 2.5 3D effects: diaphragm flexibility and torsional responses

Seismic loads for designing NSCs are specified in the code with no amplification factor to account for the in-plane dynamic behavior of the floor diaphragm or horizontal torsional responses of the supporting building. On the other hand, for developing floor spectra, input acceleration floor motions are generally extracted from a 2D numerical model of the supporting building based on the premise that the floor diaphragm is rigid in its own plane, and torsional effects can be neglected. This section intends to evaluate the influence of these 3D behaviors of FRS results of instrumented buildings.
2.5.1 In-plane diaphragm flexibility

In the seismic analysis of building structures, floor systems are generally assumed to serve as rigid diaphragms between the vertical elements of lateral-load resisting systems. However, floor deformability is a function of the geometry, the structural details of the floor system, and also the stiffness of the lateral-load resisting system of the supporting building (Bernal et al. 2014). In many cases the conditions for assuming a rigid diaphragm are not satisfied. For example, in long-floor span buildings with perimeter lateral-load resisting elements (e.g., end shear walls), or in floor systems with relatively large openings than can contribute to in-plane deformation, diaphragms can behave quite flexibly. A flexible floor diaphragm behaves as a deep beam spanning between two elements of the lateral-resisting systems (e.g., two shear walls). The fundamental period of vibration of structures with flexible diaphragms is generally longer than that of equivalent buildings with rigid floor diaphragms (Tremblay et al. 2000). In-plane diaphragm flexibility can produce unexpected seismic demands including large structural drifts, diaphragm deformation, diaphragm forces, as well as excessive gravity system drifts, and shear forces (Iverson and Hawkins 1994; Fleischman et al. 1998; Fleischman and Farrow 2001; Sadashiva et al. 2012). The lateral-load distributions in a structure with a flexible diaphragm can significantly depart from the distribution assumed as part of the equivalent static lateral force method (Costley and Abrams 1996; Tremblay et al. 2000; Lee et al. 2007).

2.5.2 Torsional responses

Seismic-induced torsional responses in building structures occur due to (i) the asymmetric arrangement of the stiffness and strength of the lateral-load resisting elements and/or the asymmetric distribution of floor masses; (ii) torsional components of earthquake ground motion; (iii) eccentricities between the centers of rigidity and mass that exist because of uncertainties in quantifying the mass and stiffness distribution of a structure. The cause of torsion listed under (i) should be explicitly addressed in the numerical models. To account for sources of torsion listed under (ii) and (iii), ASCE 7-16 requires the consideration of an accidental torsional moment for both symmetric and asymmetric buildings. Several previous research works have studied the effects of seismic-induced torsion on structural responses, mainly displacement responses, of instrumented buildings (e.g., Çelebi et al. 1991; Şafak and Çelebi 1991; Çelebi 1993; Rodgers and
Çelebi 2006; Todorovska and Trifunac 2008). Several research studies have illustrated that even buildings with nominally symmetric plans could exhibit markedly torsional responses when subjected to purely translational excitations. These significant torsional responses could be attributed primarily to the yielding of the structure predominantly in one resisting plane (De la Llera and Chopra 1994; De la Llera et al. 2001; Hegde and Sinha 2008).

2.6 Previous studies on the effect of diaphragm flexibility and torsion on FRS

Only a few studies have investigated the effects of the in-plane diaphragm flexibility and torsion on floor motions characteristics of instrumented buildings and/or numerical building models. Celebi et al. (1989) studied a single-story instrumented building with a flexible diaphragm exposed to a ground motion from the 1984 Morgan Hill earthquake. They showed that the amplification of the roof floor acceleration at the center of the diaphragm with respect to the edge was about 3.0. Through the numerical analysis of three building models, Tena-Colunga and Abrams (1996) showed that, in some cases, the in-plane diaphragm flexibility could increase floor acceleration responses. They also showed that torsional effects could reduce considerably as the in-plane diaphragm flexibility increases. In a study conducted by Qu et al. (2014) on a group of instrumented buildings, it was shown that PFA responses could be amplified by factors of up to 2.2 and 1.6 due to torsional effects and in-plane diaphragm flexibility, respectively. Bernal et al. (2014) investigated the effect of the in-plane diaphragm flexibility on FRS ordinates in a five-story instrumented building exposed to the Chino Hills earthquake. Comparing the floor spectral ordinates at different floor locations, they showed that the rigid floor assumption was valid for NSC frequencies up to approximately 2-3 Hz (i.e., $T_{\text{comp.}} = 0.33-0.50$ s), whereas beyond this range the floor diaphragm did not behave as rigid. As can be observed, except for the study conducted by Qu et al. (2014), previous research works in this area were based on only a single or a handful of instrumented buildings. Meanwhile, except for the study conducted by Bernal et al. (2014), these works only considered amplification in PFA responses but not in FRS ordinates.
2.7 Estimating the effects of diaphragm flexibility and torsion on PCA and PFA of instrumented buildings

Recorded accelerations by two parallel strong motion sensors at two different locations of a floor can be used to identify the presence of torsional responses and in-plane diaphragm flexibility in a building. In a regular building with rigid floor diaphragms, when the torsional component of the ground motion is not significant, the acceleration responses of these sensors are theoretically identical whereas differential acceleration responses are expected in buildings with plan irregularities or with flexible floor diaphragms. These behaviors, which cannot be captured using 2D building models, are denoted as 3D effects in this paper. Determining whether the mentioned differentiation is specifically due to the in-plane floor diaphragm flexibility or due to torsional responses in many instrumented buildings is very challenging because of the lack of information regarding the exact distribution of building mass, distribution and stiffness of interior partition walls, stiffness of the exterior walls, geometry of the floors plan (e.g., presence of large openings), etc. In this section, the authors consider different scenarios to distinguish the sources of the observed differentiations in floor spectral ordinates obtained from two parallel sensors of a floor. However, the main goal herein is to highlight the influence of the location of acceleration measurements on the magnitude of the floor spectral ordinates regardless of the sources of the differentiation between the responses of two parallel sensors.

Figures 2-3a and b illustrate the in-plane diaphragm flexibility effects on the floor spectra of two example single-story instrumented buildings. As the figures show, the in-plane diaphragm flexibility causes a single-story building to behave as a multi-degree-of-freedom system with multiple modes of vibrations that can affect NSCs with various periods and/or locations differently. The results shown in these figures can be used to examine the range of NSC period for which the rigid floor assumption is valid.
As shown in Figure 2-3a, for the single-story Hospital building, floor pseudo-spectral acceleration (FSa) responses at the roof center are significantly larger than (by a factor up to 2) those at the roof periphery in the NSC period range of 0-0.35 s. Beyond this period range the spectral acceleration responses of the two sensors are almost identical, suggesting that for this case the rigid diaphragm assumption is valid. An evaluation of the roof spectra of the Hemet single-story Library building illustrates reductions (by a factor up to 1.7) in spectral ordinates at the roof center with respect to the roof periphery for the NSC period range 0-0.1 s (see Figure 3b). In this case, because of the frequency content of the ground motion, the period lengthening due to the in-plane diaphragm flexibility causes a reduction in FSa responses at the roof center with respect to those at the roof periphery. For the NSC period range 0.1-0.3 s, the spectral ordinates at the roof center are larger than those at the roof periphery by a factor up to 3.0. These observations show that for the NSC period range of approximately 0-0.3 s, the floor diaphragm can be considered as flexible. For NSC periods larger than 0.3 s, the diaphragm can be considered as rigid.

Figures 2-4a and b illustrate the roof FRS results for two multistory instrumented buildings in which the 3D effects are evident. As seen in Figure 2-4a, in the period range 0-0.5 s, the floor response spectra at the roof center and roof periphery deviate, but beyond this range the two floor spectra match. For the example, as seen in Figure 2-4b, differentiation between roof spectra at the center and periphery in the period range 0-0.4 s and the periods larger than 0.6 s is insignificant,
however, this differentiation in the period range 0.4-0.6 s is significant (at NSC period of 0.5 s floor spectral acceleration at the south edge is 1.7 times that at the north edge).

Figure 2-4 0.05-damped roof FRS for two example instrumented buildings

In the database of the instrumented buildings used in this study, a total of 135 floor cases (including both principal directions of buildings) were instrumented with more than one strong motion sensor. Figure 2-5 depicts different sensor arrangements that are observed in instrumented buildings floors. A single-span floor with a symmetric plan and symmetric perimeter lateral-load resisting elements is selected as the baseline case. For this baseline case, Eq. 2.3 and 2.4 can respectively express the PFA amplification factors due to the torsional effects and due to the in-plane diaphragm flexibility.

\[
\gamma_{\text{PFA}}^{\text{torsion}} = \text{Max} \left( \frac{PFA_1, PFA_2}{\text{Avg} (PFA_1, PFA_2)} \right) \quad (2.3)
\]

\[
\gamma_{\text{PFA}}^{\text{flexibility}} = \frac{PFA_{\text{center}}}{\text{Avg} (PFA_1, PFA_2)} \quad (2.4)
\]

Similar equations as Eqs. 2.3 and 2.4 can be used to estimate the amplification of PCA due to the 3D effects. Note that PCA is the maximum FSa at the NSC period range of interest. Hence, the adopted strategy gives a single amplification factor for an entire FRS, and can predict, for example, if the assumption of the rigid floor diaphragm for the critical NSC period (the one associated with the PCA) is valid or not.
For situations other than the baseline case, Eqs. 2.3 and 2.4 should be modified accordingly. For example, if the building is symmetric in plan and the lateral-load resisting system is a box-shape core shear wall, the $PFA_1/PFA_{center}$ ratio can show the in-plane diaphragm flexibility amplification factor. Based on the in-plane stiffness of the diaphragm and the characteristics of the lateral-load resisting system (e.g., geometry), the sensor arrangement shown in Figure 2-5c can capture either the presence of the torsional responses or the in-plane diaphragm flexibility. This statement is described in detail for an example building with perimeter lateral-load resisting elements discussed next. As will be illustrated in this section, generally 3D effects do not occur simultaneously. This means that, for example, if the diaphragm can be classified as flexible in its plane, the torsional effects on acceleration responses are negligible. Therefore, for a building with perimeter lateral-load resisting elements, $PFA_{center} > PFA_1$ means that the floor diaphragm is flexible in its plane, and torsional effects are not significant; in this case $PFA_1 = PFA_2$, and using Eq. 2.4, the value of the amplification factor due to the in-plane diaphragm flexibility, $γ_{flexibility}^{PFA}$, is $PFA_{center}/PFA_1$. The opposite case, i.e., $PFA_1 > PFA_{center}$, reveals the presence of torsional responses suggesting that the diaphragm is rigid; in this case $Avg (PFA_1, PFA_2) = PFA_{center}$, and using Eq. 2.3, the amplification factor due to torsional responses, $γ_{torsion}^{PFA}$, is $PFA_1/PFA_{center}$.

For the sake of clarity, the detailed calculations of 3D effects for the example buildings whose FRS results were presented in Figures 2-4a and b are described next. For the roof floor considered in Figure 2-4a, the sensors location corresponds to the sensor layout c (Figure 2-5c). As seen in this figure, the PFA response at the roof edge and at the center is 0.19 and 0.29 g, respectively. The PCA response at the roof edge and at the center is 0.85 and 1.19 g, respectively. Considering the fact that acceleration responses at the roof center are larger than those at the roof periphery, the amplification at the roof center is most likely due to the in-plane diaphragm flexibility. Using Eq.
2.4, the parameter $\gamma_{\text{flexibility}}^{\text{PFA}}$ is 1.53, and the parameter $\gamma_{\text{flexibility}}^{\text{PCA}}$ is 1.40. The sensors location in the roof floor, evaluated in Figure 2-4b, is representative of the sensor layout b, and hence can capture both diaphragm flexibility and torsion. For this case, the PFA response at the south and north edges of the roof is 0.51 and 0.33 g respectively, whereas this quantity at the roof center is 0.41 g. The PCA response at the south edge, north edge, and the roof center is 2.33, 1.41, and 1.91 g respectively. As seen, with an acceptable accuracy, the PFA response at the roof center is equal to the average responses of the two edges (this statement is true for the PCA as well). Hence, for this building the floor diaphragm is relatively rigid. The parameters $\gamma_{\text{torsion}}^{\text{PFA}}$ and $\gamma_{\text{torsion}}^{\text{PCA}}$ at the roof level are 1.24 and 1.22, respectively.

An evaluation of the floor spectra obtained from the instrumented buildings indicates that, because of the extensive in-plane diaphragm flexibility, the 3D effects in single-story buildings are more predominant and significantly different than those in multistory buildings (see Figures 2-6 to 2-9). Hence, single-story buildings are investigated separately. Figure 2-6a illustrates $\gamma_{\text{flexibility}}^{\text{PFA}}$ and $\gamma_{\text{flexibility}}^{\text{PCA}}$ for all single-story instrumented buildings with available data that allows for the calculation of the 3D effects. As seen, on average, 3D effects are more predominant in PCA responses, although in several cases the amplification factor in PFA is larger. In addition, $\gamma_{\text{flexibility}}^{\text{PFA}}$ and $\gamma_{\text{flexibility}}^{\text{PCA}}$ are in the 0.67-4.09 and 0.62-5.07 ranges, respectively. Figure 2-6b presents amplification factors due to the torsional responses of the single-story buildings illustrating that $\gamma_{\text{torsion}}^{\text{PFA}}$ and $\gamma_{\text{torsion}}^{\text{PCA}}$ are bounded to 1.19 and 1.34, respectively.

![Figure 2-6](attachment:image.png)

**Figure 2-6** Acceleration amplification factors for the single-story instrumented buildings
The parameters $\gamma_{\text{flexibility}}^{\text{PFA}}$ and $\gamma_{\text{flexibility}}^{\text{PCA}}$ depend on the diaphragm modal periods (i.e., mass and in-plane stiffness), the frequency content of the ground excitation (i.e., tuning with the diaphragm modal periods), and the diaphragm damping ratio. In general, plywood diaphragms and irregular buildings exhibit larger $\gamma_{\text{flexibility}}$ and $\gamma_{\text{torsion}}$, respectively, which is consistent with expectations; for example, all 11 cases with a $\gamma_{\text{flexibility}}$ larger than 3.0 have flexible plywood diaphragms; or four out of five cases with a $\gamma_{\text{torsion}}$ larger than 1.05 belong to the building located at Station 89473 that exhibits plan irregularities. Exact details of the individual buildings characteristics (e.g., location of lateral-load resisting elements, and the nonstructural walls stiffness and distribution in plan, etc.) are needed to interpret the results more accurately. For example, for the building at Station 23495, the amplification factors are smaller than those of buildings with similar floor geometry. Further investigations of the building architectural plan illustrate the presence of a firewall at the floor mid-span that has reduced the free span of the floor diaphragm. Hence, the firewall is most likely the reason for the smaller amplification factor observed in this building with respect to buildings with similar geometry.

An evaluation of the parameters $\gamma_{\text{torsion}}$ and $\gamma_{\text{flexibility}}$ for individual single-story buildings that have captured both the in-plane diaphragm flexibility and torsional effects is shown in Figures 2-7a and b. As seen in Figure 2-7a, $\gamma_{\text{flexibility}}^{\text{PFA}}$ varies in the wide range of 1.12-4.09, whereas $\gamma_{\text{torsion}}^{\text{PFA}}$ is bounded to 1.19. In general, larger $\gamma_{\text{torsion}}$ values are associated with the rigid diaphragms; for example, the most critical case with $\gamma_{\text{torsion}}^{\text{PFA}} = 1.19$ is the building located at Station 54331 with a relatively rigid concrete roof diaphragm. These observations suggest that the in-plane diaphragm flexibility can mitigate torsional responses, apparently at the expense of significantly amplified acceleration responses at the floor’s mid-span.
Simultaneous evaluation of amplification factors due to diaphragm flexibility and torsional responses for the (a) PFA; (b) PCA; for individual single-story building-directions (a sample size of 18)

Single-story buildings with long free-span roofs are generally used for light industrial, commercial, and recreational purposes and represent a vast proportion of the building stock in the US. An evaluation of the 3D effects in single-story buildings illustrates that neglecting the in-plane diaphragm flexibility can significantly underestimate the maximum acceleration responses (i.e., PFA and PCA) around the center of a roof supported at its two ends. This observation indicates that the ASCE 7-16 design equations for NSCs, which do not take into account the in-plane diaphragm flexibility, may lead to unconservative seismic design accelerations for NSCs mounted at the mid-span roofs in typical single-story buildings.

Figure 2-8a illustrates the variation of the parameters $\gamma_{\text{flexibility}}^{\text{PFA}}$ and $\gamma_{\text{flexibility}}^{\text{PCA}}$ with the relative height, $z/h$, for the multistory building-directions studied in this paper. It is observed that in some cases although PFA responses of the two parallel sensors of a floor are similar, the magnitudes of their PCA responses are distinct. For example, for the building located at Station 24602 in the N-S direction at $z/h = 0.93$ the parameter $\gamma_{\text{flexibility}}^{\text{PFA}}$ is 1.17 whereas $\gamma_{\text{flexibility}}^{\text{PCA}}$ is significantly and equal to 1.51. This behavior is due to differences between the harmonics, which form part of the frequency content of the floor motion, at various NSC periods. As another observation, in two floor cases, a reduction in the acceleration responses is present. A possible hypothesis for this decrease is the beneficial effect of the in-plane diaphragm flexibility in shifting the longest predominant period of the floor motion at the roof mid-span to the low-acceleration region of the floor response spectrum. Alternatively, this reduction could also be due to the presence of a stiff nonstructural wall at the roof mid-span that has caused the in-plane diaphragm stiffness at the roof
mid-span to be larger than that at the roof periphery (enough information is not available for a definitive conclusion). According to Figure 2-8a, $\gamma_{\text{flexibility}}^{\text{PFA}}$ varies from 0.87 to 1.66 whereas this range for the PCA, shown in Figure 8b, is 0.76 to 2.00, which is slightly larger than that of the PFA. The mean and mean plus one standard deviation values for $\gamma_{\text{flexibility}}^{\text{PFA}}$ are 1.15 and 1.4, respectively. These statistics for $\gamma_{\text{flexibility}}^{\text{PCA}}$ are 1.22 and 1.5, respectively.

$\gamma_{\text{flexibility}}^{\text{PFA}}$ varies from 0.87 to 1.66 whereas this range for the PCA, shown in Figure 8b, is 0.76 to 2.00, which is slightly larger than that of the PFA. The mean and mean plus one standard deviation values for $\gamma_{\text{flexibility}}^{\text{PFA}}$ are 1.15 and 1.4, respectively. These statistics for $\gamma_{\text{flexibility}}^{\text{PCA}}$ are 1.22 and 1.5, respectively.

$\gamma_{\text{flexibility}}^{\text{PFA}}$ varies from 0.87 to 1.66 whereas this range for the PCA, shown in Figure 8b, is 0.76 to 2.00, which is slightly larger than that of the PFA. The mean and mean plus one standard deviation values for $\gamma_{\text{flexibility}}^{\text{PFA}}$ are 1.15 and 1.4, respectively. These statistics for $\gamma_{\text{flexibility}}^{\text{PCA}}$ are 1.22 and 1.5, respectively.

Figure 2-8 illustrates the variation of $\gamma_{\text{torsion}}^{\text{PFA}}$ and $\gamma_{\text{torsion}}^{\text{PCA}}$ with the relative height for the multistory instrumented buildings. As seen, the $\gamma_{\text{torsion}}$ parameter for the PFA and the PCA varies in the 1.0-1.53 and 1.0-1.46 ranges, respectively. The mean and mean plus one standard deviation for the parameter $\gamma_{\text{torsion}}^{\text{PFA}}$ is 1.10 and 1.20, respectively. These parameters are approximately the same for $\gamma_{\text{torsion}}^{\text{PCA}}$. The largest torsional amplification factor ($\gamma_{\text{torsion}}^{\text{PFA}} = 1.53$) belongs to the building located as Station 24567 (at $z/h = 0.14$) that has a U-shaped irregular plan. However, as an important observation, in several instrumented buildings with symmetric geometry and symmetric lateral load resisting elements (i.e., nominally regular) the torsional responses are identified. Examples of this behavior are listed next: $\gamma_{\text{torsion}}^{\text{PFA}}$ for buildings located at Stations 1260 (at $z/h = 0.17$) and 58480 (at $z/h = 0.6$) is 1.29 and 1.27, respectively. $\gamma_{\text{torsion}}^{\text{PFA}}$ for buildings located at Stations 58394, 24231, 24464, 47459, and 24322 at the roof level is 1.24, 1.20, 1.29, 1.31, and 1.33, respectively. Torsional effects in these nominally regular buildings could be attributed to a variety of sources such as the local yielding or the asymmetric yielding of the building, accidental torsional moments caused by eccentricities between the centers of rigidity and mass, and the torsional components of earthquake ground motion.

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An evaluation of the $\gamma_{\text{flexibility}}$ and $\gamma_{\text{torsion}}$ values for the individual instrumented buildings studied herein does not demonstrate a consistent variation of these parameters along the height of buildings. For example, for the building located at Station 24567 in the E-W direction considering the available sensors, $\gamma_{\text{torsion}}^{\text{PFA}}$ increases along the height; for the building at Station 24652 in the N-S direction, a reduction along the height is observed; for the building located at Station 58480 in the E-W direction, the variation of this parameter with height is irregular. When evaluating the 26 cases with enough information to identify a trend, in 11, nine and six cases, the variation of $\gamma_{\text{torsion}}^{\text{PFA}}$ along the height shows an increasing, decreasing and irregular trend, respectively. Furthermore, $\gamma_{\text{flexibility}}$ and $\gamma_{\text{torsion}}$ are not found to be strongly dependent on the type of lateral-load resisting system. Generally, $\gamma_{\text{torsion}}$ is larger for irregular buildings with rigid diaphragm, which is consistent with expectations.

Only in the case of nine multistory instrumented building-directions enough information is available to quantify torsional and in-plane diaphragm flexibility behaviors simultaneously. Figures 2-9a and b illustrate the evaluation of the effects of these behaviors on PFA and PCA responses respectively, in the nine considered samples. As seen, for a given floor motion, when one amplification factor is large, the other one is not significant (the only exception is case No. 4). In other words, it is apparent that, in general, these 3D effects do not occur simultaneously. This observation is consistent with the conclusion made by Tena-Colunga and Abrams (1996) using three numerical building models.

The results of this section illustrate that torsional-induced acceleration responses are more pronounced in multistory buildings than in single-story buildings, whereas floor accelerations in single-story structures tend to be most significantly affected by the in-plane diaphragm flexibility. This latter observation is consistent with the results of Sadashiva et al. (2012) for displacements responses.
Simultaneous evaluation of amplification factors for the multistory instrumented building directions (a sample size of 9): (a) for PFA; (b) for PCA

Theoretically, if the lateral-load resisting system elements or floor diaphragms of a building experience inelastic actions during an earthquake, the discussed 3D effects may be mitigated. This is because inelasticity can alter the floor motion frequency content and filter out some detrimental narrow-band motions in frequency ranges that affect NSCs the most. This behavior is investigated in the responses of instrumented building next. Figures 2-10a and b demonstrate the variation of $\gamma_{\text{flexibility}}$ with the normalized IM for the roof level of the single-story and multistory instrumented building-directions, respectively.

Variation of amplification factor due to diaphragm flexibility with the normalized IM for the roof level of the (a) single-story; (b) multistory; instrumented building-directions

Among all single-story building-directions for which the instrumentation allows for estimating the in-plane diaphragm flexibility, the recorded motions are available for more than one event in just six individual cases. These six cases can be used for a direct investigation of the effect of IM on the parameter $\gamma_{\text{flexibility}}$. An evaluation of the responses of these cases illustrates an irregular
trend in $\gamma_{\text{flexibility}}$ when IM increases. For example, for the building located at Station 89473, recorded motions are available for four different events in the north-south direction. Results show that for this case with increasing IM from 0.15 to 0.27, $\gamma_{\text{flexibility}}$ decreases whereas with increasing IM from 0.27 to 0.43 it increases. For single-story structures, even at higher intensity ground motions amplification factors are significant (i.e., $\gamma_{\text{PFA}}^\text{PFA} = 5.0$ for the normalized IM of 0.43). This could be attributed to the fact that there is a large overstrength present in many single-story structures, which causes these buildings to behave elastically even in severe ground motions. In the case of multistory building directions, only for two samples (buildings located at Stations 24571 and 24601 in the east-west direction), available data allows for such an evaluation. For both cases instrumentation results are available for only two events. For Stations 24571 with increasing IM from 0.06 to 0.10, $\gamma_{\text{flexibility}}$ decreases from 1.15 to 1.11. For Stations 24601 with increasing IM from 0.20 to 0.24, $\gamma_{\text{flexibility}}$ reduces from 1.08 to 1.04. These results are consistent with expectations, however, given the slight increase in the IM and the associated slight decrease in $\gamma_{\text{flexibility}}$ for these two examples, the data does not allow for drawing a strong conclusion regarding a general behavior. For multistory buildings’ diaphragm flexibility, there are not enough samples present. Therefore, seismic demand quantification studies that involve modern-code based designed buildings subject to ground motion intensity levels at the design level earthquake, which can cause inelasticity in the lateral-load resisting system or floor system diaphragm, are needed to propose improved code equations for single-story buildings.

Figures 2-11a and b demonstrate the variation of $\gamma_{\text{torsion}}$ with the normalized IM for the roof level of the single-story and multistory instrumented building-directions, respectively. Among all single-story building-directions for which the instrumentation allows for estimating the torsional effects, only five individual cases have available recorded motions for more than one event. These five cases can be used for a direct investigation of the effect of IM on the parameter $\gamma_{\text{torsion}}$. An evaluation of these five samples illustrates an inconsistent trend regarding the change in the value of $\gamma_{\text{torsion}}$. For the multistory buildings, the available data for seven cases allows for such an evaluation. For these cases also, a consistent trend is not observed regarding the change of $\gamma_{\text{torsion}}$ with increasing IM. As shown in Figure 2-11b, three individual multistory cases are present for which IM is higher than 0.50. For these three samples $\gamma_{\text{torsion}}$ is below 1.10. This could be viewed as evidence of the hypothesis that building inelasticity can mitigate torsional response, however,
code-based numerical results are needed to quantify these effects. Such models should incorporate all parameters affecting the torsional responses such as actual accidental torsion and asymmetric yielding of the lateral-load resisting elements. The results of this section suggest that capturing the torsion and in-plane diaphragm flexibility effects using the numerical models can be very challenging because it requires the incorporation of complicated parameters discussed. Therefore, it is recommended that in the absence of such models the mean plus one standard deviation values computed in this study be used to incorporate these 3D effects on the PFA and PCA responses, even when the supporting building is nominally regular with rigid floor diaphragms.

Figure 2-11  Variation of amplification factor due to torsional responses with the normalized IM for the roof level of the (a) single-story; (b) multistory; instrumented building-directions

2.8 Consequences of misestimating the seismic base location in buildings with basements

The seismic base location influences the estimation of acceleration demands on NSCs provided by Eq. 2.1. In ASCE 7-16, the seismic base is defined as “the level at which the horizontal seismic ground motions are considered to be imparted to a structure”. For buildings with one or more stories below grade, numerous parameters may influence the location of the seismic base including, but not limited to, the soil condition adjacent to the building, the stiffness of the basement walls, the location and extent of seismic separations, and the basement depth. In order to establish the seismic base at the grade level, stiff soils and stiff walls are required over the depth of the basement (refer to ASCE 7-16 Section C11.2). This section evaluates the effect of the misestimation of the location of the seismic base in buildings with basements on the NSCs design force given by Eq. 2.1. Within this evaluation, it is illustrated that in several instrumented
buildings, in spite of the presence of the concrete retaining walls in the basement perimeter, conditions for establishing the seismic base at the ground level (grade) are not satisfied.

Regardless of the presence of a basement, the soil-structure-interaction (SSI) can alter a building’s dynamic characteristics and influence the magnitude of the floor spectral ordinates. The foundation movements may elongate a building fundamental period, which can either increase or decrease the structural seismic demands depending on relationship between the characteristics of the building and the input excitation. Furthermore, the foundation inelasticity and energy dissipation at the soil–foundation interface may decrease the force demands induced in the structure (Raychowdhury and Ray-Chaudhuri 2015). Several past studies have illustrated that SSI may significantly alter the force and displacement demands, especially on low to mid-rise buildings (Yim and Chopra 1984; Allotey and El Naggar 2008; Harden and Hutchinson 2009; Raychowdhury 2011). As per the authors’ knowledge, only the numerical study by Raychowdhury and Ray-Chaudhuri (2015) has dealt with understanding the effect of SSI on the response of NSCs mounted on buildings with shallow foundations. No studies have been yet conducted to understand and quantify this effect on NSCs housed in buildings with basement.

In the complied database for this study a total of 22 buildings (i.e., 44 principal building-directions) have one or more stories below grade. When the basement and the ground floor were both instrumented, an evaluation of the recorded motions at these two levels can provide sufficient information to estimate the seismic base location. Theoretically, assuming the seismic base at the ground floor level implies that the section of the structure below the ground level behaves as a rigid body. In this situation, it is expected that no amplification in the floor acceleration responses would occur at the ground level with respect to the basement. The presence of amplifications may suggest that the criteria to establish the seismic base at the ground level are not satisfied. As an example, the normalized PFA and PCA profiles for the LA 19-story office building with a four-level basement (19/4) are illustrated in Figures 2-12a and b, respectively.
Figure 2-12  PFA/PGA and PCA/PGA profiles for the LA 19-story Office Building with a four-level basement (19/4)

As seen in Figure 2-12a, for the N-S and E-W directions the PCA at the ground floor is amplified by a factor of 1.80 and 1.70 with respect to the ground level PGAs, respectively. For this case, the PCA occurs in the NSC period range 0-0.5 s, which is the period range for typical NSC, and hence, such amplifications are important. These amplifications occur in the presence of 0.30 m thick concrete walls along the perimeter of the basements.

Overall for 16 building-directions (out of the overall 44 samples with below-grade stories considered in this study), available data allows for the assessment of the seismic base location using the abovementioned strategy. The complete list and characteristics of these buildings are provided in the Appendix. For these 16 building-directions, the ratios of the PFA and PCA responses at the ground floor to those at the foundation level (denoted as the amplification factor) are presented in Figure 2-13a. For floors instrumented by more than one ground motion sensor, the mean PFA (also PCA) response is used in computing the amplification factors. It should be noted that in cases No. 1 and 2, basements have exterior and interior RC shear walls; in cases No. 3 and 4, basements have perimeter concrete block walls (not connected to columns); in other cases, basements have perimeter RC walls.
Figure 2-13 Amplification of the PFA and the PCA responses at the ground floor level with respect to the foundation level

As shown in Figure 2-13a, only in three cases (i.e., LA 6-story Office Building in the E-W direction, and Pasadena nine-story Building in the N-S direction for two different events) the amplification factors for the PFA and PCA responses are near unity, meaning that the seismic base can be established at the ground floor level. Even though these buildings have concrete basement walls along the entire embedment depth, large amplifications are still observed in several cases. For discussion purposes, if an amplification factor of 1.25 is defined as a threshold, as seen in Figure 2-13a, only five out of 16 cases exhibit PFA and PCA amplification factors below this threshold. The exact construction details of these buildings are required to establish an accurate hypothesis as to why the conditions for establishing the seismic base at the ground level are not satisfied (the reason could be, for example, the separation between basement walls and the floor diaphragms, the inadequacy of the adjacent soil conditions, the inadequacy of the stiffness of basement walls, etc.). For instance, the 47-story building (case No. 13 and 14 in Figure 2-13) is located in a soft-soil profile. Çelebi (1993) showed that in this structure the maximum rotation of the basement shear wall was 6-7 times that of the building foundation. Hence, the presence of soft soil adjacent to the basement is the most likely reason why the conditions for establishing the seismic base at the ground level are not satisfied for this building. The correlation between the normalized IM and the observed amplification factors are investigated in Figure 2-13b. As seen, even for higher intensity ground motions, relatively large amplification factors are observed.

The effect of the misestimation of the seismic base on the NSCs design forces, calculated by Eq. 2.1, is more pronounced in low-rise buildings with a deeper embedment because in such buildings, the term \(1 + 2z/h\) is more sensitive to the embedment height. For example, for a
building with three and two stories above and below the grade (3/2), respectively, if the seismic base is assumed to be at the ground floor level, the term \((1 + 2z/h)\) at the ground floor level is 1.0 whereas if the seismic base is established at the foundation level, this term at the ground floor level is 1.8. For a building with a 13/2 configuration, these values are 1.0 and 1.3, respectively. Hence, for high-rise buildings, when the NSCs seismic demands are calculated using Eq. 2.1, the effect of the misestimation of the seismic base is not as significant as for the case of low-to-midrise buildings.

The conducted evaluation shows that for the instrumented buildings with basements, the ratio of PFA and PCA response at the ground floor level to the corresponding values at the lowest basement level can be as large as 2.0. The results of this section highlight the importance of considering SSI effects when designing NSCs. In addition, the results strongly suggest that when using Eq. 2.1 for calculating seismic demand force on NSCs in buildings with basements, if an exact SSI analysis is not performed or there is credible reason to doubt where to assume the seismic base, it should be established at the foundation elevation.

2.9 Effects of the other characteristics of supporting building on FRS

The ASCE 7-16 \(F_p\) equation is not a function of the supporting building characteristics (see Eq. 2.1). Many previous research works (e.g., Sankaranarayanan and Medina 2007) have shown that FRS ordinates strongly depend on modal periods and the lateral-load resisting system of the supporting structure. In this section, the effect of supporting building characteristics on the shape and magnitude of the roof FRS of the instrumented buildings is investigated. Single-story structures, which are significantly influenced by the in-plane diaphragm flexibility, are excluded from this investigation. Roof acceleration motions are available in the CESMD database for 82 out of the total 86 principal directions of the multistory buildings selected for this study. For four cases including two SWs (i.e., buildings located at Stations 25213 and 24236 in the E-W direction) and two MRFs (i.e., buildings located at Station 24567 in both directions exposed to ground motions from the 1991 Sierra Madre earthquake) roof floor motions were not provided in the CESMD database. For the two SW cases, floor motions at upper stories with \(z_1/h_1 = 2/3\) and \(z_2/h_2 = 12/14\) are used in the evaluation of this section. For the two MRF cases, information is not available for upper floor levels, hence, they are not considered. If more than one sensor is
available for a roof floor, the sensor whose response is less affected by the in-plane diaphragm flexibility and/or torsional responses is selected. With these modifications and criteria, a total of 84 roof floor motions are included in the evaluation of this section.

The 0.05-damped elastic spectra for all of the 84 roof-directions are derived and shown in Figure 2-14a. In this figure, the NSC period, \( T_{\text{comp.}} \), is normalized to \( T_{1\text{bldg}} \) to allow for a comparison between different floor spectra. Furthermore, the floor spectral acceleration (FSa) responses are normalized to the PGA of individual records because herein the spectral shape is of interest, but the absolute values are not.

\[ \frac{\text{FSa}}{\text{PGA}} \]

\[ \frac{\text{T}_{\text{comp.}}}{\text{T}_{1\text{bldg}}} \]

(a) all 84 roof acceleration motions

(b) examples that exhibit spectral shapes consistent and inconsistent with those expected from typical MRF numerical models

\textbf{Figure 2-14} \hspace{1cm} 0.05-damped roof FRS for multiistory instrumented buildings selected for this study

From Figure 2-14a, it can be observed that if all of the 84 cases are grouped together, regular trends cannot be identified in the shape and magnitude of the floor spectra. As shown, the minimum and maximum values of PFA/PGA (i.e., FSa/PGA at \( T = 0 \)) are 0.40 and 5.58, respectively. FSa/PGA at \( T_{1\text{bldg}} \) varies from 0.23 to 25.15. The mean values of PFA/PGA, and FSa/PGA at \( T_{1\text{bldg}} \) are 1.93 and 6.00, respectively. The standard deviations of these parameters are 0.99 and 5.39, respectively. These observations highlight the significant variation present in the normalized spectral ordinates, especially in proximity to the supporting buildings modal periods. In the remainder of this section, an effort to group floor response spectra based on their most salient characteristics is made. As a first step, roof floor response spectra are classified based on their lateral-load resisting system. An evaluation of the roof floor spectra illustrates that 48, 25 and five spectra are derived from the instrumented building-directions with MRF, SW and BR systems,
respectively. Three roof spectra correspond to the instrumented building-directions with a hybrid system, and three roof spectra correspond to dual lateral-load resisting systems. Further investigations illustrate that the floor spectra of the hybrid and dual systems exhibit characteristics similar to the MRF systems. Hence, they are considered part of the MRF family. Since only five cases cannot provide a sufficient population size to identify a reliable trend for the BR family, they are not considered herein.

Many research works conducted on the estimation of seismic demands on NSCs are based on simplified two-dimensional numerical building models. These numerical models primarily represent the behavior of only the lateral-load resisting system and not necessarily of a complete building. For example, the roof FRS of typical midrise MRF numerical models exhibit two discernible spikes at the NSC periods near the first mode of the building, $T_{\text{bdg.}}$, and near the second mode of the building $\sim T_{\text{bdg.}}/3$, whereas at other NSC periods, the spectral ordinates are not significant. Further evaluation of the results shown in Figure 2-14a illustrates that roof spectra extracted from several instrumented building-directions exhibit significant inconsistencies with respect to the responses of the simplified numerical models generally used in the literature. In this section, based on the trend observed when using typical numerical models, the floor spectra are categorized into two groups; consistent and inconsistent spectra. For instance, for an instrumented building-direction with an MRF system, if the FRS exhibits ordinates or a shape dissimilar to an FRS obtained from typical MRF numerical models, it is referred to as an inconsistent FRS within the MRF family. Figure 2-14b presents a single consistent, and two inconsistent roof spectra for the MRF instrumented buildings. As seen, one inconsistent case exhibits relatively low ordinates at the modal periods of the supporting building, and the other one exhibits a significantly distinct shape with several spikes at the NSC periods not predicted by numerical building models.

The assessment of the roof spectra shows that the mentioned inconsistencies are mostly present because of special behaviors such as in-plane diaphragm flexibility, torsional effect, vertical irregularity in mass and stiffness that are not taken into account in typical numerical building models. Because ASCE 7-16 is meant to provide equations for new building designs, the authors postulate that these special cases, which are not representative of code compliant modern buildings, should be filtered out from the basic (baseline) evaluation of the ASCE 7-16 $F_p$ equation. It is proposed that the basic design equations for NCSs be established based on the responses of
regular buildings (consistent cases). As for the inconsistent cases, based on the reason for their inconsistency (e.g., the in-plane diaphragm flexibility), a correction/modification factor could be incorporated into the design equations. The evaluation of the results illustrates that the inconsistent cases can be considered under two broad groups: identified by their (i) shapes (i.e., jagged or irregular shapes); (ii) magnitude (i.e., relatively small or large normalized spectral ordinates). A detailed description of the most salient cases exhibiting these behaviors is provided in the subsequent sections.

2.10 Inconsistent roof spectra identified by their shapes

As an example, for a midrise building with an MRF system, theoretically, two discernible spikes are expected at $T_{\text{comp}}/T_{1\text{bldg}} = 1.0$ and $1/3$. Several instrumented building-directions do not follow this pattern. For instance, the LA 13-story Office Building (Station 24567) and the LA nine-story Office Building (Station 24579) in both principal directions are U-shape in plan, hence, their spectra are affected by torsional responses resulting in irregular shapes with significant spikes at unexpected NSC periods (i.e., other than $T_{1\text{bldg}}$ and $T_{1\text{bldg}}/3$). The LA five-story Warehouse Building (Station 24463) in the E-W direction is another example with this inconsistency.

2.11 Inconsistent roof spectra identified by their magnitude

1) Inconsistent roof spectra with relatively low normalized spectral ordinates at the building modal periods. This behavior is usually caused by the very low energy content of the ground motion records near the building modal periods. For example, the Sherman Oaks four-story Commercial Building (Station 24680) in both directions exhibits this behavior. The Oakland 24-story Residential Building (Station 58483) in both principal directions is another example of this behavior.

2) Inconsistent roof spectra with relatively large normalized spectral ordinates: the primary reason for these relatively large normalized responses is the fact that many of these buildings experienced relatively small intensity ground motions with a consequent elastic behavior. As shown by Anajafi and Medina (2018a) if only the responses of instrumented buildings that experienced ground motion intensity levels consistent with the DE are considered, normalized 0.05-damped floor spectral ordinates are limited to 12.0. However, in Figure 2-14a, where the
responses of all buildings are present relatively large values such as 33 are observed. Besides this elastic behavior, some other causes are identified that might have further amplified these large spectral ordinates. In the following paragraphs these possible causes are discussed.

2-1) Amplification of $PCA/PGA$ due to additional irregularity effects: for example, the 4-Story Hospital (Station 12299) in the N-E direction exhibited a ratio $PCA/PGA = 18.2$ at the roof level most likely because of the presence of a vertical irregularity; the San Jose 3-story Office Building (Station 57562) in both principal directions exhibited $PCA/PGA = 17.2$ and 12.8 most likely because of the presence of a plan irregularity.

2-2) Amplification of $PCA/PGA$ in upper stories that behave like appendages (or substructures) supported by the rest of the building: to clarify this behavior, the response of the Mammoth Lake single-story Hospital (Station 54331) exposed to the 2016 Morgan Hill earthquake is discussed next. This building has a relatively small penthouse whose mass is approximately 10% of the roof mass. In the E-W direction of the building the normalized PFA and PCA responses at the roof level are 1.64 and 5.87, respectively whereas for the penthouse these quantities are significantly larger (equal to 5.58 and 23.3, respectively). The same trend is observed in the N-S direction of this structure. This behavior is consistent with the response of a common tuned-mass-damper in which the damper’s mass, typically ranging from 1-5% of the primary building mass, experiences significantly larger seismic responses with respect to those of the main floors (Anajafi and Medina 2018b). Hence, in this building, the penthouse was most likely tuned to the first mode of the rest of the primary structure (i.e., a section of the structure that does not include this semi appendage) resulting in significantly large normalized acceleration responses. The PFA/PGA and PCA/PGA profiles for two example multistory buildings exhibiting this behavior are illustrated in Figures 2-15a and b.
Responses of two examples multistory instrumented buildings with relatively large normalized acceleration responses at top stories caused by the possible tuning of top stories (with a significantly smaller mass than the typical stories mass) to the rest of the primary building modal periods: (a) PFA/PGA; (b) PCA/PGA

As seen in Figure 2-15a, the PFA/PGA response at the elevator room floor of the Eureka 4-story hospital (CSMIP Station No. 89770) in the E-W direction is 5.60. This value is several times larger than the mean (PFA/PGA), previously observed in Figure 2-14a, which was only 1.93, and is much larger than the amplification at the other floor levels of this building. The same trend is observed in the PCA/PGA responses of this building. Evaluation of the architectural drawings illustrates a significant vertical mass irregularity in this building (several setbacks along the height). The floor mass at the top two stories is significantly smaller than the mass of other stories. Hence, these upper stories were most likely tuned to the first mode of the rest of the primary building resulting in significantly large PCA/PGA responses.

Another example for this tuning behavior is the SF 47-story building (CSMIP Station 58532) in the N-S direction whose roof mass is roughly estimated to be less than one-third of the typical floors’ mass. In this structure, the top story was possibly tuned to the second mode of the rest of the primary structure (the large spike at $T_{\text{comp.}}/T_{\text{1bdg.}} = 0.20$ in Figure 2-14a corresponds to this case). As shown in Figure 2-15b, for this building PCA/PGA at the roof floor in the N-S direction is 33.0, which is 5.5 times as large as the mean (PCA/PGA) observed in Figure 2-14a. Furthermore, the acceleration responses at the roof floor are significantly larger than those at the lower story levels (the PFA and PCA responses at the roof level in the N-S direction are respectively 3.6 and 5.9 times those at the floor below the roof). The response of the SF 47-story in the E-W direction, shown in Figures 15a and 15b, can be considered a non-tuned case. In this case, the ratio between
the acceleration responses at the roof level and the acceleration responses at the lower floor levels is smaller than the corresponding ratio in the tuned case (i.e., the same building in the N-S direction). The SF four-story Office Building (Station 58261), in both principal directions, exposed to a ground motion from the 1989 Loma Prieta earthquake also exhibits very large normalized PCA in both principle directions (i.e., 23.3 and 24). In this structure, the mass of the roof is approximately the same as the mass of the typical floors but still one hypothesis for these large responses could be the potential tuning of the entire roof to the first mode of the rest of the building.

These observations suggest a potential need to revise the ASCE 7-16 provisions for designing a penthouse whose mass is generally less than typical floors mass. Consider the top stories with smaller mass discussed above are treated as a penthouse. As per the ASCE 7-16 Table 13.5-1, the penthouse is treated as an architectural component and should be designed using a NSC amplification factor of $a_p = 2^{1/2}$. For the penthouse elevation, the magnitude of in-structure amplification factor, (i.e., $1 + 2z/h$) is 3.0, and hence, using Eq. 2.2 results in a design PCA/PGA=7.5. Thus, the ASCE 7-16 upper limit of PCA/PGA= 4.0 governs the design. This value is smaller than the normalized floor acceleration responses experienced at the penthouse level in the discussed examples. As another important observation, if a NSC is mounted on a penthouse, it may experience relatively large acceleration demands, as shown in Figure 2-15b. It should be noted that these conclusions are based on recorded ground motions with relatively small intensities. The code equation for designing a penthouse could potentially be improved using results from code-compliant numerical models exposed to DE level ground motions.

2.12 Effect of the supporting building modal periods and lateral-load resisting system on FRS

In this section, the effects of the supporting building lateral-load resisting system and modal periods on the roof spectra of instrumented buildings are evaluated. In the previous sections, inconsistent roof floor motions were identified with behaviors that do not conform to the responses obtained from the typical regular building models. These inconsistencies were found to be primarily due to significant in-plane diaphragm flexibility, supporting building irregularity, or tuning of a top story to the modal periods of rest of the supporting building. It is postulated herein that these inconsistent cases should not be used as the basis for understanding the effect of
supporting building lateral-load resisting system and modal periods on the roof spectra because they are heavily influenced by a secondary parameter (i.e., the mentioned special behaviors). Hence, these roof floor motions are filtered out from the MRF and SW clusters. The 0.05-damped roof spectra for the consistent MRF (33 cases) and SW (16 cases) building-directions are presented in Figures 2-16a and b, respectively. The most salient observations from Figures 2-16a and b are summarized below:

1) The modal periods of most MRF building-directions follow the general rule of thumb approximation of \( T_{2\text{bldg.}}/T_{1\text{bldg.}} = 1/3 \). For the SW cases this ratio is between 0.25 and 0.75, which is different from the well-known \( T_{2\text{bldg.}}/T_{1\text{bldg.}} = 1/5 \) approximation for flexural SWs.

2) In most MRF building-directions the contribution of higher modes is evident. It is observed that some SW cases, referred to as w/o 2\textsuperscript{nd} spike in Figure 2-16b, do not exhibit a clear contribution of higher modes. This behavior generally occurs in short SWs whose second modal period falls in the range 0.1-0.2 s, for which typical ground motions do not have significant energy content.

3) The variation in the roof spectra for the MRF cases is larger than that for the SWs because the range of supporting building modal periods in the MRF building-directions is larger.

4) For taller and flexible MRF building-directions a discernible third-mode spike is observed in the roof spectra (referred to as w/ third spike in Figure 2-16a) because the third-mode of these cases is relatively large and situates in the period range that can be excited by typical ground motions.

Figure 2-16 0.05-damped roof FRS for the (a) consistent MRF multistory building-directions (a sample size of 33); (b) consistent SW multistory building-directions (a sample size of 16)

3-4
5) The normalized PCA responses (i.e., maximum FSa/PGA over the entire spectrum) of most SWs and most short- to midrise MRF systems occur at the vicinity of the first mode of vibration of the building, $T_{1\text{bldg}}$, whereas for the taller MRF cases, the spike in the vicinity of $T_{3\text{bldg}}$ governs the FRS. The dominance of roof spectra of short buildings by the fundamental mode occurs because, on the one hand, typical ground motions pose low energy content at the higher modes of these structures, and on the other hand, most of the instrumented buildings behaved in the elastic or near-elastic behavior range. This latter reason causes significant normalized acceleration demands at NSC in the vicinity of $T_{1\text{bldg}}$. Additionally, it is observed that the normalized PCA responses of the SWs are, on average, larger than those of the MRF buildings.

Significant variations observed in the floor spectra of different instrumented-building directions, the fact that PCA responses occur at different $T_{\text{comp.}}/T_{1\text{bldg}}$ ratios for different buildings, the special inconsistent behavior, and the difference between the experienced ground motion intensity by various buildings raise significant questions on some previous studies that have used the average spectra of all instrumented buildings for proposing NSC design equations. As seen, using such an approach can lead to either significantly underestimated or overestimated FRS demands for buildings with different heights (modal periods).

The evaluation presented in this section using instrumented building responses, consistent with previous numerical studies, reveals significant shortcomings associated with the ASCE 7-16 $F_p$ equation. This evaluation shows a conceptual difference between the shape and magnitude of the FRS for MRF and SW systems, as well as for short-period and long-period buildings. These observations illustrate that the FRS ordinates strongly depend on the type of the lateral-load resisting system and modal periods of the supporting building, which are neglected in the current ASCE 7-16 equivalent static equation. This observation suggests that one may need to consider using the dynamic analysis methods provided in Section 13.3.1.4 of the ASCE 7-16, which explicitly incorporate the characteristics of the building.

2.13 Conclusions

In this chapter, floor response spectrum (FRS) results of a total of 118 instrumented building-directions in California are evaluated. The selected buildings encompass a wide range of supporting building characteristics (e.g., lateral-load resisting system and modal periods), and
recorded ground motion characteristics (e.g., intensity levels and frequency contents). The primary objective of this evaluation is to identify and quantify the most important parameters that can significantly influence the magnitude of nonstructural components (NSCs) acceleration demands but are not explicitly considered in the simplified ASCE 7-16 $F_p$ equivalent static equation (Eq. 13.3-1) and are also difficult to capture using numerical models. Additionally, an objective is to evaluate and validate observations from numerical studies included in the literature regarding shortcomings associated with the ASCE 7-16 $F_p$ equation. Significant skepticism has been always present, due to the adoption of simplified two-dimensional (2D) buildings models in previous studies, in regard to whether these models represent the characteristics of actual buildings. Hence, the validity of some observations in these numerical studies has been always questioned.

The most salient conclusions of the present study are summarized below:

1) The conducted evaluation on the floor spectra of instrumented buildings reveals significant behaviors that are not consistent with the results obtained in the past using equivalent simplified 2D numerical building models. In many studied instrumented buildings, the shape and magnitude of the floor spectra significantly depart from those obtained based on numerical models of a given type of a regular lateral-load resisting system. The primary reasons for these inconsistencies are the in-plane flexibility of the floor system diaphragm, torsional responses of the supporting building, and the supporting building’s vertical mass or stiffness irregularity:

1-1) The in-plane diaphragm flexibility and torsional responses of the supporting buildings can significantly alter the shape and magnitude of the floor spectra with respect to those obtained from equivalent numerical building models that incorporate rigid diaphragms and symmetry in strength and stiffness of the lateral-load resisting system elements. In buildings with perimeter lateral-load resisting elements, the in-plane diaphragm flexibility can amplify peak floor acceleration (PFA) and peak component acceleration (PCA) demands at the floor mid-span with respect to values at the floor edges. For buildings with a core lateral-load resisting system a reverse trend may be observed. This amplification in single-story buildings with plywood diaphragms can be as large as 5.0, whereas for the studied multistory buildings it is bounded to 2.0. Torsional responses of the supporting building, even in nominally regular buildings, can increase the floor acceleration responses as well as acceleration demands on NSCs that are located in the floor periphery. This amplification is bounded to 1.53 for the studied instrumented buildings.
1-2) Although torsional amplification is highlighted more in torsionally irregular structures, for several buildings that are nominally symmetric in plan and layout of seismic force resisting systems, torsional responses are identified. Torsional effects in the torsionally regular buildings could be attributed to a variety of sources such as local yielding or the asymmetric yielding of the building, accidental torsional moments caused by eccentricities between the centers of rigidity and mass that exist because of uncertainties in the distribution of the mass and stiffness of the buildings, as well as the torsional components of earthquake ground motions. Results of the conducted evaluation illustrate that in most cases, amplification due to the torsional effects and in-plane diaphragm flexibility do not occur simultaneously. In other words, the in-plane diaphragm flexibility can mitigate torsional responses at the expense of amplified responses at the middle/edge of floors.

1-3) Relatively low or large normalized spectral acceleration responses, $FSa/PGA$, are observed at the vicinity of modal periods of several instrumented buildings that are not consistent with the trend observed in the responses of typical numerical building models and other studied instrumented buildings. In some of these cases, the magnitude of $FSa/PGA$ at tuning situation is larger or smaller than the corresponding mean value of $FSa/PGA$ of all instrumented building-directions by factors larger than 10. The relatively low normalized spectral ordinates at tuning situations are due to the low energy content of the recorded ground motion at those specific NSC periods. The significantly large normalized responses generally occur in instrumented buildings that experienced low-intensity ground motions with a consequent elastic or near elastic behavior. However, it is observed that additional plan or vertical mass and stiffness irregularities might have further amplified these large normalized responses. For example, in some of these instrumented buildings large normalized PCA responses are observed at top floor levels with a significantly smaller mass (e.g., by a factor of 10) than the typical floors mass. These large responses are most likely because of tuning of the top story to the modal periods of the rest of the building (the building without this story).

2) It is postulated in this paper that the buildings with significant in-plane diaphragm flexibility, significant torsional responses, or the mentioned special behaviors should not be used to establish the basis for NSCs design equations. The design equations – as they apply to new buildings – should be based on the responses of regular, modern code-compliant building designs. Then,
correction factors could be incorporated into the basic design equations to account for the effects of the mentioned inconsistencies.

3) The roof floor motions obtained from regular instrumented buildings that are not greatly influenced by the abovementioned causes are evaluated to identify and quantify the effect of the supporting building lateral-load resisting system and modal periods on the FRS. This evaluation shows a conceptual difference between the shape and magnitude of the FRS for shear-dominated systems (e.g., MRFs) and flexural-dominated systems (e.g., tall SWs), as well as for short-period and long-period buildings. These observations illustrate that the FRS ordinates strongly depend on the type of lateral-load resisting system and modal periods of the supporting building, which are neglected in the current ASCE 7-16 equivalent static approach. This observation suggests that one may need to consider using the dynamic analysis methods provided in Section 13.3.1.4 of the ASCE 7-16, which explicitly incorporate the characteristics of the supporting building.

4) In buildings with below-grade stories (basements) the seismic base location influences the estimation of acceleration demands on NSCs provided by the ASCE 7-16 Eq. 13.3-1. The acceleration responses of two parallel ground motion sensors, one installed at the ground floor level and the other one at the foundation level, in buildings with below-grade stories can be used to evaluate the seismic base location. An evaluation of responses of several instrumented buildings reveals significant amplifications in ground floor acceleration responses with respect to the floor levels below ground (in some cases amplification factors up to 2.0 are observed even with the presence of perimeter basement concrete walls). This observation implies that the conditions to establish the seismic base at the ground level are not satisfied. For buildings with basements, a soil-structure-interaction analysis incorporating the exact characteristics of the adjacent soil is required to estimate the location of the seismic base. In the absence of such analyses, if doubt exists as where to locate the seismic base, strong consideration should be given to establishing the seismic base at the foundation level when using the ASCE 7-16 Eq. 13.3-1.

This study provides significant insight into influential parameters that should be considered in designing NSCs. Further studies are needed to develop alternative forms for the ASCE7-16 $F_p$ design equation. In a study that is in progress by the authors using data from instrumented buildings and simulation results from several code-compliant building models, the effect of ground motion intensity level (or inelasticity of the supporting building), and the effect of NSCs
characteristics (i.e., period, damping ratio other than the prevalent 0.05 and inelasticity) on NSCs design forces is being investigated.

2.14 Acknowledgments

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2.15 Appendix

Table A1. Characteristics of instrumented buildings and ground motions selected for this study (Information on these buildings and floor motions data were downloaded from the Center for Engineering Strong Motion Data (CESMD) http://strongmotioncenter.org/)

<table>
<thead>
<tr>
<th>#</th>
<th>CESMD Station #</th>
<th>Building Name</th>
<th># of stories</th>
<th>Seismic-force resisting system</th>
<th>Event</th>
<th>1st mode period (s)</th>
<th>N-S</th>
<th>E-W</th>
<th>Recorded PGA (g) N-S</th>
<th>E-W</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>23495</td>
<td>Redlands 1-story Warehouse</td>
<td>1</td>
<td>Concrete tilt-up SW and plywood diaphragm (with wood joints)</td>
<td>B+</td>
<td>0.35</td>
<td>0.60</td>
<td>0.49</td>
<td>0.13</td>
<td>0.17</td>
</tr>
<tr>
<td>2</td>
<td>23495</td>
<td>Redlands 1-story Warehouse</td>
<td>1</td>
<td>Concrete tilt-up SW and plywood diaphragm</td>
<td>Li</td>
<td>0.29</td>
<td>0.39</td>
<td>0.49</td>
<td>0.12</td>
<td>0.11</td>
</tr>
<tr>
<td>3</td>
<td>57187</td>
<td>San Ramon 1-story Warehouse</td>
<td>1</td>
<td>Metal diaphragm on precast concrete SW</td>
<td>Li</td>
<td>0.65</td>
<td>0.65</td>
<td>0.60</td>
<td>0.28</td>
<td>0.08</td>
</tr>
<tr>
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<td>58235</td>
<td>Saratoga 1-story Gymnasium</td>
<td>1</td>
<td>Isolated perimeter RC-SW and sheathing plywood over steel trusses as diaphragm</td>
<td>LP</td>
<td>0.32</td>
<td>0.30</td>
<td>0.69</td>
<td>0.24</td>
<td>0.35</td>
</tr>
<tr>
<td>5</td>
<td>57187</td>
<td>San Ramon 1-story Warehouse</td>
<td>1</td>
<td>Metal diaphragm on precast concrete SW</td>
<td>Li</td>
<td>0.63</td>
<td>0.63</td>
<td>0.60</td>
<td>0.15</td>
<td>0.06</td>
</tr>
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<td>Saratoga 1-story Gymnasium</td>
<td>1</td>
<td>Isolated perimeter RC-SW and sheathing plywood over steel trusses as diaphragm</td>
<td>MH</td>
<td>0.25</td>
<td>0.26</td>
<td>0.69</td>
<td>0.10</td>
<td>0.04</td>
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<tr>
<td>7</td>
<td>36531</td>
<td>1-story School Bldg. (South Wing)</td>
<td>1</td>
<td>Wood frame in both dir., a 12' long SW in south side; tropical composite wood diaphragm</td>
<td>PF</td>
<td>0.48</td>
<td>0.86</td>
<td>0.69</td>
<td>0.29</td>
<td>0.23</td>
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<td>8</td>
<td>1699</td>
<td>1-story Hospital (North Wing)</td>
<td>1</td>
<td>Composite floor diaphragm &amp; RC block wall for north wing</td>
<td>C</td>
<td>0.31</td>
<td>0.34</td>
<td>0.40</td>
<td>0.10</td>
<td>0.12</td>
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<td>9</td>
<td>1699</td>
<td>1-story Hospital (South Wing)</td>
<td>1</td>
<td>Composite floor diaphragm &amp; tube braces in both dir. for southwest wing</td>
<td>C</td>
<td>0.46</td>
<td>0.34</td>
<td>0.40</td>
<td>0.10</td>
<td>0.12</td>
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<td>10</td>
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<td>Fortuna 1-story Supermarket Bldg.</td>
<td>1</td>
<td>Plywood roof diaphragm and block masonry SW</td>
<td>F</td>
<td>0.26</td>
<td>0.34</td>
<td>0.61</td>
<td>0.14</td>
<td>0.14</td>
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<td>Plywood roof diaphragm and block masonry SW</td>
<td>P</td>
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<td>0.42</td>
<td>0.61</td>
<td>0.14</td>
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<td>Plywood roof diaphragm and block masonry SW</td>
<td>PA</td>
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<td>0.43</td>
<td>0.61</td>
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<td>0.61</td>
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<td>Story</td>
<td>Building</td>
<td>Type</td>
<td>Location</td>
<td>Material</td>
<td>Year</td>
<td>Age</td>
<td>Damage Factor</td>
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<td>14</td>
<td>Hemet 1-story Library</td>
<td>1</td>
<td>Masonry block and RCSW at various locations along perimeter &amp; plywood sheathing over steel joists diaphragm</td>
<td>PS</td>
<td>1986</td>
<td>0.21</td>
<td>0.20</td>
<td>0.62</td>
<td>0.10</td>
<td>0.11</td>
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<td>15</td>
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<td>1</td>
<td>Masonry block and RCSW at various locations along perimeter &amp; plywood sheathing over steel joists diaphragm</td>
<td>H</td>
<td>2014</td>
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<td>0.17</td>
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<td>1</td>
<td>Steel chas. -braced frames in both dir. with 3.25&quot; concrete roof diaphragm</td>
<td>ML</td>
<td>2016</td>
<td>0.14</td>
<td>0.13</td>
<td>0.43</td>
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<td>Oakland 2-story Office Bldg.</td>
<td>2</td>
<td>RC block SW</td>
<td>LP</td>
<td>1989</td>
<td>0.35</td>
<td>0.45</td>
<td>0.47</td>
<td>0.20</td>
<td>0.26</td>
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<td>3</td>
<td>Steel MRF</td>
<td>LP</td>
<td>1989</td>
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<td>0.69</td>
<td>0.52</td>
<td>0.20</td>
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<td>3-story UCSB Office Bldg.</td>
<td>3</td>
<td>RCSW</td>
<td>SB</td>
<td>1978</td>
<td>0.35</td>
<td>0.43</td>
<td>0.80</td>
<td>0.38</td>
<td>0.28</td>
</tr>
<tr>
<td>20</td>
<td>LA 3-story commercial Bldg.</td>
<td>3/2</td>
<td>Lower/upper stories: RCSW/steel bracing frame</td>
<td>N</td>
<td>1994</td>
<td>0.55</td>
<td>0.53</td>
<td>0.59</td>
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<td>4</td>
<td>Steel MRF</td>
<td>LP</td>
<td>1989</td>
<td>0.67</td>
<td>0.83</td>
<td>0.62</td>
<td>0.14</td>
<td>0.16</td>
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<td>4</td>
<td>RCSW</td>
<td>LP</td>
<td>1989</td>
<td>0.24</td>
<td>0.35</td>
<td>0.53</td>
<td>0.27</td>
<td>0.36</td>
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<td>23</td>
<td>4-story Hospital</td>
<td>4/1</td>
<td>Steel MRF</td>
<td>PS</td>
<td>1986</td>
<td>0.71</td>
<td>0.63</td>
<td>0.50</td>
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<td>4-story Office Bldg.</td>
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<td>Isolated exterior RC SW</td>
<td>SB</td>
<td>1978</td>
<td>0.50</td>
<td>0.60</td>
<td>0.77</td>
<td>0.12</td>
<td>0.23</td>
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<td>Eureka 4-story Hospital</td>
<td>4/1</td>
<td>RCSW</td>
<td>F</td>
<td>2010</td>
<td>0.35</td>
<td>0.30</td>
<td>0.85</td>
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<td>0.18</td>
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<td>LA 6-story Office</td>
<td>5/1</td>
<td>Chevron steel braced frame</td>
<td>N</td>
<td>1994</td>
<td>0.86</td>
<td>0.90</td>
<td>0.56</td>
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<td>5/1</td>
<td>RC MRF and basement SW</td>
<td>N</td>
<td>1994</td>
<td>1.61</td>
<td>1.45</td>
<td>0.61</td>
<td>0.25</td>
<td>0.19</td>
</tr>
<tr>
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<td>LA 5-story Warehouse</td>
<td>5/1</td>
<td>RC MRF and basement SW</td>
<td>W</td>
<td>1987</td>
<td>1.40</td>
<td>1.30</td>
<td>0.61</td>
<td>0.13</td>
<td>0.17</td>
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<td>Steel MRF</td>
<td>N</td>
<td>1994</td>
<td>1.32</td>
<td>1.38</td>
<td>0.62</td>
<td>0.21</td>
<td>0.36</td>
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<tr>
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<td>6</td>
<td>Steel MRF</td>
<td>W</td>
<td>1993</td>
<td>1.32</td>
<td>1.30</td>
<td>0.62</td>
<td>0.22</td>
<td>0.17</td>
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<td>6</td>
<td>RCSW</td>
<td>N</td>
<td>1994</td>
<td>0.60</td>
<td>0.45</td>
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<td>0.29</td>
<td>0.15</td>
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<td>Imperial County Service Bldg.</td>
<td>6</td>
<td>RCSW and MRF in different dir.</td>
<td>I</td>
<td>1979</td>
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<td>0.41</td>
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<td>Sylmar 6-story County Hospital</td>
<td>6</td>
<td>Lower/upper stories: RCSW</td>
<td>N</td>
<td>1994</td>
<td>0.40</td>
<td>0.35</td>
<td>0.80</td>
<td>0.80</td>
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<tr>
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<td>LA 3-story UCLA Match Bldg.</td>
<td>7</td>
<td>Lower/upper stories: RCSW</td>
<td>N</td>
<td>1994</td>
<td>0.85</td>
<td>1.11</td>
<td>0.60</td>
<td>0.28</td>
<td>0.22</td>
</tr>
<tr>
<td>35</td>
<td>Van Nuys 7-story Hotel</td>
<td>7</td>
<td>RC column-spanned beam frame</td>
<td>N</td>
<td>1994</td>
<td>2.15</td>
<td>2.40</td>
<td>1.35</td>
<td>2.10</td>
<td>0.58</td>
</tr>
<tr>
<td>36</td>
<td>Van Nuys 3-story Hotel</td>
<td>7</td>
<td>RC column-spanned beam frame</td>
<td>W</td>
<td>1987</td>
<td>1.20</td>
<td>1.40</td>
<td>0.58</td>
<td>0.16</td>
<td>0.14</td>
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<tr>
<td>37</td>
<td>LA 8-story CSULA Admin Bldg.</td>
<td>8/1</td>
<td>RC SW expect for 1st level which is columns</td>
<td>N</td>
<td>1994</td>
<td>1.62</td>
<td>1.54</td>
<td>0.70</td>
<td>0.11</td>
<td>0.16</td>
</tr>
<tr>
<td>38</td>
<td>LA 8-story CSULA Admin Bldg.</td>
<td>8/1</td>
<td>RC SW expect for 1st level which is columns</td>
<td>W</td>
<td>1987</td>
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<td>1.10</td>
<td>0.70</td>
<td>0.29</td>
<td>0.39</td>
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<tr>
<td>39</td>
<td>San Bruno 9-story Government Office</td>
<td>9</td>
<td>RC SW</td>
<td>LP</td>
<td>1989</td>
<td>1.30</td>
<td>1.20</td>
<td>0.66</td>
<td>0.16</td>
<td>0.11</td>
</tr>
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<td>40</td>
<td>Pasadena 9-story Commercial Bldg.</td>
<td>9/1</td>
<td>RC MRF</td>
<td>N</td>
<td>1994</td>
<td>1.20</td>
<td>1.23</td>
<td>0.77</td>
<td>0.18</td>
<td>0.15</td>
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<td>41</td>
<td>Pasadena 9-story Commercial Bldg.</td>
<td>9/1</td>
<td>RC MRF</td>
<td>SM</td>
<td>1991</td>
<td>1.20</td>
<td>2.00</td>
<td>0.77</td>
<td>0.23</td>
<td>0.11</td>
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<td>42</td>
<td>LA 9-story Office Bldg.</td>
<td>9/1</td>
<td>Concrete frame with 13&quot; infill masonry walls on the perimeter</td>
<td>N</td>
<td>1994</td>
<td>1.52</td>
<td>1.32</td>
<td>0.62</td>
<td>0.18</td>
<td>0.13</td>
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<td>43</td>
<td>Burbank 10-story Residential Bldg.</td>
<td>10</td>
<td>RC SW</td>
<td>N</td>
<td>1994</td>
<td>0.56</td>
<td>0.60</td>
<td>0.62</td>
<td>0.34</td>
<td>0.26</td>
</tr>
<tr>
<td>44</td>
<td>Burbank 10-story Residential Bldg.</td>
<td>10</td>
<td>RC SW</td>
<td>W</td>
<td>1987</td>
<td>0.51</td>
<td>0.57</td>
<td>0.62</td>
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<td>0.21</td>
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<td>45</td>
<td>Pasadena 12-story Office Bldg.</td>
<td>12</td>
<td>Steel MRF</td>
<td>N</td>
<td>1994</td>
<td>1.00</td>
<td>2.50</td>
<td>0.61</td>
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<tr>
<td>46</td>
<td>Sherman Oaks 13-story Commercial Bldg.</td>
<td>13/2</td>
<td>RC MRF</td>
<td>E</td>
<td>2014</td>
<td>1.50</td>
<td>1.50</td>
<td>0.58</td>
<td>0.24</td>
<td>0.11</td>
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<tr>
<td>47</td>
<td>Sherman Oaks 13-story Commercial Bldg.</td>
<td>13/2</td>
<td>RC MRF</td>
<td>N</td>
<td>1994</td>
<td>2.04</td>
<td>3.12</td>
<td>0.58</td>
<td>0.45</td>
<td>0.21</td>
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<tr>
<td>48</td>
<td>LA 13-story Office Bldg.</td>
<td>13/1</td>
<td>Composite steel-concrete frame with infill unreinforced masonry walls on the perimeter</td>
<td>N</td>
<td>1994</td>
<td>2.25</td>
<td>2.55</td>
<td>0.64</td>
<td>0.18</td>
<td>0.17</td>
</tr>
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<td>49</td>
<td>LA 13-story Office Bldg.</td>
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<td>Composite steel-concrete frame with infill unreinforced masonry walls on the perimeter</td>
<td>SM</td>
<td>1991</td>
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<td>1.95</td>
<td>0.64</td>
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<td>0.13</td>
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<tr>
<td>50</td>
<td>LA 14-story Hollywood Storage</td>
<td>14/2</td>
<td>RC MRF and RC SW in different dir.</td>
<td>N</td>
<td>1994</td>
<td>2.40</td>
<td>2.50</td>
<td>0.59</td>
<td>0.28</td>
<td>0.21</td>
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<tr>
<td>51</td>
<td>Sherman Oaks 14-story Commercial Bldg.</td>
<td>14</td>
<td>RC SW</td>
<td>E</td>
<td>2014</td>
<td>1.18</td>
<td>1.47</td>
<td>0.59</td>
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<td>Code</td>
<td>Description</td>
<td>Floor(s)</td>
<td>Type</td>
<td>Design PGA</td>
<td>Notes</td>
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<tr>
<td>52</td>
<td>24569</td>
<td>LA 15-story Govt Office Bldg.</td>
<td>15/2</td>
<td>Steel MRF</td>
<td>3.22 3.13</td>
<td>0.66 0.21 0.14</td>
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<td>24601</td>
<td>LA 17-story Residential Bldg.</td>
<td>17</td>
<td>RC SW</td>
<td>1.06 1.16</td>
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<td>54</td>
<td>58480</td>
<td>SF 18-story Commercial Bldg.</td>
<td>18/1</td>
<td>RC SW (1-3); Steel MRF (4-18)</td>
<td>2.27 3.12</td>
<td>0.40 0.16 0.13</td>
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<tr>
<td>55</td>
<td>24643</td>
<td>LA 19-story Office Bldg.</td>
<td>19/4</td>
<td>Steel MRF and X-bracing in different dir.</td>
<td>3.44 3.85</td>
<td>0.60 0.20 0.32</td>
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<tr>
<td>56</td>
<td>24464</td>
<td>North Hollywood 20-story Hotel</td>
<td>20/1</td>
<td>RC MRF</td>
<td>2.64 2.50</td>
<td>0.67 0.32 0.11</td>
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<tr>
<td>57</td>
<td>58483</td>
<td>Oakland 24-story Residential Bldg.</td>
<td>24</td>
<td>RC SW</td>
<td>2.50 2.25</td>
<td>0.47 0.18 0.14</td>
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<tr>
<td>58</td>
<td>58532</td>
<td>SF 47-story Office Bldg.</td>
<td>47/2</td>
<td>Steel MRF in both dir. &amp; eccentrically braced in only the transverse dir.</td>
<td>5.26 6.30</td>
<td>0.40 0.11 0.16</td>
<td></td>
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<tr>
<td>59</td>
<td>24602</td>
<td>LA 52-story Office Bldg.</td>
<td>52/5</td>
<td>X-braced steel frames at the core with outrigger MRF in both dir.</td>
<td>5.88 6.25</td>
<td>0.63 0.15 0.11</td>
<td></td>
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<td></td>
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</tr>
</tbody>
</table>

Notes:

a. 0.40S0s (calculated using “USGS tool” https://earthquake.usgs.gov/designmaps/us/application.php) corresponds to the design PGA for the building site and is presented for comparing a recorded ground motion intensity with its corresponding design value.

b. The following nomenclature is used for different earthquake events:
   B: Big Bear; La: Lander; Li: Livermore; LP: Loma Prieta; MH: Morgan Hill; PF: Parkfield; C: Calexico; F: Femdale; P: Petroleum; PA: Petroleum Aftershock; B: Bayview; PS: Palm Springs; H: Hemet; ML: Mammoth Lakes; SB: Santa Barbara; N: Northridge; W: Whittier; I: Imperial; SM: Sierra Madre; E: Encino; SM: Sierra Madre.

c. Numbers in “italic” are fundamental periods roughly estimated based on the spikes observed in roof floor response spectra; “bolded” numbers are fundamental periods calculated using system identification method performed by Bernal et al. (2015); other numbers are fundamental periods based on Goel and Chopra (1997);

d. m/n implies “m” stories above and “n” stories below the ground level. For this particular building, the existence of a significant difference between floor motions at the lowermost basement and the basement level implies that the seismic base could be established at the lowermost basement; in other words, this building essentially behaves like a five-story structure (estimating the real base level for a building with a basement is challenging; in this document when it is not possible to investigate this issue because of the lack of enough sensors at different levels, the seismic base is, conservatively, assumed to be at the foundation level).

e. This building is highly irregular in height (i.e., multiple setbacks along the height is observed).

f. Because of the presence of thick shear walls between the 1st and 3rd floor levels, floor motions at the ground and at the 3rd floors are nearly close, and hence, a better estimate of the seismic base location could be the 3rd floor (in other words, this building seems to be an equivalent five-story structure).

g. This building was significantly damaged during the 1994 Northridge earthquake.

h. Estimating the 1st mode periods based on floor spectra alone is challenging.

List of buildings that are used in the evaluation of the seismic base location (with the order shown in Figure 2-13a):

2.16 References


Chapter 3

Uncertainties in Using the Spectrum Matching Technique for Generating Synthetic Ground Motions
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Chapter 3

Uncertainties in Using the Spectrum Matching Technique for Generating Synthetic Ground Motions


3.1 Abstract

Twenty historical earthquake records are selected as seed motions to be matched to single-damping target spectra using the widely applied wavelet adjustment approach in the time domain. The viscous damping ratio associated with the target spectrum is referred to as the target damping, $\xi_T$. The spectral matching is conducted assuming five different $\xi_T$, resulting in five spectrum-compatible (SC) record sets and an overall of 100 SC records. It is demonstrated that matching to a single-damping target spectrum cannot guarantee an acceptable match to the target spectrum at damping ratios other than, especially smaller than, $\xi_T$. Elastic and inelastic buildings are exposed to different sets of the SC records. Results reveal that dispersion in the fundamental-mode dominated responses under a set of SC records tightly matched to the same target spectrum can be significant if $\xi_T$ deviates from the structure viscous damping at the fundamental mode. This observation is particularly important given that the current ASCE 7-16 provisions require applying 2.5% viscous damping ratio to the fundamental mode of a structure when conducting nonlinear response history analyses, whereas design spectra are based on $\xi_T = 5\%$. For higher-mode dominated responses, irrespective of the system damping, the dispersion increases with increasing $\xi_T$. These observations illustrate the potential need for using a relatively large number of SC records in response history analyses.

Keywords: Spectrum compatible ground motion; target damping; nonlinear response history analysis; Spectral misfit; ASCE 7-16 amplitude-scaling approach; Record-to-record variability.
3.2 Introduction

Response history analyses are usually conducted using a suite of historical ground motions. A main disadvantage of using historical ground motions is the lack of strong motion records that can properly reflect the seismological and geological conditions of a site. The necessity to select a suite of ground motion records instead of a single record, which can increase the required time and effort to analyze a system, to account for the record-to-record variability is another challenge associated with this method. A useful alternative is to generate a single or a handful of acceleration time history records that incorporate the average damage potential of a suite of ground motions. This approach can decrease the time and effort involved in conducting response history analyses and is also believed to mitigate the record-to-record variability. However, results from several research studies show the presence of relatively large dispersions in structural responses under a set of spectrum-compatible (SC) records that are well-matched to the same target spectrum (e.g., see Reyes et al. 2014; Jiang et al. 2015). In these cases, it would be better to use a larger number of SC records to quantify mean structural responses for seismic design or evaluation. However, this approach can partially contradict the fundamental goal of using spectral matching in response history analyses (i.e., using a single or only a handful of records). The present study intends to identify and quantify the primary reasons behind the presence of significant dispersions in structural responses when using a set of SC records, which are generated through tightly matching their response spectra to the same target spectrum using the widely adopted wavelet adjustment technique. Potential solutions for mitigating this shortcoming are also investigated.

Spectral matching is usually conducted on the basis of a single- or multiple-damping target spectrum. The single-damping approach can trace its root in the method proposed by Kaul in 1978 (Kaul 1978). Then, several researchers (e.g., Lilhanand and Tseng 1987, 1988) attempted to extend the Kaul’s algorithm to generate a time history record that matches a design spectrum at multiple damping levels simultaneously. Except for a few works, e.g., in the optimization process of a seismically isolated bridge by Ozbulut and Hurlebaus (2011), the multiple-damping matching approach is seldom applied to the design and analysis of non-nuclear facilities. In many research works and practical applications, a single-damping target spectrum is implemented to generate SC records. Even in routine structural engineering software packages such as the recent versions of ETABS (Computers and Structures Inc. 2015), the option of using a single-damping target
spectrum is available, which can readily provide users with a simulated record based on a given target spectrum. A 0.05 viscous damping ratio is generally assumed for the analysis and design of typical structures as well as the development of ground motion prediction models. Hence, in many previous studies, a 0.05-damped design spectrum has been used as the target. However, as the discussion in this chapter illustrates, when the spectral matching technique is used to generate input for a response history analysis, the value of the damping ratio for the target spectrum, referred to as the target damping, $\xi_T$, is critical in the quantification of seismic responses.

Herein, 20 reference ground motions are selected from the FEMA P695 (2009) far-field record set as seed motions. These ground motions are individually matched to a single-damping target spectrum at five different $\xi_T$ of 0.01, 0.02, 0.05, 0.15 and 0.30, resulting in five SC record sets and a total of 100 SC records. The single-damping spectrum matching is conducted using the wavelet adjustment approach presented by Al Atik and Abrahamson (2010). For each individual SC record, acceleration response spectra at 30 different damping ratios ranging from 0.01 to 0.30, which is believed to cover the wide range of equivalent viscous damping ratios of typical civil engineering structures, are derived. Finally, the spectral misfit is estimated for the 100 SC records. An evaluation of the spectral misfit for different record sets, categorized based on $\xi_T$, reveals the impact of $\xi_T$ on the frequency content of the simulated records. Elastic and inelastic example buildings are exposed to the SC record sets to understand and quantify the variability in different seismic responses, i.e., fundamental- and higher-mode dominated responses.

### 3.3 Single-Damping Spectral Matching Assuming Different Target Damping Ratios

The spectral matching is conducted based on five different $\xi_T$ of 0.01, 0.02, 0.05, 0.15 and 0.30. The target spectrum with a $\xi_T$ of 0.05 is a risk-based 0.05-damped design spectrum for a region of high seismicity (US Zip Code 94107) assuming a Site Class C and a long-period transition of 8.0 s, as per ASCE 7-16 (2016). Target spectra corresponding to other $\xi_T$ values are generated through adjusting the 0.05-damped design spectrum using the damping modification factors provided in ASCE 41-13 (2014). Based on matching the 20 reference records to the target spectrum at the five selected $\xi_T$, five different SC sets and a total of 100 SC records are obtained. Figures 3-1(a), (b) and (c) illustrate response spectra of the simulated record sets for three representative $\xi_T$ of 0.02,
0.05 and 0.30, respectively. Spectra for $\xi_T$ of 0.01 and 0.15 are not shown for brevity. The arbitrary spectra of 1.1 and 0.9 times the Target Spectrum shown in these figures allow for a relative estimate of the tightness and dispersion of response spectra among different SC sets. The trend observed from Figures 3-1(a) to (c) suggests that the spectral matching conducted based on a target spectrum with a higher $\xi_T$ provides an apparent tighter fit with smaller dispersions. However, as discussed next, a high target damping causes a fictitious frequency content matching, especially in the high-frequency region.

![Figure 3-1](image)

**Figure 3-1  Ground response spectra of the SC record sets adjusted to the target spectrum with a target damping ratio of: (a) 0.02, (b) 0.05, (c) 0.30.**

To clarify the effect of $\xi_T$ on the frequency content of simulated records, an example is presented in detail. A ground motion from the 1994 Northridge earthquake is selected as the seed motion. Three $\xi_T$ of 0.02, 0.05 and 0.30 are examined. Spectral matching is conducted three times, and each time the seed motion is matched to the single-damping target spectrum adjusted for the $\xi_T$ of interest. This process results in three different simulated records. Response spectra corresponding to various damping ratios for the records generated through matching to the target spectrum with a $\xi_T$ of 0.02 are computed and shown in Figure 3-2a. Similar graphs are presented in Fig. 2b and 2c for $\xi_T$ of 0.05 and 0.30, respectively. As seen, the conducted process results in significantly different response spectra for a constant damping ratio (e.g., compare the 0.02-damped spectrum in Figure 2-2b with those in Figures 2-2b and c). This observation implies that the frequency contents of the three simulated records are substantially different, meaning that the wavelets used for adjusting the spectral ordinates of a given seed motion to a single-damping target spectrum significantly vary with $\xi_T$. These results illustrate that at a low damping ratio spectral matching is controlled by the frequency content of the ground motion, whereas, a high $\xi_T$ can mitigate the amplitude of high-frequency record components over a wide range of frequencies.
providing a fictitious match in which the expected frequency content of the ground motion is not obtained. These observations also suggest that if the spectral matching is performed based on an \( x\% \)-damped target spectrum, there is no guarantee that the \( y\% \)-damped response spectrum of the simulated record (\( x, y \) are arbitrary values) reasonably matches the \( y\% \)-damped target spectrum. The spectral misfit associated with this method is more pronounced in the response spectra of the simulated records with a damping ratio lower than \( \xi_T \). This can be readily observed by an evaluation of the 0.02-damped response spectrum of the simulated record correspond to \( \xi_T = 0.30 \) (i.e., Figure 3-2c).

![Figure 3-2](image)

**Figure 3-2**  Ground response spectra of the SC records generated via matching the North-Mul009 historical record to the single-damped target spectrum corresponding to target damping ratio of (a) 0.02, (b) 0.05, (c) 0.30.

### 3.4 Average Spectral Error for a Spectrum Compatible Record

To quantify the impact of \( \xi_T \) on the frequency content of the SC records, the average spectral misfit is computed for each individual SC record. Herein the purpose is to estimate the average spectral error associated with a SC record, given a target spectrum that is not related to the seismic evaluation of any specific structure. Consider the \( j^{th} \) SC record. For this record, response spectra are calculated assuming different damping ratios ranging from 0.01 to 0.30 with increments of 0.01. First, the average value of the spectral misfits of the response spectrum corresponding to a damping ratio of \( \xi_m \) over a period range defined as \([0:0.05:3.0 \text{ s}]\) is computed using Eq. (3-1a). The assumed period range is believed to cover the modal periods of most common short- to mid-rise buildings. It is recognized that the derived spectral misfits are strongly dependent on the selected period and damping ratio ranges. For example, selecting a short- or long-period range de-emphasizes the spectral misfit at the long- or short-period region of the spectrum, respectively. However, the adopted approach in this chapter represents an average criterion, and a compromise
needs to be accepted if the objective is to provide a SC ground motion that is not meant to be building specific. Given that $\xi_m$ varies in the range $[0.01:0.01:0.30]$, the $e_j^{\xi_m}$ criterion of Eq. (3-1a) provides 30 misfit values for the $j^{th}$ SC record. The average of these 30 error values is calculated by Eq. (3-1b) and denoted as the average spectral misfit associated with the $j^{th}$ SC record. The estimated error can be considered as the average deviation of the response spectra of a SC record from the corresponding target spectra irrespective of any specific structure, modal period or damping ratio. The median and maximum values of the spectral misfits are calculated using equations analogous to Eq. (3-1b). These equations, which are referred to as Eqs. (1c) and (1d), are not shown here for brevity.

$$e_j^{\xi_m} = \frac{1}{N_T} \sum_{k=1}^{N_T} [(S_a^{\xi_m}(T_k))^j - S_a^{\xi_m}(T_k)]/S_a^{\xi_m}(T_k), j = 1:100 \quad (3 - 1a)$$

$$\frac{(e_j)^{\text{avg.}}}{\xi_m} = \frac{1}{N_{\xi}} \sum_{m=1}^{N_{\xi}} e_j^{\xi_m}, \quad m = 1:30 \quad (3 - 1b)$$

where $(S_a^{\xi_m}(T_k))^j_j$ is the ordinate of the $\xi_m$-damped response spectrum of the $j^{th}$ SC record computed at a period $T_k$; $S_a^{\xi_m}(T_k)$ is the target spectral ordinate at the respective damping ratio and period; $N_T$ and $N_{\xi}$ are the number of discretized periods and damping ratios in the ranges of interest, respectively. The spectral misfits of the $\xi_m$-damped response spectrum, computed based on Eq. (3-1a) at six representative $\xi_m$, for the simulated record set corresponding to $\xi_T = 0.02$ are presented in Figure 3-3a. Each point in this figure is the average deviation of the $\xi_m$-damped response spectrum of an individual SC record from its respective target spectrum.
Figure 3-3 Spectral errors associated with different response spectra of the individual SC records, for target damping ratios of (a) 0.02, (b) 0.05, (c) 0.15, (d) 0.30.

Figures 3-3(b) to (d) illustrate similar results for the record sets with $\xi_T = 0.05$, 0.15 and 0.30. As consistently seen in Figures 3-3(a) to (d), the spectral misfits of the response spectra of the simulated records with damping ratio of $\xi_m$ smaller than $\xi_T$ can be significant. For example, in Figure 3-3(b), where $\xi_T$ is 0.05, the spectral misfit of the 0.01-damped response spectrum of a given SC record is significantly larger than the spectral misfit of the 0.30-damped response spectrum of that SC record. In Figure 3-3(d), where $\xi_T$ is 0.30, the spectral misfits of the response spectra at low damping ratios (e.g., 0.01) are drastically large (several spectral errors exceed 0.50).

The results of the statistical analyses conducted on of the spectral misfits across $\xi_m$ based on Eqs. 3-1(b) to (d) are illustrated in Figures 3-4(a) to (c) for individual SC records. According to Figure 3-4(a), the average spectral error associated with different SC records is within 0.06-0.22, whereas, the maximum spectral error, shown in Figures 3-4(c), for the higher target damping ratios (e.g., 0.15 and 0.30) can have values just below 0.70, i.e., up to a factor of three larger. As seen in Figure 3-4(a), the minimum value of the average spectral error is associated with the record ID number 18 (i.e., the SC record generated through matching the HECTOR-HEC090 historical record to the 0.02-damped target spectrum).
3.5 Spectral Error for Inelastic Response Spectra of the SC Records

The spectral matching is conducted through adjusting the pseudo-spectral acceleration of the reference time series to the spectral ordinates of the target spectrum at a given period. Given a damping ratio, the pseudo-spectral acceleration at a specific period, \( T_s \), is the maximum absolute value of the displacement response history of an elastic SDOF oscillator, with a period equal to \( T_s \), times its squared angular frequency. Conceptually, two solutions to the equation of motion of an SDOF oscillator could be different in the time domain but exhibit very similar pseudo-spectral accelerations. Consequently, the acceleration time histories of two SC records with the same response spectral ordinate for a given damping ratio and period are not necessarily identical. This can lead to a significant dispersion in structural responses under a set of SC records, especially in nonlinear response history analyses.

To clarify the abovementioned statement, constant-ductility inelastic response spectra for two representative SC record sets are developed and discussed in this section. A bilinear model with a 3% strain hardening represents the inelastic behavior of the SDOF oscillators. The SDOFs viscous damping ratio is assumed to be 0.02 of the critical damping. Response spectra for the record set with \( \xi_T = 0.02 \) are illustrated for displacement ductility ratios of \( \mu = 1.0 \) (elastic case), 1.5 and 3.0 in Figure 3-5(a), (b) and (c), respectively. Figures 3-6(a) to (c) illustrate similar graphs for the SC record set with \( \xi_T = 0.15 \). An evaluation of Figures 3-5(a) to (c) illustrate that even with a relatively small dispersion in the elastic response spectra of a SC record set (see Figure 3-5a), significant dispersions about the mean values can be present in the inelastic response spectra,
especially in the high-frequency (short period) region. These dispersions increase with increasing the ductility demand.

![Figure 3-5](image)

**Figure 3-5** 0.02-damped constant-ductility response spectra for the SC record set corresponding to a target damping of 0.02 assuming a displacement ductility ratio of (a) 1.0 (elastic case), (b) 1.5, (c) 3.0.

Figures 3-6(a) to (c), where $\xi_T = 0.15$, illustrate that when the viscous damping of SDOF oscillators is smaller than $\xi_T$ associated with a SC record set, dispersion in acceleration response spectra decreases as the oscillators transition from elastic to inelastic behavior (a reverse trend was observed in Figures 3-5a to c). In this case, the viscous damping ratio of elastic oscillators (Figure 3-5a) is significantly smaller than $\xi_T$, and hence the dispersion is significant. When oscillators transition from elastic to inelastic behavior, their equivalent viscous damping ratio increases and approaches $\xi_T$. In the next section, a six-story archetype building is simulated under different SC record sets to investigate the influence of variations in spectral matching on the fundamental- and higher-mode dominated responses when the structure responds elastically or inelastically.

![Figure 3-6](image)

**Figure 3-6** 0.02-damped constant-ductility response spectra for the SC record set corresponding to a target damping of 0.15 assuming a displacement ductility ratio of (a) 1.0 (elastic case), (b) 1.5, (c) 3.0.
3.6 Case Study

In this section, the dynamic response of a code-based designed six-story special moment resisting frame (SMRF) building is simulated under the SC records to investigate the correlations between the dispersion in the structural responses and the target damping ratio. The building is consistent with the buildings designed as part of the ATC-63 project (for details see Anajafi and Medina 2018). The design spectrum used for sizing the structural elements was similar to the 0.05-damped target spectrum used in the present study. The building lateral-load resisting system comprises three-bay moment resisting frames of equal spans at the building perimeter. Simplified two-dimensional elastic and inelastic models of the building are developed. Global P-delta effects are incorporated using a leaning P-Delta column attached with rigid links to the frame. The inelasticity is modeled using localized rotational hinges assigned to the two ends of beams and columns. The viscous damping of the superstructure is approximated using the Rayleigh approach in which 0.02 and 0.03 damping ratios are assigned to the first two modal periods. The first two elastic modal periods of the building are 1.92 and 0.65 s with modal participating mass ratios of 0.83 and 0.11, respectively. Response history analyses are conducted and dispersions in different Engineering Demand Parameters (EDPs) are evaluated.

An assessment of the floor spectra (generated using a cascading approach) for this building provides information to evaluate dispersions for both higher-mode and first-mode dominated responses simultaneously. The 0.03-damped roof spectra for the inelastic building exposed to three representative SC record sets are illustrated in Figures 3-7(a) to (c). The floor response spectrum corresponding to the best-fitted ground motion in this study (i.e., a SC record generated through matching the response spectrum of the HEC090 historical record to the of 0.02-damped target spectrum; record ID number 18 in Figure 3-4a) is also shown in these figures. As seen, with increasing $\xi_T$, the dispersion in floor spectral ordinates, especially in the vicinity of the building second mode, increases. Moreover, with increasing $\xi_T$, the mean floor spectrum increases.
An evaluation of Figures 3-7(a) and (b), where $\xi_T$ is relatively small, illustrates that spectral ordinates associated with this best-fitted SC record in the higher mode region, which dominate the maximum floor spectral ordinates, are nearly the same as the mean floor response spectrum. However, at the building fundamental period, this record underestimates the mean demand by 30% and 49% in Figures 3-7(a) and (b), respectively. These observations illustrate that if higher-mode dominates responses of are interest, a response history analysis that focus on estimating mean responses can be performed only using this single SC record in lieu of using a set of SC records and evaluating the mean response.

The dispersion in different EDPs, including peak roof displacement (PRD), peak floor acceleration (PFA), and peak component acceleration (PCA) are evaluated. Herein, PCA is defined as the maximum value of the floor response spectrum in the period range 0-0.5 s in which most typical nonstructural components are situated. This EDP represents the higher-mode dominated responses. The statistical evaluation of different EDPs for the six-story inelastic building exposed to different record sets is presented in Table 3-1. In this table, the results for the original record set (i.e., unmatched records) scaled based on the ASCE 7-16 amplitude scaling approach are also provided. Both the coefficient of variation (COV) and the Max/Min parameter are quantified. As seen, for all considered EDPs, dispersions and mean values associated with all SC record sets are significantly smaller than the corresponding values associated with the original record set scaled based on the ASCE 7-16 amplitude scaling approach. This issue is more highlighted for the higher-mode dominated response (i.e., PCA). This latter observation illustrates a significant drawback associated with the amplitude scaling approach using a constant scale factor to the entire ground motion record. In other words, when using a set of amplitude scaled historical records, individual

Figure 3-7 Dispersions in the 0.03-damped elastic roof response spectra for the inelastic SMRF building under SC record sets corresponding to a target damping ratio of (a) 0.02- (b) 0.05- and (c) 0.15.
ground motions can significantly exceed the response input of the target spectrum especially at short periods, which can tend to overstate the importance of higher mode responses.

Results show that dispersion in the first-mode dominated response (i.e., PRD) under the SC record set with a $\xi_T$ near the viscous damping of the structure at the fundamental mode (i.e., 0.02 in this example) has the smallest value. The dispersion in higher-mode dominated response (i.e., PCA) tends to decrease as $\xi_T$ decreases and is not dependent on the building higher modal damping ratios. These observations suggest that for the inelastic response history analysis of a building with relatively low viscous damping at fundamental mode, which is usually the case, a SC record set matched to a low-damped target spectrum can result in smaller dispersions in both higher- and first-mode dominated responses.

<table>
<thead>
<tr>
<th>Record set type</th>
<th>PRD</th>
<th>PFA</th>
<th>PCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min (m)</td>
<td>Max /Min</td>
<td>Mean (m)</td>
<td>COV</td>
</tr>
<tr>
<td>SC</td>
<td>0.01</td>
<td>0.25</td>
<td>1.76</td>
</tr>
<tr>
<td>0.02</td>
<td>0.24</td>
<td>1.80</td>
<td>0.33</td>
</tr>
<tr>
<td>0.05</td>
<td>0.29</td>
<td>1.86</td>
<td>0.40</td>
</tr>
<tr>
<td>0.15</td>
<td>0.28</td>
<td>2.23</td>
<td>0.42</td>
</tr>
<tr>
<td>Scaled</td>
<td>0.18</td>
<td>5.39</td>
<td>0.41</td>
</tr>
</tbody>
</table>

1) The system damping ratio at the 1st-mode is 0.02; 2) Original records scaled based on ASCE 7 with a constant factor of 2.04.

The statistical evaluation of different EDPs for the six-story structure assuming a linear behavior is also shown in Table 3-2. The overall trend observed in the structural responses under different SC record sets is consistent with results of the inelastic building. Table 2 illustrates that for the elastic system, unless the $\xi_T$ is significantly higher than the structural damping, the mean value of the first-mode dominated responses (e.g., PRD) is nearly constant for all SC record sets. An evaluation of Tables 3-1 and 3-2 illustrates that for a given record set, mean PRD demands of the nonlinear system are smaller than those of the linear system, which is consistent with expectations. The same statement is valid for the PFA and PCA responses. The COV of the inelastic system’s PRD response under all SC record sets, except for the set with $\xi_T = 0.15$, is larger than that of the linear system, which is consistent with the trends observed in Figures 3-5(a) to (c). The dispersion in the PFA under all record sets for the inelastic system is smaller than the dispersion.
for the linear system. The dispersion in the PCA responses for the inelastic system is in some SC record sets greater than and in some other sets smaller than that for the linear system.

Table 3-2  Statistical evaluation of various EDPs for the six-story elastic bldg. under different record sets.

<table>
<thead>
<tr>
<th>Record set type</th>
<th>PRD</th>
<th>PFA</th>
<th>PCA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min (m)</td>
<td>Max /Min</td>
<td>Mean (m)</td>
</tr>
<tr>
<td>II</td>
<td>0.01</td>
<td>0.41</td>
<td>1.18</td>
</tr>
<tr>
<td>SC</td>
<td>0.021</td>
<td>0.40</td>
<td>1.12</td>
</tr>
<tr>
<td>Scaled</td>
<td>0.05</td>
<td>0.39</td>
<td>1.54</td>
</tr>
<tr>
<td>0.15</td>
<td>0.24</td>
<td>3.66</td>
<td>0.59</td>
</tr>
</tbody>
</table>

1) The system damping ratio at the 1st-mode is 0.02; 2) Original records scaled based on ASCE 7 with a constant factor of 2.04.

3.7 Conclusions

In this chapter challenges associated with the application of the single-damping spectral matching technique based on the wavelet adjustment approach for generating synthetic ground motion records are addressed. The influence of the damping ratio of the target spectrum (referred to as the target damping ratio, $\xi_T$) on the characteristics of spectrum-compatible (SC) records is investigated. 20 reference historical records are closely matched to a target spectrum at five different $\xi_T$ of 0.01, 0.02, 0.05, 0.15, and 0.30, resulting in five different simulated record sets for a total of 100 SC records. The spectral misfit of the response spectra over a period range of 0.05 to 3.0 s is estimated for each simulated record at multiple damping ratios varying from 0.01 to 0.30. Spectral misfits are statistically evaluated demonstrating that matching a ground motion to a single-damping target spectrum does not guarantee a reasonable match to target spectra at other damping ratios, especially at those smaller than $\xi_T$. As $\xi_T$ increases, the spectral misfit of a SC record at the $\xi_T$ of interest reduces significantly; however, the spectral misfit at damping ratios smaller than $\xi_T$ drastically increases. This observation illustrates a fictitious spectral matching at higher $\xi_T$ values. An archetype building that behaves in some cases elastically and in others inelastically is simulated under different SC record sets. Results reveal a record-to-record variability in the structural responses under all SC record sets, especially in terms of higher-mode dominated responses. This dispersion depends on $\xi_T$, implying a damping-to-damping variability.
The most salient conclusions obtained from simulating the archetype building, based on the assumptions adopted in this chapter, are summarized below:

1. For elastic structures, unless the $\xi_T$ is significantly higher than the structural damping at the first mode, the mean value of the first-mode dominated responses (e.g., roof displacement) is nearly the same for all SC record sets. This implies that in these cases, evaluating the mean responses using a suite of SC records, regardless of the value of the $\xi_T$, is warranted. However, using a suite of records may partially contradict a fundamental goal of applying spectral matching, which is to generate a single or only a handful of ground motions to be used in the analysis. For inelastic structures, unlike the linear ones, the mean value of the first-mode dominated responses increases with deviating $\xi_T$ from the building viscous damping ratio at first mode. This observation is particularly important given that the current ASCE 7-16 provisions require applying 2.5% viscous damping ratio to the fundamental mode of a structure when conducting nonlinear response history analyses, whereas design spectra are based on $\xi_T =0.05$.

2. In terms of the higher-mode dominated responses, irrespective of the structural damping and elastic or inelastic behaviors, the SC records with a low $\xi_T$ (e.g., 0.02) can reasonably represent the frequency content of the target spectrum. The dispersion in the higher-mode dominated responses for a SC record set increases with the value of $\xi_T$. In addition, in this case, the mean value of higher-mode dominated responses is sensitive to the value of $\xi_T$.

3. If the spectral matching is conducted based on a low-damped target spectrum (e.g., 0.02-damped) to reliably estimate mean responses, using a handful of records may be sufficient, whereas, if $\xi_T$ is larger, a larger number of SC records would be required. If one is only interested in first-mode dominate responses of a building with a first-mode close to the $\xi_T$, a handful of SC would be sufficient.

These observations illustrate that using the spectral matching technique to estimate mean responses may not significantly decrease the number of required records, and consequently, the time involved for response history analyses. However, when using a set of historical records that are amplitude scaled, individual spectra can significantly exceed the target spectrum especially at short periods, which can tend to overstate the importance of higher mode responses.
3.8 References


Computers and Structures Inc. 2015. ETABS 2015 integrated building design software, version 15.2.0, Computers and Structures, Inc., Walnut Creek, CA.


Chapter 4

Evaluation of ASCE 7-16 Equations for Designing Acceleration-sensitive Nonstructural Components Using Data from Instrumented Buildings
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Chapter 4

Evaluation of ASCE 7-16 equations for designing acceleration-sensitive nonstructural components using data from instrumented buildings


4.1 Abstract

This study uses instrumented buildings in the US and models of code-based designed buildings to validate the results of previous studies that highlighted the need to revise the ASCE 7-16 $F_p$ equation for designing nonstructural components (NSCs) through utilizing simplified linear and nonlinear building models. The evaluation of floor response spectra of instrumented buildings illustrates that, unlike the ASCE 7-16 approach, the amplification of peak component acceleration (PCA) with respect to the peak floor acceleration (PFA), is a function of the ratio of NSC period to the supporting building modal periods, the ground motion intensity and the NSC location. Increasing the ground motion intensity, or building nonlinearity, in most cases can reduce the PFA/PGA (i.e., in-structure amplification factor) and PCA/PGA responses. The conducted evaluation illustrates that the recorded ground motions at the base of the instrumented buildings in most cases are significantly lower than design earthquake (DE) ground motions. As a result, most of these buildings might have responded elastically. Because ASCE 7-16 is meant to provide demands at a DE level, for a more reliable evaluation of the $F_p$ equation, two representative archetype buildings are designed based on the ASCE 7-16 seismic provisions and exposed to ground motions with intensity levels varying from 0.25 DE to 2.0 DE. This intensity range can represent the intensities consistent with the ones experienced by instrumented buildings as well as larger intensities. Simulation results of archetype buildings, consistent with previous numerical
studies, illustrate the tendency of the ASCE 7-16 in-structure amplification factor, \([1 + 2(z/h)]\), to significantly overestimate demands at all floor levels and the ASCE 7-16 limit of \(a_p = 2^{\frac{1}{2}}\) to in many cases underestimate the calculated component amplification factors. Furthermore, the product of these two amplification factors, which that represents PCA/PGA, in some cases exceeds the ASCE 7-16 equation by a factor up to 1.50.

**Keywords:** Instrumented buildings; Acceleration-sensitive nonstructural components; ASCE 7-16 design equations; Floor response spectra; Archetype buildings; Spectrum compatible ground motion.

### 4.2 Introduction

Current *equivalent static equations* provided in ASCE 7-16 (2016) and adopted by the International Building Code (2016) for estimating seismic design force of acceleration-sensitive nonstructural components (NSCs), supports, and attachments are based on the 2003 NEHRP Provisions (2003). These equations, which can trace their roots primarily in the works conducted in the 1990’s (e.g., Bachman and Drake 1995; Drake and Bachman 1996), are supported primarily by past experience and engineering judgment rather than by experimental and numerical studies. Since the 1990’s, these equations have been gradually modified, however, their general forms have remained the same. A comprehensive literature review on the history and development of various equations that have been used for seismic design of NSCs was provided in Fathali and Lizundia (2011). In accordance with the ASCE 7-16 Equations 13.3-1 to 13.3-3, the horizontal seismic design force \((F_p)\) applied at the component’s center of gravity, can be calculated based on a simplified *equivalent static force* given by Equation (4.1a), subject to a lower limit and upper limit illustrated in Equation (4.1b)

\[
F_p = 0.4S_{DS}W_p[1 + 2(z/h)](a_p/R_p) I_p \tag{4.1a}
\]

\[
0.3S_{DS} I_p W_p \leq F_p \leq 1.6S_{DS} I_p W_p \tag{4.1b}
\]

where \(S_{DS}\) is the short-period 0.05-damped, pseudo-spectral acceleration for the building site; the term \(0.4S_{DS}\) essentially represents the peak ground acceleration (PGA) at the base of the supporting building at the design earthquake (DE), denoted as the design PGA in this study. \(W_p\) is the NSC operating weight. The term \([1 + 2(z/h)]\) is the amplification of the peak horizontal
acceleration at an upper-floor (i.e., peak floor acceleration, PFA) with respect to the design PGA. This term, which is PFA/PGA, is referred to as the an “in-structure amplification factor”. The ratio $a_p/R_p$ is essentially the amplification of peak component acceleration (PCA) with respect to the PFA. This term, which is PCA/PFA, is denoted as “component amplification factor”. The coefficient $R_p$ is the NSC response modification factor that varies from 1 to 12 for different type of NSCs (see Chapter 4 for a detailed discussion of $R_p$). $a_p$ is the amplification factor for an elastic NSC $= 1$ for rigid NSCs that are rigidly attached to the floor; $= 2\frac{1}{2}$ for all flexible or flexibly mounted NSCs; and $= 1\frac{1}{4}$ for the fasteners of the connecting system in exterior nonstructural wall elements and connections; as per Section 11.2 of ASCE 7-16, the period 0.06 s is defined as the threshold for differentiating rigid and flexible NSCs. The remaining parameters in Equations (4.1a) and (4.1b) are as follow: $I_p$ is the NSC importance factor that is either 1.0 to 1.5; $z$ is the height in the structure of point of attachment of NSC with respect to the base; and $h$ is the average roof height of structure with respect to the base. $F_p$ is not required to be taken as greater than $1.6S_{DS}W_PI_p$ and shall not be taken as less than $0.3S_{DS}W_PI_p$.

Three alternative dynamic analysis methods are also provided in ASCE 7-16 for calculating $F_p$. For instance, one approach requires a dynamic analysis to determine floor accelerations; a second approach involves the implementation of nonlinear response history procedures of Chapters 16 to 18 in ASCE 7-16, and a third approach uses the floor response spectrum (FRS) method. Because the vast majority of seismic design efforts in practice are based on the equivalent static method, the dynamic analysis approaches are not discussed herein (the term ASCE 7 equation in this paper implies the equivalent static equation).

Based on Equation (4.1), the design PCA (i.e., $F_p/W_p$) normalized to the design PGA and its lower/upper limits, assuming an elastic NSC ($R_p = 1$) with an importance factor of $I_p = 1.0$, can be written in the form given by Equations (4.2a) and (4.2b)

\[
\frac{PCA}{PGA} = \frac{F_p/W_p}{0.4S_{DS}} = a_p [1 + 2(z/h)] \quad (4.2a)
\]

\[
0.75 \leq \frac{PCA}{PGA} \leq 4.0 \quad (4.2b)
\]
In Equation (4.2a), the parameter $\alpha_p$ is essentially the ratio of PCA to the peak floor acceleration (i.e., PCA/PFA) for an elastic NSC. The lower and upper limits for PCA/PFA are obtained to be 0.75 and 4.0, respectively.

As seen in Equation (4.2a), the design PCA/PFA is simply a function of the component’s flexibility (i.e., tuning condition) and its vertical location within the supporting building. Over the years, numerous studies have been prompted to improve the understanding of seismic demands on NSCs. A wide variety of these research works can be found in the literature: form several studies that have focused on single-degree-of-freedom (SDOF) buildings (e.g., Lin and Mahin 1985; Chen and Soong 1989; Igusa 1990; Zhu et al. 1994; Sullivan et al. 2013) to works that have studied multistory three-dimensional building models (e.g., Wieser et al. 2013; Jiang et al. 2015). In this regard, several research studies have reported potential shortcomings associated with the ASCE 7 equivalent static equation used for estimating seismic forces on NSCs, supports, and attachments. Nowadays, it is well understood that these design-oriented methods do not account for all the factors that significantly affect the response of NSCs (Villaverde 1997). The parameters that are not explicitly incorporated into the design equations, but are influential, include the effect of supporting building characteristics (i.e., modal periods, ductility demand, and diaphragm flexibility), and NSC damping ratio, which is assumed to be 5% in the current seismic provisions. There are also significant shortcomings in the parameters that are taken into account in the design equation: the lack of dependency of the component amplification factor on the NSC location along the building height, the assumption of a linear distribution of the in-structure amplification factor over the building height regardless of building system and strength, and the somewhat arbitrary definition of 0.06 s as the period threshold to separate rigid and flexible NSCs. In the following paragraphs, examples of studies that have highlighted shortcomings associated with both the in-structure and the component amplification factors given by the ASCE 7 equation are briefly presented.

Miranda and Taghavi (2005), through presenting a closed-form solution to the equation of motion for a continuous linear-elastic shear–flexural beam, illustrated that the type of lateral-load resisting system, the fundamental period and damping ratio of the supporting building have a significant influence on the PFA responses (i.e., in-structure amplification). Later in 2006, Taghavi and Miranda using the same linear-elastic models evaluated the two components of the
ASCE 7 equation in structures with different fundamental periods. They showed that (i) unlike the adopted approach in ASCE 7, the component amplification factor is a function of the supporting building fundamental period and the NSC vertical location (i.e., relative height of point of attachment of NSC to the structure); (ii) the value of component amplification factor for NSCs tuned to different modal periods of the supporting building is different; (iii) the computed component amplification factor in many cases is significantly larger than the ASCE 7 limit of $2\frac{1}{2}$; (iv) in many cases the computed in-structure amplification factor (i.e., PFA/PGA) is smaller than the ASCE 7 limit of $(1 + 2z/h)$. Medina et al. (2006) evaluated NSCs acceleration demands in generic one-bay two-dimensional moment frames with different number of stories, fundamental periods and ductility demands. They illustrated that current seismic code provisions do not always provide an adequate characterization of PCA responses, especially when the period of the NSC is near one of the modal periods of the supporting structure. Later in 2007, Sankaranarayanan and Medina (2007) using models consistent with those of Medina et al. (2006) investigated the main factors that influence floor spectral accelerations in the generic moment frames. They illustrated that the acceleration demands on NSCs depend not only on the vertical location of a NSC in the supporting structure and the period of the NSC but also on damping ratio of the NSC, the supporting building modal periods, and the level of inelasticity of the supporting building. In another study, Taghavi and Miranda (2012) using the models consistent with those used in Medina et al. (2006), showed that PFA responses are heavily dependent on the floor relative height, and the supporting building nonlinearity, fundamental period and lateral-load resisting system.

These research works, while providing valuable insight into understanding the influential parameters on NSCs seismic demands, in many cases cannot adequately represent important characteristics present in the response of actual buildings. Some of the previously mentioned works (e.g., Miranda and Taghavi 2005) are based on adopting the linear-elastic behavior assumption for the supporting building. This assumption seems adequate for the design of essential facilities such as emergency centers and nuclear power plants, which are typically designed to remain elastic or nearly elastic during severe earthquake ground motions. However, this assumption is not directly applicable to most nonessential buildings that are designed to undergo inelastic deformations during the DE even when the presence of overstrength is accounted for. In this context, several numerical studies have highlighted the significant influence of supporting building nonlinearity on
NSCs seismic acceleration demands (see e.g., Sewell et al. 1986; Igusa 1990; Adam and Fotiu 2000; Rodriguez et al. 2002; Medina et al. 2006; Politopoulos and Feau 2007; Sankaranarayanan and Medina 2007; Adam and Furtmüller 2008; Chaudhuri and Villaverde 2008; Wieser et al. 2013; Petrone et al. 2015; Vukobratović and Fajfar 2015; Petrone et al. 2016).

Many other research works, which have considered the supporting building nonlinearity, have been mostly based on SDOF or generic building models that do not adequately represent relevant characteristics present in the response of actual multistory buildings. Only a few studies have evaluated the ASCE 7 equations using code-compliant buildings. For example, in 2013, Wieser et al. (2013) studied four different special steel moment resisting frame (SMRF) structures including the post-Northridge SAC three-, nine-, and 20-story office buildings, and one three-story hospital building designed based on the ASCE 7 seismic provisions. They showed that when the supporting building is in the elastic range, the ASCE 7 in-structure amplification factor could overestimate floor acceleration responses by a factor up to 3.0; for a building ductility of 2.5, this factor could be as large as 4.0. Wieser et al. also illustrated that a constant NSC amplification factor could not capture the variation of NSCs acceleration responses located through the height of the studied buildings. In 2016, Petrone et al. evaluated equations evaluated in different seismic codes and standards including those of ASCE 7 for designing NSCs. They conducted the evaluations based on the responses of a set of five benchmark reinforced concrete frame buildings with different heights varying from 1 to 10 stories. These buildings were designed according to the Eurocode 8 provisions. Petrone et al. illustrated that at the DE level the ASCE 7 in-structure amplification factor overestimates PFA/PGA responses in many cases (e.g., by a factor of 1.50-1.75 at the roof of the studied structures), whereas the computed component amplification factors are in several cases significantly larger than the ASCE 7 limit of 2.1/2 (e.g., by a factor of 1.30-1.80 at the roof of the studied structures). Petrone et al. also showed that at the roof level, the product of the two amplification factors, which represent the normalized PCA response, could exceed the code ASCE 7 limit of 4.0 by a factor of 2.10-2.40 for the analyzed structural models.

In this chapter, floor response spectra of a total of 118 instrumented building-directions in the US are studied to evaluate the adequacy of the current ASCE 7-16 equivalent static equation for designing acceleration-sensitive NSCs. An evaluation of instrumented buildings facilitates the understanding and quantification of underlying phenomena and characteristics that can
significantly affect the NSCs acceleration responses but are difficult to capture using simplified numerical models. These relevant phenomena and characteristics include, but are not limited to, the in-plane flexibility of the floor diaphragm system; torsional responses of the primary building; vertical mass and stiffness irregularities; the real distribution of seismic damage; the contribution of infill and partition walls to the building modal periods; soil-foundation-structure interaction; the equivalent elastic viscous damping of the supporting structure; as well as potential interactions between heavy NSCs and their supporting building; among others.

In the past years, a few studies have used instrumented buildings responses to evaluate the ASCE 7 $F_p$. In 1998, Naeim et al. compared the imposed seismic demands and the extent of acceleration-sensitive NSCs damages in six different instrumented buildings with force demands estimated by several seismic design codes and guidelines including ASCE 7. They showed that the experienced seismic force demands on NSCs in several cases exceeds the design force levels recommended by various design provisions. Fathali and Lizundia (2011) evaluated the ASCE 7 equation based on the responses of a large number of instrumented buildings and illustrated that in many cases the in-structure and component amplification factors exceeds the ASCE 7 upper limits. Conducting statistical analyses, Fathali and Lizundia suggested new equations for the in-structure and component amplification factors. Wang et al. (2014) compared seismic demands on acceleration-sensitive NCSs in two instrumented multistory reinforced concrete buildings with the ASCE 7 equation. They concluded that the empirical linear distribution of $[1 + 2(z/h)]$ in the ASCE 7 provisions, in most cases, results in an overestimation of the in-structure amplification factor, in particular when the supporting building response is strongly nonlinear. They also illustrated that the constant value of $a_p = 2^{\frac{1}{2}}$ is not always capable of capturing the dynamic amplification effects of NSCs. Qu et al. (2014), selecting a large number of instrumented buildings, concluded that the measured in-structure amplification factor in many cases significantly exceeds the ASCE 7-16 limit of $[1 + 2(z/h)]$. This latter study is in contradiction with the conclusion drawn by Wang et al. (2014).

Except for the study conducted in Wang et al. (2014), which was based on responses of only two buildings, the other aforementioned studies that used instrumented buildings, evaluated the ASCE 7 equations based on the normalized acceleration demands (i.e., PFA/PGA and PCA/PGA), without explicitly considering the effect of the ground motion intensity level (or the supporting
building nonlinearity). The present study illustrates that an adequate evaluation of the ASCE 7-16 equation should account for the effect of the ground motion intensity level. It is shown herein that the recorded PGA at the base of the instrumented buildings in most cases was significantly less than the design PGA, and only in few cases these buildings experienced a ground motion intensity near the DE. Thus, the authors hypothesize that the instrumented buildings in the US primarily remained in their linear-elastic range. Since ASCE 7-16 is meant to provide seismic demands at the DE level, an adequate evaluation of the ASCE 7-16 equations cannot be performed solely based on the responses of these instrumented buildings and/or linear-elastic models. To further investigate the effect of the supporting building nonlinearity on the evaluation of the ASCE 7-16 equation, two representative archetype buildings (i.e., a six-story moment resisting frame and an eight-story reinforced concrete shear wall) are designed based on the ASCE 7-16 seismic provisions. The archetype buildings are exposed to ground motions scaled to different intensity levels (i.e., smaller than, equal to, and greater than the DE), and the adequacy of the ASCE 7-16 equivalent static equation is discussed. The effects of other characteristics of instrumented buildings (e.g., lateral-load resisting system, modal periods, diaphragm flexibility, and torsional responses) on NSCs acceleration demands are not part of this study and were evaluated comprehensively in Anajafi and Medina (2018).

4.3 Instrumented buildings and main assumptions for generating floor response spectra

For the study conducted in this chapter, instrumented buildings are selected from the CESMD database (Center for Engineering Strong Motion Data www.strongmotioncenter.org). All multistory steel and concrete instrumented buildings that have recorded earthquake ground motions with a PGA greater than 0.15 g at least in one principal horizontal direction are selected. The lower limit applied on PGA is believed to represent a reasonable threshold of seismic damages, and can render a large enough sample of multistory instrumented buildings. Since this study aims to evaluate the ASCE 7-16 design equations that are used for common buildings in the US, multistory buildings equipped with control systems (e.g., base-isolation and dampers) and masonry building configurations, which can bias the results, are excluded from this evaluation. For single-story buildings, adopting these criteria renders only three individual buildings; therefore, in this
case the PGA threshold is changed to 0.10 g to provide a bigger sample size. Furthermore, for the single-story buildings, masonry configurations, which are common for single-story building in the US, are also included. In the entire CESMD database 44 individual buildings (nine single-story and 35 multistory) satisfy the mentioned criteria. For some buildings, recording motions are available for more than one event that satisfy the mentioned PGA thresholds. Hence, the compiled database has an overall size of 59 building-earthquake samples. Given that the buildings dynamic characteristics and the recorded ground motions in two orthogonal horizontal directions are different, a database with a total size of 118 building-directions is compiled. A comprehensive description of the selected instrumented buildings in terms of material, lateral-load resisting system, number of stories, fundamental periods, PGA of the records, etc. is presented in Anajafi and Medina (2018). All floor acceleration motions (approximately 600 motions) available for these buildings in the CESMD database are used in this study.

Consistent with the prevalent procedure adopted in building codes for designing the primary building, and also consistent with many previous studies, floor response spectrum (FRS) in this study is based on a %5 damping and linear NSC assumption. The objective herein is not to determine the most realistic value of NSC damping ratio but to use a value that will permit the direct evaluation of the ASCE 7-16 equations using the same basic assumptions. Elastic 0.05-damped floor spectra are developed for all existing floor motions over a component period, $T_{\text{comp.}}$, range varying from 0 to at least 2.0 times each building-direction fundamental period, $T_{\text{1bdg.}}$. For a given building floor, pseudo-spectral acceleration response at $T_{\text{comp.}} = 0$ corresponds to the PFA; the maximum value of the pseudo-spectral accelerations over the period range of interest is selected as the peak component acceleration (PCA) response. As an example, Figure 4-1(a) illustrates FRS for different floor levels of the Sylmar six-story County Hospital (CSMIP Station No. 24514) in the North-South (N-S) direction exposed to a ground motion from the 1994 Northridge Earthquake (it should be noted that in this case the recoded PGA was almost equal to the design PGA at the building site). Figure 4-1(b) shows normalized PCA and PFA responses versus the relative height, $(z/h)$, along with the ASCE 7-16 equations.
As seen in Figure 4-1(a), a significant amplification occurs in upper-floor spectral accelerations with respect to those at the ground level, especially near the modal periods of the supporting building (i.e., the tuning situations); for example, the value of the floor pseudo-spectral acceleration (FSa) normalized to the recorded PGA at the ground level (i.e., the first floor) in the vicinity of the building fundamental mode (i.e., $T_{\text{bldg.}} = 0.39$ s) is 2.61, whereas the corresponding response at the roof level is 7.53. For other NSC periods away from the tuning condition, this amplification is not significant; for example, at $T_{\text{comp.}} = 1.0$ s the amplification factor, $\text{FSa/PGA}$, at the roof level is only 1.12, which is near the corresponding amplification factor at the ground level, i.e., 1.05. These observations, consistent with fundamentals of structural dynamics and also findings included in previous numerical studies (e.g., Medina et al. 2006), highlight the importance of the ratio between the NSC period and the supporting building period for the $i$-th mode, $T_{\text{comp.}}/T_{\text{bldg.}}$, and also the NSC vertical location in estimating acceleration demands on NSCs. As seen in Figure 4-1(b), for this example the normalized PCA responses at upper floor levels exceed the ASCE 7-16 equation, however, for the normalized PFA responses the ASCE 7-16 equation consistently overestimates the demands at all floor levels.

### 4.4 Evaluation of ASCE 7-16 design equations using floor spectra obtained from instrumented buildings

In several previous research works, the evaluation of the ASCE 7 $F_p$ equation using the responses of instrumented buildings or numerical building models was conducted based on a comparison between the measured (computed) PCA/PGA and the $F_p$ equation in its normalized format, i.e.,
Equation (4.2a). The main purpose of this section is to illustrate the shortcomings associated with this approach, when the supporting building remain in elastic range. Figures 4-2(a) and (b) illustrate 0.05-damped floor spectra for all existing roof acceleration motions normalized to the recorded PGA and to the PFA, respectively. In these figures, the NSC period is also normalized to the fundamental period of the individual building-directions. It is important to note that this section intends to identify the behavioral trends of floor spectra; if the main objective is to conduct a statistical evaluation across floor spectra of diffident instrumented buildings, an improved component period normalization is needed because the way that $T_{\text{comp.}}$ is normalized herein will cause higher-mode spikes of individual floor spectra to occur at different period ratios, and averaging over all floor spectra may de-emphasize these individual spikes.

Figure 4-2(a) illustrates that when FSa responses are normalized to the recorded PGAs, in many individual cases the ASCE 7-16 limit of PCA/PGA = 4.0 is significantly exceeded (see values such as 33.0 and 36.0). Furthermore, a significant dispersion about the mean roof FRS is observed. Figure 4-2(b) shows a similar trend for PCA/PFA (i.e., $a_p$). An evaluation of Figures 4-2(a) and 4-2(b) reveals that PCA/PGA and $a_p$ are a function of $T_{\text{comp.}}/T_{i,\text{bldg.}}$, which is not explicitly incorporated into the current ASCE 7-16 $F_p$ equation. It should be noted that the PCA/PGA and $a_p$ estimated by ASCE 7-16 are, at some degree dependent on the tuning ratio, $T_{\text{comp.}}/T_{i,\text{bldg.}}$. The component period threshold of 0.06 intends to account for the fact that rigid NSCs are less likely tuned to one of the modal periods of the supporting building, and hence $a_p = 1.0$, whereas the flexible NSCs are most likely to be tuned and hence $a_p = 2\frac{1}{2}$. While the lack of direct dependence of prescribed acceleration demands on component tuning ratio illustrates a shortcoming in the ASCE 7-16 equation, the relatively large normalized acceleration ratio responses observed in Figures 4-2(a) and (b) do not necessarily mean that the estimated design values by ASCE 7-16 equation are unconservative. An evaluation of the results shows that most of these large values belong to the buildings that were exposed to relatively small ground motion intensity levels whereas ASCE 7-16 is meant to provide design values at the DE. In other words, most of the instrumented buildings might have remained in their linear-elastic behavior range, and these large values are highlighted because they are in the normalized format (i.e., divided by PGA). This statement is clarified next.
The rest of this section aims to show that the observed large normalized acceleration responses occur if the primary building is linear-elastic or the ground motion intensity level is significantly lower than the design PGA. Figure 4-3 depicts the PGA/PGA_{Design} ratio, where PGA_{Design} = 0.4 S_{DS} for the instrumented building-directions considered in this chapter. This ratio relates the intensity level of the recorded earthquakes at the base of individual building-directions with that of the DE level ground motion at each individual building site. For each building S_{DS} is computed using the “USGS tool” (https://earthquake.usgs.gov/designmaps/us/application.php) utilizing the information provided in the CESMD website. When the soil profile characteristic is not available for an instrumented building, an ASCE 7-16 site classification D is assumed. S_{DS} for individual instrumented buildings was provided in Table A.1 of Anajafi and Medina (2018).

As seen in Figure 4-3, most of the recorded PGAs are significantly smaller than the design PGA. An evaluation of the percentage of the data points smaller than a specific PGA/PGA_{Design} illustrated in Figure 4-4 reveals that, for example, PGA of 85% of the records is less than
0.50 PGA\textsubscript{design} or PGA of 96% of the records is less than 0.75 PGA\textsubscript{design}. Hence, it is reasonable to infer that most of these structures behaved in the linear-elastic range. A more accurate conclusion regarding this issue can be drawn through a comparison of the ground spectrum ordinate of a record at the building fundamental period and the corresponding value from the design spectrum of the building site (see Anajafi and Medina 2018, Section 2.3) This possible elastic behavior could significantly bias any evaluation of the ASCE 7-16 equation based on the responses of the instrumented buildings alone. To clarify this statement, Figure 4-5(a) shows the PCA responses (i.e., maximum FSa over the entire component period range) normalized to the recorded PGA versus the relative height (z/h) for the instrumented building-directions. In this figure, the ASCE 7-16 equation without applying the upper limit of 4.0, denoted as ASCE w/o limit, is also illustrated. As seen, if all normalized PCA response are considered without taking into account the ground motion intensity, at all relative heights, especially at the roof level, the ASCE 7 equation in many cases will be significantly exceeded (i.e., by factors up to 9.0 in several cases). Given that demands are evaluated at the DE level, a more consistent way of assessing the ASCE 7-16 equation can be postulated by normalizing PCA demands to the design PGA (i.e., 0.4S\textsubscript{DS}) at each building site, as illustrated in Figure 4-5(b). As seen, unlike the trend observed in Figure 4-5(a), in this case only a few data points exceed the ASCE 7-16 equation (the maximum exceedance is by a factor of 2.3).

![Figure 4-5](image.png)  
**Figure 4-5** PCA responses normalized to: (a) recorded PGA; (b) design PGA; vs. the relative height for the instrumented building-directions (w/o limit: without applying the upper limit of 4.0)
Figures 4-6(a) and (b) illustrate the percentage of data points at the roof level of the instrumented building-directions that are greater than a specific normalized PCA, when the PCA responses are normalized to the recorded PGA or the design PGA, respectively.

According to Figure 4-6(a), at the roof level 76.1% and 37.3% of the data points exceed the values of 4.0 (i.e., the ASCE 7-16 upper limit) and 7.5 (i.e., the ASCE 7-16 equation without applying the upper limit), respectively. However, in Figure 4-6(b) only 15.9% and 2.0% of the data points exceed these limits illustrating that NSCs mounted on many of the instrumented buildings investigated experienced acceleration demands smaller than the design values estimated by ASCE 7-16.

To investigate the effect of the ground motion intensity level on the evaluation of the ASCE 7-16 equation, normalized PCA responses obtained from instrumented building-directions are shown in Figures 4-7(a) to (c) for three different scenarios: (i) for all 118 building-directions regardless of the recorded PGA (i.e., same as Figure 4-5a), (ii) for the building-directions with \( \frac{PGA}{PGA_{design}} \geq 0.50 \), and (iii) for the building directions with \( \frac{PGA}{PGA_{design}} \geq 0.75 \). As seen, with increasing the ground motion intensity level, the relatively large normalized PCA responses disappear. While in Figure 4-7(a) the maximum value of the PCA/PGA is 36, this quantity in Figure 4-7(c), for buildings with \( \frac{PGA}{PGA_{design}} \geq 0.75 \) is only 7.80.
The in-structure amplification factor (i.e., PFA/PGA) versus the relative height for different ground motion intensity scenarios is illustrated in Figures 4-8(a) to (c). As illustrated, while for low intensity ground motions the in-structure amplification factor can exceed the ASCE 7-16 estimate (e.g., by a factor as large as 1.87 at the roof level shown in Figure 4-8a), for ground motions with higher intensity levels the ASCE 7-16 significantly overestimates the magnitude of most measured in-structure amplification factors (e.g., by a factor varying from 3.0 to 1.5 for different cases at the roof level shown in Figure 4-8c).

An evaluation of Figures 4-7(a) to (c) and Figure 4-8(a) to (c) suggests that increasing the ground motion intensity level has a stronger influence on the PCA/PGA responses than on the PFA/PGA responses. For example, the maximum PCA/PGA responses observed in Figures 4-7(a) and 4-7(c) are 36.0 and 7.5, respectively, meaning a reduction factor of 4.80 in the maximum value of observed PCA/PGA. However, the maximum PFA/PGA responses shown in Figures 4-8(a) to (c), are 5.50 and 2.01, respectively, implying a reduction factor of 2.75. Although this conclusion is based on responses of buildings with different modal periods, lateral-load resisting systems, and experienced ground motion excitations, it is consistent with expectations: PFA responses are mostly affected by the rigid-body translation of the ground but PCA responses (especially if they occur in the vicinity of the fundamental period of the building) are highly influenced by the supporting building inelasticity. Hence, with increasing the ground motion intensity level, and consequently the primary building inelasticity, it is expected that PCA/PGA responses be reduced at a higher rate than PFA/PGA responses.

**Figure 4-7** PCA/PGA vs. the relative height for the instrumented building-directions with a PGA ratio larger than (a) 0.05; (b) 0.5; and (c) 0.75 (w/o limit: without applying the upper limit of 4.0)
Figures 4-9(a) to (c) present the component amplification factor (i.e., PCA/PFA) for different ground motion intensity scenarios. As seen, consistent with the trend observed in Figures 4-7(a) to (c) and 4-8(a) to (c), with increasing the ground motion intensity level the value of the component amplification factor tend to decrease. For example, while in Figure 4-9(a) the maximum observed PCA/PFA at the roof level is 7.7, in Figure 4-9(c), where only instrumented building-directions with PGA/PGA\textsubscript{design} $\geq$ 0.75 are considered, this quantity is limited to 4.5.

However, even for higher ground motion intensity levels, the ASCE 7-16 upper limit of $2\frac{1}{2}$ for the component amplification factor is exceeded at different floor levels (i.e., by a factor as large as 2.0 in several cases).

To further investigate the effect of increasing the ground motion intensity level on the magnitude of FSa responses, two instrumented buildings, for which acceleration motions of several earthquake events were recorded, are studied in more detail. The first example is the Sherman 13-story Commercial Bldg. (CSMIP Station No. 24322) with a reinforced concrete
moment resisting frame system. Ground pseudo-spectral acceleration (GSa) responses for three selected events recorded in this site are illustrated in Figure 4-10(a). Note that this building was strengthened with friction dampers after the 1994 Northridge Earthquake (Goel and Chadwell 2007), hence, the recorded motions after the rehabilitation should not be used combined with the recorded motions before the rehabilitation because the rehabilitated building is a new building with a different dynamic behavior.

As illustrated in Figure 4-10(a), the frequency contents of the three recorded ground motions at the short period-ratio region (e.g., \( T_{\text{comp.}} / T_{\text{bldg.}} < 0.25 \)) are comparable allowing for a relatively fair comparison of the normalized FSa responses of these records in the mentioned period region. Figure 4-10(b) illustrates the roof FSa/PGA responses for different events. As seen, for the smallest PGA value (i.e., 0.04 g that corresponds to a ground motion from the 1992 Landers Earthquake), the maximum value of FSa/PGA in the higher-mode region (i.e., \( T_{\text{comp.}} / T_{\text{bldg.}} < 0.25 \)) is 8.20. With increasing PGA to 0.10 and 0.45 g for the ground motions from the 1987 Whittier and the 1994 Northridge earthquakes, this quantity decreases to 6.50 and 4.80, respectively. With increasing the ground motion intensity (i.e., PGA) from 0.04 to 0.10 and 0.45 g the value of PFA/PGA (i.e., FSa at \( T_{\text{comp.}} = 0 \)) decreases from 2.65 to 2.04 and 1.39, respectively. The observed reductions in the normalized acceleration responses could be attributed to the cracking/nonlinearity in the supporting building due to increasing the ground motion intensity level. The relatively large spike of 13.90 in the vicinity of the fundamental period of the building under the ground motion from the 1992 Landers event is also most likely a consequence of the relatively small ground motion intensity, or in other words, the linear-elastic behavior of the supporting building.

Figure 4-10(c) shows the absolute (non-normalized) roof FSa responses for the 13-story building exposed to the considered ground motions. For the evaluation purposes, the absolute design PCA provided by ASCE 7-16 (i.e., Equation 4.2a multiplied by \( 0.4S_{DS} \) of building site), is also shown. As illustrated, unlike Figure 4-10(b) in which the ASCE 7-16 equation in many NSC periods is exceeded for FSa/PGA responses, absolute FSa responses are consistently below the ASCE 7-16 design VALUES. In terms of the PFA, the ASCE 7-16 estimation in all cases for both normalized and absolute responses is higher than the measured demands.
As another example, a similar analysis is performed for the Burbank 10-story Residential Bldg. (CSMIP Station No. 24385) with a reinforced concrete shear wall system. For this building, floor spectra for four different recorded ground motions are depicted in Figures 4-11(a) to (c). According to the results shown in Figure 4-11(b), the records from the 2014 Encino and 1991 Sierra Madre Earthquakes with relatively low intensity levels, have caused large spikes of $F_{Sa}/PGA = 17.6$ and 18.4 at the short-period (i.e., $T_{comp.}/T_{1bldg.} < 0.25$) and fundamental-period region of the floor spectra (i.e., near $T_{comp.}/T_{1bldg.} = 1.0$), respectively. However, for the records from the 1987 Whittier and 1994 Northridge Earthquakes with higher intensity levels, the magnitude of $F_{Sa}/PGA$ is limited to 9.0. With increasing the ground motion intensity from 0.05 to 0.10, 0.17 and 0.30 g, the normalized PFA response changes from 3.30 to 3.13, 1.82 and 2.57, respectively. As seen, for this example with increasing the PGA, the normalized PFA tends to exhibit a general decreasing trend, although an inconsistency (i.e., an increase) is observed when PGA increases from 0.17 to 0.30 g.

An evaluation of Figure 4-11(b) and (c), consistent with the example provided in Figure 4-10, shows that, compared to a larger ground motion, a smaller ground motion may cause larger normalized acceleration responses but not necessarily larger absolute acceleration responses.
Based on the evaluation presented in this section using the floor spectra of the instrumented buildings, several shortcomings are observed in the ASCE 7-16 $F_p$ equation for designing NSCs. The most salient shortcomings include: the lack of dependency of component amplification factor to the vertical location of NSCs; the linear assumption for the distribution of in-structure amplification factor along the building height; the lack of dependency of in-structure and component amplification factors on the building modal periods and level of inelastic behavior. However, a final conclusion about the adequacy of the ASCE 7-16 upper limits cannot be performed solely based on the responses of instrumented buildings given that they were mostly exposed to relatively small ground motions. In other words, given that at higher ground motion intensity levels typical nonessential buildings are designed to experience nonlinear actions, a linear extrapolation of $FS_a$/PGA ratios from low to higher ground motion intensities is highly questionable. To further investigate the effect of the supporting building nonlinearity on $FS_a$ responses, and to evaluate the adequacy of the ASCE 7-16 equation, in the next section simulation results of two representative archetype buildings designed based on ASCE 7-16 seismic provisions are presented.

### 4.5 Archetype buildings to evaluate ASCE 7-16 equivalent static equation for designing NSCs

In most of previous research works that have investigated the effect of the primary building nonlinearity on $FS_a$ responses, generic frames were used. These structures while helpful to explore the influential parameters on NSCs acceleration demands, in many cases do not adequately
represent important characteristics present in the responses of real buildings. Hence, these models cannot be solely used to evaluate the adequacy of the ASCE 7-16 $F_p$ equation, especially its upper limits. In other words, since ASCE 7-16 is meant to provide design values at the DE level, a more consistent approach is to evaluate the $F_p$ equation using responses obtained from code-based designed buildings exposed to DE level ground motions. In this section, an archetype steel moment resisting frame (SMRF) and an archetype reinforced concrete shear wall (RCSW) are designed based on the ASCE 7 seismic provisions, and their responses are evaluated numerically under ground motions with different intensities varying from 0.25 DE to 2.0 DE. These archetype buildings are consistent with the buildings used as part of the ATC-63 project, which was documented as FEMA P695 (2009).

4.5.1 Criteria and assumptions for analysis and design of archetype buildings

The lateral-load resisting system elements are located at the buildings perimeter in two principal directions. Two-dimensional numerical models of archetype buildings are developed; the global/structure P-delta effects of gravity loads that are not tributary to the lateral-load resisting elements are incorporated via a zero flexural-stiffness leaning P-delta column that is attached to the model with axially rigid pin-ended members. To approximate viscous damping of the structure, the Rayleigh approach is used. In this approach, generally a constant damping ratio is specified to two modal periods $T_i$ and $T_j$. The damping ratios specified to other modal periods in between $T_i$ and $T_j$ remain nearly constant; however, for modal periods larger than this period range the specified damping ratio rapidly increases. In this study, a 0.025 damping ratio is assigned to $2T_{1\text{bldg}}$ and $T_{2\text{bldg}}$. The value of $2T_{1\text{bldg}}$ is selected to limit the viscous damping ratio of the effective fundamental period when the system goes nonlinear, and the effective fundamental period elongates.

A spectrum-compatible ground motion record is used in response history analyses. The target spectrum (with $S_{DS} = 1.0 \text{ g}$ and $S_{D1} = 0.55 \text{ g}$) is similar to the design spectrum used in ATC-63. It is recognized that applying the spectral matching approach cannot completely remove the record-to-record variability in seismic responses. For example, a large dispersion can be observed about the mean FSa response of a set of spectrum-compatible records closely matched to the same target spectrum (Reyes et al. 2014; Jiang et al. 2015; Anajafi and Medina 2018c). Anajafi and
Medina (2018c) illustrated that if spectral matching is conducted at a lower damping ratio (e.g., when the reference time series is matched to the 0.02-damped target spectrum) such variability will decrease significantly. They used 100 spectrum-compatible ground motions, generated through matching 20 reference records to a target spectrum at five different damping ratios varying 1% to 30%, and estimated the spectral misfit (i.e., average deviation of a response spectrum with respect to the design spectrum) of all simulated records in a wide period and damping range. They also calculated floor spectra for several elastic and inelastic building models (including the six-story SMRF building selected for the present study) exposed to the simulated records. Anajafi and Medina showed that the HECTOR-HEC090 record matched to the 0.02-damped target spectrum, which was associated with the minimum spectral misfit, exhibits the minimum deviation from the mean FRS compared to the other simulated records (this record underestimates the mean roof FSa responses of the six-story SMRF building by 30% in the vicinity of the first mode but it in the vicinity of the second mode, which dominates the FRS, it underestimates the mean by 8% only). This record, whose ground response spectra for different damping ratios are shown in Figure 4-12, is used for the response history analyses in this study. The study presented in this chapter is an initial step toward improving design equations for acceleration-sensitive NSCs. Hence, the analyses are conducted using this single spectrum-compatible record and two archetype buildings to understand behavioral patterns. Later in Chapter 5, a set of 20 SC records and a set of 44 far-field records will be used and several archetype buildings will be analyzed for final conclusions regarding the potential drawbacks of the ASCE 7-16 $F_p$ equation.

**Figure 4-12** Ground response spectra for the spectrum-compatible record used in this study

The archetype buildings are simulated under the SC records scaled to different intensity levels varying form 0.25 DE to 2.0 DE with a 0.25 DE increment. Floor spectra are developed based on an uncoupled analysis of the supporting building and NSCs, meaning that the component-structure
dynamic interaction (coupling) is neglected. This assumption is considered to be sufficiently accurate if the NSC mass is smaller than that of the supporting building by a factor of at least 1000 (Adam and Furtmüller 2008; Adam et al. 2013). If this factor is smaller, a FRS is usually conservative (Adam and Furtmüller 2008; Adam et al. 2013). Note that the dynamic coupling criterion is not unique in the literature; for example the mentioned factor in ASCE (2000) is 100).

4.5.2 Simulation results for a six-story steel moment resisting frame archetype building

A nonlinear 2D model for a six-story special moment resisting frame (SMRF) archetype building is developed. The building is loaded and designed according to the ATC-63 building group RSA-Dmax characteristics (designed for a high seismic loading based on the Response Spectrum Analysis approach). The building is rectangular in plan with dimensions of 140 ft. (42.67 m) and 100 ft. (30.48 m) in two principal directions. The lateral-load resisting system is a reduced beam section (RBS) special SMRF comprising three-bay frames of equal spans of 20 ft. (6.10 m) at the building periphery. Localized rotational plastic hinges are assigned to the middle of the RBS segments. Localized moment-axial force hinges are assigned to the end of columns at the base. Nonlinear hinges are defined assuming a bilinear behavior with a 3% strain hardening. Material properties for structural steel sections are based on the ATC-63 with a nominal yielding stress of $F_y = 50$ ksi (345 MPa); the expected yielding stress is assumed to be 1.10 of the nominal. The sizing of columns and beams and their variation along the height is performed using a similar trend adopted in ATC-63 (see Table 4-1). The elastic modal periods of the building, including P-delta effects, at the first two modes are 1.92 s and 0.65 s with modal participating mass ratios of 0.83 and 0.11, respectively.

<table>
<thead>
<tr>
<th>Story No.</th>
<th>Beam Size</th>
<th>Exterior Column Size</th>
<th>Interior Column Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W690×140 *</td>
<td>W610×195</td>
<td>W610×241</td>
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<tr>
<td>2, 3</td>
<td>W760×161</td>
<td>W610×195</td>
<td>W610×241</td>
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<tr>
<td>4</td>
<td>W610×125</td>
<td>W610×195</td>
<td>W610×241</td>
</tr>
<tr>
<td>5, 6</td>
<td>W610×125</td>
<td>W610×195</td>
<td>W610×195</td>
</tr>
</tbody>
</table>

* A6M section size (metric).
Nonlinear response history analyses are performed for all ground motion intensity levels, and floor spectra are calculated for all floor levels of the building. As an example, Figures 4-13(a) and (b) illustrate normalized and absolute (non-normalized) roof FSa responses for some selected intensity levels, respectively. According to Figure 4-13(a), with increasing the intensity level, normalized FSa responses (FSa/PGA) decrease. In other words, with successive yielding of the supporting building (i.e., softening of the lateral-load resisting system) and the subsequent detuning, the amplification in NSC pseudo-spectral acceleration response with respect to the PGA reduces. This reduction rate in the vicinity of the modal periods, especially the first mode, is larger than near other NSC periods. As illustrated in Figure 4-13(a), if an earthquake induces inelastic behavior in this building, the normalized peak acceleration demands do not increase and instead, high acceleration demands start to affect a wider range of NSC periods because the effective period of the supporting building lengthens.

As seen in Figure 4-13(a), for the intensity levels lower than the DE, the ASCE 7-16 equation in its normalized format (i.e., Eq. 4-2a) is significantly exceeded, especially in the vicinity of the second mode; for example, the maximum value of FSa/PGA under the 0.25 DE level ground motion is 7.50 (i.e., it exceed the ASCE 7-16 limit of 4.0 by 88%). However, as previously discussed, using normalized PCA responses corresponding to intensity levels lower than the DE may not lead to a fair evaluation of the ASCE 7-16 $F_p$ equation in its normalized format given than a linear extrapolation from a smaller earthquake to the DE may not be accurate. For the DE level, where the evaluation of the ASCE 7-16 equation is supposed to be performed, the value of FSa/PGA in the vicinity of the building second mode is 5.0 meaning that it exceeds the ASCE 7-16 upper limit of 4.0 by 25% whereas at other NSC periods, including those tuned to the building first mode, the ASCE 7-16 estimation is lower than the computed demands. Figure 4-13(b) shows absolute roof FRS for the selected intensity levels. As illustrated, unlike the trend observed in Figure 4-13(a) for the normalized floor spectral acceleration responses, the higher intensity levels control the absolute FSa responses. For intensity levels lower than the DE, absolute FSa responses in all cases (except for the 0.75 DE level in the vicinity of the building second mode) are below consistently the ASCE 7-16 equation. In other words, if NSCs are designed for the DE level, they will experience smaller absolute acceleration responses under intensity levels smaller than the DE although the normalized PCA responses are larger for smaller intensity levels. These observations
suggest that the adequacy of the ASCE 7-16 $F_p$ equation should not be assessed primarily using normalized floor spectral acceleration responses obtained from exposing buildings to ground motion intensity levels that do not induce inelastic responses. Such elastic responses are, instead, useful for understanding behavioral patterns, and in the cases of instrumented buildings, useful to identify and quantify the effects of 3D behaviors (e.g., in-plane diaphragm flexibility and torsion) on NSC seismic demands (see Chapter 2).

Figure 4-13 5%-damped roof spectra for the six-story SMRF archetype building

Figure 4-14(a) illustrates the normalized PFA responses, and the normalized PCA responses in the vicinity of the first two modes of the buildings versus the ground motion intensity level for the roof floor of the SMRF archetype building. As seen, with increasing the ground motion intensity, the normalized peak acceleration responses (both PFA/PGA and PCA/PGA) consistently decrease. The reduction rate at the start point of the nonlinearity (almost at 0.50 DE) has the highest value (see the curves slopes for different intensity levels in Figure 4-14a). According to the results shown in Figure 4-14(a), with increasing the intensity from 0.25 DE to 1.0 DE, the normalized PFA responses decrease from 1.79 to 1.27. This reduction for the normalized PCA in the vicinity of the second mode is from 7.50 to 4.99, and for the normalized PCA in the vicinity of the first mode is from 5.33 to 2.60. Considering the 0.25 DE level as the linear-elastic case, for the DE level the acceleration response modification factors (i.e., ratio of roof normalized spectral acceleration ordinate obtained from the linear-elastic building to that of an inelastic building) of 1.41, 1.50 and 2.05 are observed for the PFA, the second-mode PCA and the first-mode PCA responses, respectively. As seen, the primary building nonlinearity mitigates PFA responses at a lower rate than PCA responses. This observation, consistent with the results obtained from the instrumented
buildings, suggests that PFA is mostly governed by the rigid-body translation of the ground than
the primary building inelasticity when compared to the PCA responses. An evaluation of the
response modification factor of the first-mode and second-modes PCA reveals that first-mode
dominated responses exhibit levels of inelasticity that are larger than those experienced by the
higher-mode dominated responses, which is consistent with expectations.

Figure 4-14(b) illustrates the absolute (non-normalized) PFA and PCA responses at the roof
level of the six-story SMRF building versus the ground motion intensity level. As seen, for all
ground motion intensity levels, even for those greater than the DE level, the absolute PFA
responses are consistently below the ASCE 7-16 design value of 1.2 g. With increasing the ground
motion intensity, unlike the trend observed in the second-mode absolute PCA responses, the first-
mode absolute PCA and absolute PFA responses increase at a relatively slow rate while
approaching a near-saturation condition.

It is important to note herein that past studies (e.g., Sewell et al. 1986; Sankaranarayanan and
Medina 2007) suggest that the rate of increase observed in the absolute (non-normalized) values
of the PCA responses could be larger when the supporting building exhibits a concentration of
inelasticity along its height, the NSC has a damping ratio smaller than 0.05, and the NSC has a
period in the vicinity of the higher modal periods of the supporting building. Under these
conditions, inelasticity in the supporting building can amplify demands with respect to those
observed for a linear-elastic building when both buildings are exposed to the same ground motion
intensity levels (see Chapter 5 for a detailed discussion).

Figure 4-14 PCA and PFA responses at the roof level vs. the earthquake intensity level for the six-story
SMRF archetype building

4-25
The normalized and absolute (non-normalized) PCA responses versus the relative height for the six-story SMRF building exposed to the spectrum-compatible record scaled to different intensity levels are illustrated in Figures 4-15(a) and (b), respectively. As seen in Figure 4-15(a), at the DE level the ASCE 7-16 design value is exceeded for the mid-height and roof floors by a factor of 1.10 and 1.25, respectively. For the lower intensity levels while the normalized PCA responses exceed the ASCE 7-16 design value (e.g., by a factor of 1.90 at the roof for the 0.25 DE level), the absolute (non-normalized) values are mostly below those obtained from the ASCE 7-16 equation.

![Diagram](image)

*Figure 4-15 PCA responses vs the relative height for the six-story SMRF archetype building*

An evaluation of the in-structure and the component amplification factor is shown in Figures 4-16(a) and (b), respectively. Figure 4-16(a) illustrates that with increasing the ground motion intensity level, the PFA/PGA profile (i.e., the calculated in-structure amplification factor) shifts to the left. The ASCE 7-16 linear distribution for the in-structure amplification consistently overestimates the demands, especially when the supporting building experiences inelastic behavior. At the DE level, the maximum value of the computed PFA/PGA, which occurs at the roof, is equal to 1.25, which is significantly lower than the ASCE 7-16 limit of 3.0. According to the results shown in Figure 4-16(b), for this building, the maximum component amplification factors are generally observed at the mid-height floor level. At this level, with increasing the intensity from 0.25 DE to 2.0 DE, the value of PCA/PFA decreases from 6.0 to 3.5. Despite these reductions, the computed PCA/PFA (i.e., the component amplification factor) at the DE level at all floors above the base exceeds the ASCE 7-16 upper limit of $2^{\frac{1}{2}}$.
Note that the value of the computed PFA/PGA, shown in Figure 4-16(b), at the ground level (i.e., $z/h = 0$) should be theoretically equal to unity. However, when generating spectrum-compatible records matching to the target spectrum at relatively small periods (i.e., near 0 that corresponds to PGA) is a challenging task. As a result, PGA of the simulated record (i.e., ground pseudo-spectral acceleration, $S_a$, at a period of 0) deviates from the target PGA that is $0.4 S_{DS}$. For example, in the 0.05-damped ground response spectrum previously shown in Figure 4-12, PGA of the spectrum-compatible record, i.e., $S_a(T = 0)$, is 0.49 g, which is different than the target PGA that is $0.4 S_{DS} = 0.40$ g. In this study when normalizing PFA and PCA responses, $0.4 S_{DS}$ is used instead of the PGA of the spectrum-compatible record. Hence, at the ground level the mentioned discrepancy has occurred.

4.5.3 Simulation results for an eight-story RCSW archetype building

In the ATC-63 project, the reinforced concrete shear wall (RCSW) archetype buildings were designed for two axial force levels and two seismic design categories. An eight-story building from the group with the lower axial force and the higher seismic design load is selected for this study. The RCSW is modeled in SAP2000 (Computers and Structures Inc. 2014) using the nonlinear shell elements based on steel reinforcing and concrete material characteristics provided in ATC-63 (the expected compressive strength of the unconfined concrete and the expected yielding stress of the reinforcing steel are 6.25 and 68.0 ksi (i.e., 44.8 and 468.8 MPa), respectively). With respect to the ATC-63, the reinforcing steels are modified in some cases to improve the system global ductility. Table 4-2 presents the RCSW archetype building design characteristics. Modal periods
of the first two modes of the un-cracked model, including P-delta effects, are 0.61 s and 0.12 s with modal participating mass ratios of 0.68 and 0.22, respectively.

Table 4-2  Design characteristics of the special RCSW building

<table>
<thead>
<tr>
<th>Story No.</th>
<th>Thickness (m)</th>
<th>Longitudinal Reinf. Web</th>
<th>Boundaries</th>
<th>Confinement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2</td>
<td>0.35</td>
<td>#13*@0.25 m</td>
<td>14 #36</td>
<td>#13@0.15 m</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.35</td>
<td>#13@0.25 m</td>
<td>14 #29</td>
<td>#13@0.15 m</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.35</td>
<td>#13@0.25 m</td>
<td>14 #13</td>
<td>#13@0.20 m</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.35</td>
<td>#13@0.25 m</td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

* Metric rebar size.

Simulation results for the RCSW building, summarized in Figures 4-17 to 4-20, consistently present similar general trends obtained in the previous section from studying floor spectra of the SMRF building. The results show that at the DE level the ASCE 7-16 equation overestimates the PFA responses at all floor levels whereas the PCA responses, especially when the NSCs are tuned to the RCSW second-mode, tend to be underestimated. The results also confirm that the relatively large peak normalized acceleration responses are due to the elastic behavior of the supporting building, and with increasing the ground motion intensity (or the supporting building inelasticity), peak normalized acceleration responses significantly decrease. In the following paragraphs, some of observations that are slightly different than those obtained in the previous section are discussed.

![Image of normalized and absolute values](image)

*Figure 4-17 0.05-damped roof spectra for the eight-story RCSW archetype building*

An evaluation of the 0.05-damped roof spectrum of the RCSW building, illustrated in Figures 4-17(a) and (b), suggests that peak acceleration demands in this building are consistently larger than those of the SMRF building. Considering the modal periods of the two buildings, this observation is consistent with expectations.
The absolute (non-normalized) and normalized peak acceleration responses at the roof floor of the RCSW building versus the ground motion intensity level are illustrated in Figures 4-18(a) and (b), respectively. As seen in Figure 4-18(a), with increasing the ground motion intensity from 0.25 DE to 1.0 DE, normalized PFA response decreases from 3.22 to 1.61. This reduction for the PCA of the second-mode region is from 12.80 to 5.88, and for the PCA of the fundamental-mode region is from 12.58 to 4.05. Considering the 0.25 DE level as the elastic case, for the DE level responses, the acceleration response modification factors of 2.00 and 2.18 and 3.10 are obtained for the PFA, the PCA of the second mode and the PCA of the first mode, respectively. These values are larger than those observed in the SMRF example, however, the overall message regarding the difference between the response modification factors of the PFA and PCA is the same. Figure 4-18(b) illustrates that for the RCSW building, only the PFA response exhibits a near-saturation limit, whereas the PCA responses consistently increase with increasing the ground motion intensity.

![Graph](image)

*Figure 4-18 PCA and PFA responses at the roof level vs. the ground motion intensity level for the eight-story RCSW archetype building*

The normalized and absolute PCA responses versus the relative height for the RCSW exposed to different ground motion intensities are presented in Figures 4-19(a) and (b), respectively. Results illustrate the same overall trends as those observed in the SMRF example. Figure 4-19(a) reveals that at the DE level, the value of PCA/PGA at the roof and mid-height floor of the RCSW exceeds the ASCE 7-16 design value by a factor of 1.47 and 1.40, respectively. As shown in Figure 4-19(b), in this RCSW example, for the intensity levels lower than the DE the absolute PCA responses in the vicinity of the building second-mode exceed the ASCE 7-16 equation in several cases.
Figure 4-19 PCA responses vs. the relative height for the eight-story RCSW archetype building

An evaluation of the PFA/PGA profile (i.e., the computed in-structure amplification factor responses) for the RCSW building at the DE level, shown in Figure 4-20(a), reveals that the ASCE 7-16 equation all floor levels consistently overestimates the demands. For example, at the DE level, the computed PFA/PGA ratio is 1.60, which is smaller than the ASCE 7-16 design value of 3.0. Figure 4-20(b) shows the component amplification factor versus the relative height for different ground motion intensity levels. As seen, at the DE level, the PCA/PFA ratios at all upper-floors exceed the ASCE 7-16 limit of 2.5.

Figure 4-20 Evaluation of the two components of the ASCE 7-16 equation using responses of the eight-story RCSW archetype building

Although results obtained from the evaluation of the SMRF and RCSW buildings in Sections 4.5.2 and 4.5.3 are different in some details, both examples consistently reveal potential shortcomings associate with the ASCE 7-16 $F_p$ equation for designing NSCs. Furthermore, the comparison of the responses of these examples and the responses of the instrumented buildings confirms that the relatively large PFA/PGA and PCA/PFA responses belong to the low intensity ground motions (i.e., when the supporting buildings are in the linear-elastic behavior range). This
statement is described in detail next. Sections 4.5.2 and 4.5.3 illustrate that at the 0.25 DE, which corresponds to the elastic behavior for the studied buildings, the PCA/PGA response for the SMRF and RCSW buildings is limited to 7.50 and 12.80, respectively. For the instrumented buildings, which are generally exposed to low intensity ground motions, significant PCA/PGA responses (e.g., see 33 and 36 in Figure 4-2a) are also observed. In another study, Anajafi and Medina (2018) illustrated that these large normalized acceleration responses occur because of special behaviors such as tuning of an entire floor to the modal periods of the supporting building, vertical building irregularities in mass and stiffness, torsional responses and in-plane diaphragm flexibility effects. These behaviors are not modeled in the simplified two-dimensional numerical models used in this paper. Anajafi and Medina (2018) showed that if these special cases are excluded, PCA/PGA ratios for the rest of the selected instrumented buildings are limited to 12.0 (refer to Figure 2-16 of this dissertation), which complies with the PCA/PGA ratios observed in the studied archetype buildings in the linear-elastic domain. In the next section a regular instrumented building whose responses were not significantly influenced by the mentioned special behaviors is selected to further validate the adequacy of the archetype models in capturing acceleration responses of actual regular buildings with rigid diaphragms.

4.5.4 A brief validation of the archetype buildings

In this section, the Burbank six-story Commercial Bldg. (CSMIP Station No. 24370) is selected to investigate the adequacy of the archetype buildings in capturing the acceleration responses of regular buildings. This building was constructed in 1977 and instrumented in 1980 with 13 accelerograms on four floor levels. The building has a square 120’ × 120’ (36.6 m × 36.6 m) plan at all floors. The story height at the first story is 17’6” (5.33 m) and at upper stories is 13’ (3.96 m). The vertical load carrying system consists of a 3” (0.076 m) concrete slab over a metal deck supported by steel frames. The lateral-load resisting system comprises steel moment frames located at the building perimeter. The foundation system includes concrete caissons approximately 32’ (9.75 m) deep. Figures 4-21(a) and (b) illustrate 0.05-damped floor spectra for different floor levels of the building exposed to a ground motion from the 1994 Northridge Earthquake in the North-South (N-S) and East-West (E-W) directions, respectively. For this structure, PGA/PGA\textsubscript{Design} for the N-S and E-W directions was 0.34 and 0.58, respectively. Hence, the building has most likely remained in the linear-elastic range, especially in the N-S direction.
Figure 4-21 0.05-damped floor spectra for the Burbank six-story building exposed to a ground motion from the 1994 Northridge Earthquake: (a) N-S direction; (b) E-W direction

As seen in Figure 4-21(b), floor spectra captured by the sensors at the two sides of the roof are almost identical. The same behavior is observed for the second floor. These observations imply that the building was not affected by torsional responses. For validation purposes, simplified two-dimensional models of the building in both principal directions are developed based on the six-story SMRF archetype building model developed in Section 4.5.2. The lateral stiffness and modal damping ratios of the SMRF building are adjusted to match the building dynamic characteristics. The results of the evaluation for the N-S direction are presented herein, while similar results are observed for the E-W direction.

Based on the system identifications performed by Goel and Chopra (1997), the first two modal periods and viscous damping ratios of the Burbank six-story building in the N-S direction are $T_{1\text{bldg.}} = 1.36$ s, $T_{2\text{bldg.}} = 0.47$ s, and $\beta_{1\text{bldg.}} = 0.04$ and $\beta_{2\text{bldg.}} = 0.053$. The required modifications are applied to the SMRF archetype building model to achieve the above-mentioned dynamic characteristics. Numerical models with two different assumptions are considered. In the first model, horizontal displacements of all nodes of individual floors are constrained together in the lateral direction. In the second model, no constraint is applied, and the average floor spectra of the floor nodes is derived. Figure 4-22 illustrates 0.05-damped roof spectra obtained from the two numerical models along with the one obtained based on the roof floor acceleration motion available in the CSMIP database (denoted as the actual FRS). In Figure 4-23, the FSa responses of the numerical models are normalized to the actual FSa responses. As seen, the numerical model without the constraint provides a closer match to the actual FRS. This model overestimates the demand by 15% in the vicinity of the second mode; at the first period, which dominates the FRS
in this building, this model underestimates the demand only by 5%. These observations illustrate
that such simple numerical models, which are obtained by adjusting the first two modal periods
and modal damping ratios, can reasonably represent the response of regular instrumented buildings
in the linear-elastic range.

Figure 4-22 0.05-damped roof spectra for Burbank six-story bldg exposed to a record from
the 1994 Northridge (w/: with; w/o: without)

Figure 4-23 Roof spectra of the numerical models normalized to actual roof spectrum for Burbank six-
story bldg exposed to a record from 1994 Northridge

4.6 Summary and conclusions

In this study, the adequacy of the ASCE 7-16 equivalent static \( F_p \) equation for designing
acceleration-sensitive nonstructural components (NSCs) is evaluated. The main purpose of this
study is to use the responses of instrumented buildings and models of code-based designed
buildings to validate the results of previous studies that highlighted the need to revise the ASCE
7-16 \( F_p \) equation when utilizing simplified generic linear and nonlinear numerical models. Floor
motions corresponding to all single-story and multistory instrumented buildings in the US that
have experienced a peak ground acceleration (PGA) greater than 0.10 and 0.15 g at least in one
horizontal direction, respectively, are selected for this evaluation. Since this study aims to evaluate
the ASCE 7-16 \( F_p \) design equation, which is used for common buildings in the US, buildings
equipped with control systems (e.g., base-isolation systems and dampers). Single-story masonry
buildings, which are common in the US, are included in this selection, whereas for the multistory
buildings, masonry configurations are excluded. Adopting these criteria results in 59 building-
earthquake samples (i.e., 118 building-directions) and approximately 600 floor motions to be
evaluated. It is shown that many of the instrumented buildings experienced relatively small ground
motion intensities. For example, for 85% of the cases, the recorded PGA was lower than
0.50 \text{PGA}_{\text{Design}}$, where \text{PGA}_{\text{Design}} is $0.4S_{\text{DS}}$ for a given building site. Hence, it is reasonable to infer that most of these buildings behaved in their linear-elastic range. A consequence of this linear behavior is the presence of relatively large normalized peak floor acceleration (PFA/PGA) and normalized peak component acceleration (PCA/PGA) responses; for example, at the roof level of the studied instrumented buildings, the PFA/PGA and PCA/PGA responses as large as 5.5 and 36.0 are observed, respectively. If these normalized acceleration responses are selected to evaluate the ASCE 7-16 $F_p$ equation in its normalized format (i.e., Equation 4-2a), they can exceed the ASCE 7-16 estimation for PFA/PGA and PCA/PGA by factors up to 1.8 and 9.0, respectively. However, as the discussion presented in the next paragraph shows, these relatively large normalized acceleration responses belong to buildings that most likely respond elastically, and hence, they cannot be used for a direct evaluation of the adequacy of the magnitude of the design values in their normalized format.

The effect of ground motion intensity on floor response spectra of the instrumented buildings is investigated revealing that as the ground motion intensity increases, normalized acceleration responses significantly decrease; for example, when only the instrumented buildings with a \text{PGA}/\text{PGA}_{\text{design}} \geq 0.75 are considered, the PFA/PGA and PCA/PGA responses at the roof level are limited to 2.0 and 7.5, respectively, which are values significantly lower that the previously discussed 5.5 and 36.0 values. This latter observation demonstrates the drawbacks of an evaluation of the ASCE 7-16 $F_p$ equation in the normalized format based on the responses obtained from the linear-elastic models or instrumented buildings that have experienced relatively small recorded ground motions. However, the results of the conducted evaluation on the instrumented buildings responses illustrate significant shortcomings associated with the two components of the ASCE 7-16 equation. It reveals that, unlike the current ASCE 7-16 approach, the component amplification factor (PCA/PFA) is a function of the ratio of NSC period to the modal periods of the supporting building; ground motion intensity level; and the NSC location along the building height. It also illustrates that the ASCE 7-16vperiod threshold used to determine the component amplification factor warrants modification. In addition, the evaluation of the responses of instrumented buildings suggest that with increasing ground motion intensity, the PFA/PGA responses tend to decrease. These observations corroborate results of previous numerical building models available in the literature.
To further investigate the effect of the ground motion intensity level (or the supporting building nonlinearity) on the evaluation of the ASCE 7-16 $F_p$ equation, two representative archetype buildings are designed based on the ASCE 7-16 seismic provisions. Nonlinear response history analyses are conducted on the archetype buildings exposed to various ground motion intensity levels (including ground motions consistent with the ones experienced by instrumented buildings and the design earthquake). Simulation results, consistent with previous studies, illustrate the tendency of the ASCE 7-16 in-structure amplification factor, $[1 + 2(z/h)]$, to significantly overestimate demands at all floor levels and the ASCE 7 limit of $a_p = 2\frac{1}{2}$ to in many cases underestimate the calculated component amplification factor. Furthermore, the product of these two amplification factors (that represents the normalized PCA) in some cases exceeds the ASCE 7-16 equation by a factor up to 1.50.

This study provides much needed information useful for the development of improved design equations for acceleration-sensitive NSCs. In order to develop such equations, archetype buildings with different number of stories and modal periods should be developed. Other influential parameters such as the effect of diaphragm flexibility, NCS damping ratio, and NSC ductility, should be also incorporated into the proposed DFRS. For example, many of the research studies referenced in this chapter have assumed a 5% viscous damping ratio for NSCs. However, some recent experiments show damping ratios lower than 5% for typical NSCs (e.g., Watkins et al. 2009; Astroza et al. 2015). Based on a study by the authors, the effect of NSC damping ratio in the vicinity of the building modal periods can significantly change the NSC acceleration demands (see Chapter 5).

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4.8 References


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Improved Equations for Designing Anchored Acceleration-sensitive Nonstructural Components
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Chapter 5

Improved Equations for Designing Anchored Acceleration-sensitive Nonstructural Components


5.1 Abstract

Many studies conducted to understand the influential parameters on the horizontal seismic responses of nonstructural components (NSCs) assume that NSCs and/or their supporting buildings respond in the linear-elastic domain. In seismic design provisions and guidelines (e.g., ASCE 7-16 provisions) a response modification factor, \( R_p \), which accounts for the NSC ductile behavior, is used to reduce the horizontal elastic seismic forces on anchored acceleration-sensitive NSCs whereas no term is introduced to explicitly incorporate the effect of the supporting building inelasticity. The value of \( R_p \) factor in ASCE 7-16, which varies in the wide range of 1.0-12.0 for different types of NSCs, has been prescribed based on engineering judgment rather than experimental or numerical studies. In addition to accounting for the NSC ductile behavior, \( R_p \) was also incorporated to take into account the effect of dynamic characteristics of NSCs (personal communication with Robert E. Bachman, 2018). For example, it was recognized by the ASCE 7 committee of NSCs that viscous damping of different types of NSCs could be different, and this fact was reflected by prescribing lower \( R_p \) values for potentially low-damping NSCs. In the prescribed equations, NSCs are implicitly assumed to behave as single-degree-of-freedom (SDOF) systems. However, it was acknowledged that distributed systems such as piping systems are far away from a SDOF system. The cumulative modal mass of such systems is not significantly large even after considering significantly large number of modes. In addition, the out-of-phase vibration of different modes of such a distributed mass system can cancel out one another. Based on these interpretations, a larger value was assumed for some NSCs such as piping systems.
The present study uses the floor spectra obtained from several code-based designed (referred as to archetype) steel moment resisting frame (SMRF) and reinforced concrete shear wall (RCSW) buildings to evaluate the effect of the supporting building and/or NSC nonlinear behavior on NSCs seismic demands. These buildings are consistent with the buildings designed as part of ATC-63 project. Nonlinear response history analyses are conducted with a set of 20 spectrum-compatible ground motions and a set of 44 amplitude-scaled far-field ground motions. The archetype buildings are exposed to ground motions scaled to intensity levels varying from 0.25 DE to 1.5 DE (where term DE refers to design earthquake level) to simulate different levels of supporting building nonlinearity. The effect of the value of viscous damping of the supporting building on NSCs seismic demand is quantified for linear and nonlinear building cases. An important argument regarding the use of archetype buildings is that these buildings were near-optimally designed based on the modern seismic provisions. Simulation results corroborate this argument given that a near-full nonlinear (beam-hinge) mechanism is developed in the archetype buildings’ lateral-load resisting system elements under the DE level ground motions. However, in practice, buildings are generally designed with significantly overdesigned structural members. To investigate the effect of overdesiging the supporting buildings on NSCs demands, an additional version of the archetype buildings with the presence of an overdesign factor of 1.5 is considered. This overdesign factor is applied to the lateral stiffness and strength of archetype buildings, and hence is in addition to the inherent overstrength present in the system. Furthermore, the effect of the localized plasticity versus the widespread plasticity in the supporting building on NSC seismic demands is investigated by studying a representative archetype building with a weak-story mechanism.

Constant-ductility floor spectra assuming displacement ductility ratios (from here on denoted as ductility for brevity) varying from 1.0 to 8.0 and viscous damping ratios ranging from 2% to 8% are developed for different floor levels of the linear and nonlinear buildings. A parameter denoted as $R_{cc}$, which is equivalent to $R_p$ used in ASCE 7-126 provisions, is evaluated to quantify the effect of component nonlinearity on its seismic force demand. The results of nonlinear response history analyses reveal $R_p$ is, at different extents, a function of NSC characteristics (i.e., expected ductility demand, tuning period ratio, and viscous damping ratio) and the floor motion frequency content, which the latter is primarily dictated by the relative height of the floor of attachment of NSC, ground motion excitation characteristics, and the supporting building nonlinearity, lateral-
load resisting system and modal periods. The largest reduction in seismic force demands due to the NSC nonlinearity occurs for a low-damping tuned NSC located at the roof floor of a supporting building responding in the linear-elastic range. In other words, this effect is highlighted for NSC exposed to floor motions with more harmonic-like characteristics. The beneficial effect of component nonlinearity can significantly decrease in the absence of one of the aforementioned four conditions, especially the tuning condition. The effect of supporting building nonlinearity on the NSC seismic demands is quantified using a parameter denoted as \( R_{cb} \). Results show that the maximum beneficial effect of the supporting building nonlinearity on NSC seismic demands is obtained for an elastic low-damping roof-mounted tuned NSC.

Nonlinear floor displacement spectra are also developed for different archetype buildings. To quantify the effect of NSC ductile behavior on its displacement demand, a parameter denoted as nonlinear displacement ratio, \( C_{cc} \), is evaluated. The value of \( C_{cc} \) is compared with the predictions of nonlinear displacement ratio for ground spectra (e.g., the well-known equal displacement principle). Results show that for the most critical NSCs (i.e., those in tune with one of the modal periods of the supporting building), NSC nonlinearity not only reduces the force demands on NSCs, but also can significantly reduce their displacement demands. A direct application of constant-ductility spectra in practice is not consistent with the current seismic design provisions, which are force-based. In addition, this method needs targeting a specific NSC ductility value that is very challenging to control in reality. Therefore, to be consistent with the current provisions for designing supporting buildings, the \( R_{cc} \) factor could be incorporated into the baseline equations that are based on a linear component assumption. Subsequently, ductility and displacement demands associated with a given \( R_{cc} \) value could be checked and compared with values permitted in design. As part of this chapter, the consequences of using the current ASCE 7-16 \( R_p \) values for designing NSCs is investigated. As the most important conclusion, it is illustrated that following current provisions for rigid NSCs, which allow them to become inelastic, causes excessive ductility demands on these NSCs. Recently, the Applied Technology Council launched the ATC-120 Project on Seismic, Analysis, Design, and Installation of Nonstructural Components and Systems. The authors of the present study were members of this project. In the final section of this chapter, the accuracy of the recently proposed equations by ATC-120 for designing NSCs is evaluated. Significant parts of this chapter were included in the ATC-120 Project final report (NIST GCR 18-
In the last section of this chapter, results of studies mostly conducted after the ATC-120 equations were established are presented. The results suggest additional potential improvements to these equations.

**Keywords:** Acceleration-sensitive nonstructural components; Archetype buildings; Constant-ductility nonlinear spectrum; Response modification factor; Nonlinear displacement ratio; ATC-120 design equations.

### 5.2 Introduction

Nonstructural components (NSCs) are those parts, elements, or systems attached to or mounted on or hanged from the building floors or walls that are not part of the intended structural load-resisting system. In other words, the term NSC can be applied to elements and sub-systems in a building that are not part of the structural skeleton. Therefore, it is not surprising that NSCs account for most of the total investment in typical building structures. Post-earthquake reconnaissance reports have illustrated that many affected facilities during the past earthquake events, whereas remained structurally intact, completely or partially lost their functionality because of seismic damage to NSCs (Sullivan et al. 2013; Filiatrault and Sullivan 2014). These reports have indicated that losses from damage to NSCs can be significant and even exceed losses from structural damages (McKevitt et al. 1995; Filiatrault et al. 2001; Filiatrault et al. 2002; Myrtle et al. 2005; Gupta and McDonald 2008). In addition, failures of NSCs may pose serious life safety concerns to the building occupants due to falling ceilings, walls, etc., and disrupt the operation of post-disaster facilities and critical public services such as emergency shelters, hospitals, fire and police stations, power stations, water supply and water treatment plants (Lin and Mahin (1985)).

Despite the significant importance of NSCs, the seismic design provisions for NSCs in building codes are rather simple, arguably more prescriptive and with a weaker research-basis than those developed for designing the supporting building structural components. NSCs are generally categorized based on their controlling mode of failure into two broad classes of primarily acceleration-sensitive (e.g., suspended ceilings and water tanks) and primarily drift-sensitive NSCs (e.g., partitions walls and architectural facades). The present study focuses on acceleration-sensitive NSCs, or more specifically, on the horizontal seismic demands on those acceleration-sensitive NSCs that can be reasonably modeled as SDOF systems.
Seismic induced forces on acceleration-sensitive NSCs are a function of the characteristics of the supporting building, and the component-support-attachment system that is briefly denoted as NSC herein. The effect of the primary building characteristics on NSCs seismic demands has been extensively addressed in the literature (e.g., Lin and Mahin 1985; Sewell et al. 1986; Toro et al. 1989; Singh et al. 2006; Sankaranarayanan and Medina 2007; Chaudhuri and Villaverde 2008; Vukobratović and Fajfar 2015; Anajafi and Medina 2018b, c). Nowadays, it is well established that the supporting building nonlinear behavior, viscous damping ratio, lateral-load resisting system, modal periods, in-plane diaphragm flexibility, torsional responses, the height of the point of attachment of NSC to the structure, etc. substantially influence seismic demands on NSCs, although some of these effects are not explicitly (and is some cases adequately) incorporated into seismic design provisions such as those in ASCE 7-16 (Anajafi and Medina 2018b, c). For example, the nonlinear behavior of a building may significantly influence the response of NSCs. This effect is mainly in the form of a reduction with respect to the corresponding linear building response, but also in the form of an amplification in some special cases as reported by Lin and Mahin (1985); Toro et al. (1989); Chaudhuri and Villaverde (2008); Sankaranarayanan and Medina (2008). It is worthy to note that the detrimental effect of building nonlinearity is highlighted in the case of non-tuning NSCs, especially when a NSC is elastic, its viscous damping ratio is relatively low and its frequency is high.

The most salient characteristics of a typical NSC include fundamental period, viscous damping ratio and ductility capacity [of its support(s) and attachment(s)]. The effect of NSCs period on their seismic demands has been extensively investigated through the concept of elastic floor response spectrum. For many years, it has been well understood that the ratio of NSC period to the modal periods of the supporting building (i.e., the tuning ratio) is a key parameter in the estimation of demands on NSCs. Consistent with the fundamentals of structural dynamics, is has been shown that demands on a tuned NSC can be several times that of a non-tuned NSC. This effect is [partially] reflected in the ASCE 7-16 equation using the \( a_p \) factor, which is 1.0 for rigid (i.e., less likely tuned) and 2.5 for flexible (i.e., more likely tuned) NSCs. The effects of the two other influential characteristics of NSCs, i.e., NSC viscous damping and nonlinear behavior, are incorporated into the ASCE 7-16 design equations though the \( R_p \) factor (personal communication with Robert E. Bachman, 2018). However, at present time, there is no clear understanding of the
consequences (e.g., imposed displacement and ductility demands on NSCs) of designing NSCs adopting these $R_p$ factors. Only few studies are available in the literature addressing the effects of these two parameters on NSCs seismic demands, or at least these effects have been not quantified in such a way that can be incorporated into the design equations.

Despite the existence of some early studies (e.g., Kawakatsu et al. 1979; Viti et al. 1981; Igusa 1990) that considered the NSC inelasticity (particularly for equipment in nuclear facilities), and also some works that investigated the effect of NSCs damping ratio on their seismic demands (e.g., Singh et al. 2006; Sankaranarayanan and Medina 2007; Sullivan et al. 2013; Calvi and Sullivan 2014a; Vukobratović and Fajfar 2017), the vast majority of recent studies have been limited to elastic NSCs with a 5% viscous damping ratio (e.g., Naeim et al. 1998; Rodriguez et al. 2002; Adam and Furtmüller 2008; Taghavi and Miranda 2008; Fathali and Lizundia 2011; Wieser et al. 2013; Wang et al. 2014; Anajafi and Medina 2018b). However, very limited evidence exists to suggest that this 5% viscous damping ratio is appropriate in all (or at least most) cases. Recently, discussions were ensued by members of ATC-120 Project regarding a literature review they conducted on the value of NSCs viscous damping. Based on these discussions, it was determined that only few experimental studies have been conducted to quantify the NSCs viscous damping ratio (e.g., Watkins et al. 2009; Watkins 2011; Archila et al. 2012; Astroza et al. 2015). These studies, which were mostly conducted on mechanical and electrical equipment (at low stress levels), consistently suggest that the viscous damping ratio of the majority of tested NSCs falls below the nominal 5% value. Given the low stress level during these tests, further studies are required to draw a final conclusion regarding an appropriate value of viscous damping for different types of NSCs. However, the fact that the viscous damping mechanisms in NSCs are much lesser than those in their supporting buildings for which a 2-5% damping ratio is typically used, can further justify the use of a lower viscous damping ratio for NSCs. Based on the authors experience working with practicing engineers, an argument is usually made that inelasticity, which occurs at high intensity stress level, can significantly increase the damping ratio of NSCs. However, herein, one should be mindful that viscous damping refers to the energy dissipation through the damping mechanisms in the elastic range of NSCs but not the inelastic range that is already accounted for by applying the $R_p$ factor.

To the best of the authors’ knowledge, the effects of NSCs viscous damping ratio and/or
inelastic behavior on their seismic demands were not adequately quantified in such a way that could be applied for adjusting the baseline code-prescribed NSC seismic design equations, which assume an elastic NSCs with a 5% viscous damping ratio. In the ATC-120 project, discussions ensued on using a lower viscous damping ratio for NSCs and also an improved approach for the quantification of the advantages of NSC nonlinearity. As part of this project, Miranda et al. (2018) proposed a capacity design approach for the nonlinear design of NSCs. In their study, they used the recorded roof floor motions from instrumented buildings in the US, and applied a NSC damping ratio of 2% in addition to the traditional value of 5%. The discussions ensued in the ATC-120 project and the work inspired by Miranda and his coworkers, D. Vamvatsikos and A. K. Kazantzi, served as motivation for sections in this chapter related to the development of nonlinear floor spectra using the floor motions obtained from several archetype buildings exposed to two different sets of ground motion records. Nonlinear floor spectra are developed assuming various NSC ductility values and damping ratios when the supporting building responds either linearly or nonlinearly. Finally, the accuracy of the recently proposed equivalent static equation for designing NSCs by the ATC-120 Project is investigated.

5.3 Background

Inelastic spectra for designing primary buildings have been extensively studied in the past for various levels of ductility capacity (e.g., see Veletsos and Newmark 1960; Fajfar and Vidic 1994; Vidic et al. 1994; Borzi et al. 2001) or yield strength (e.g., see Hatzigeorgiou and Beskos 2009; Ray et al. 2013). However, inelastic floor spectra for designing NSCs have been investigated in a limited number of publications as described next.

In one of the earliest studies, Viti et al. (1981) provided a numerical procedure for the development of inelastic floor spectra. They applied the proposed procedure for computing inelastic floor response spectra with three different viscous damping ratios of 2, 4 and 7% for equipment in a reactor building exposed to a missile impact. They showed that the largest peaks of elastic floor spectra can be strongly reduced by NSCs inelasticity. Viti et al. showed that peak-acceleration reduction factors for the equipment at the top of the building were fairly close to the inverse of the ductility value. They also illustrated that for a ductility of 3.0 and larger, the maximum value of the floor spectrum tends to the peak floor acceleration response. Furthermore,
their study showed that for any ductility factor, inelastic floor spectrum values at very high frequencies (e.g., higher than 20 Hz) tend to the elastic values. Igusa (1990) studied a 2 DOFs primary-secondary system exposed to a stochastic base excitation, and showed for the case of tuned NSCs with intermediate to low frequencies, a relatively small nonlinearity (ductility) in NSCs can significantly reduce their seismic force demands.

Adam and Fotiu (2000) proposed analysis methods, based on decomposition into undamped substructure modes, to evaluate the seismic response of elastic and inelastic NSCs attached to inelastic structures. As a case study, they evaluated responses of a four-story structure exposed to a record from the 1940 El Centro earthquake. An elastic/inelastic 0.3%-damped SDOF secondary system (i.e., NSC) was attached to the roof floor of the studied building. Adam and Fotiu showed that for the case of elastic primary building, the inelastic behavior of the tuned secondary system can reduce its peak displacement demands by a factor as large as 4.0. Villaverde (2006) proposed an approximate method to assess the seismic response of inelastic NSCs attached to inelastic building structures. Villaverde used different strength reduction factors, $R$, to account for the inelastic behavior of NSCs and supporting structure based on their target ductility values. In an illustrative example, Villaverde assumed a NSC ductility value of 2.0 and used the Newmark relationships to estimate the associated $R$ factor (it is important to note that based on the results of the present study, the Newmark relationships that were developed for ground spectra may not be applicable to floor spectra). Chaudhuri and Villaverde (2008) evaluated the seismic response of tuned inelastic NSCs at the roof and second floor levels of eight moment resisting buildings. They assumed two different NSC damping ratios of 0.5 and 2%, and a constant $R$ factor of 6 for all cases. Chaudhuri and Villaverde showed that the NSC displacement ductility demand depends on the relative height of the floor of attachment of NSC, and the type of supporting building, and is different for NSCs tuned to different modal periods of the supporting building. They also illustrated that, for a constant $R$ value, displacement ductility demand for NSCs tuned to the fundamental mode of the supporting building increases with the supporting building inelasticity. In the last three studies, NSCs fundamental periods in all cases were set equal to one of the modal periods of supporting structures (i.e., only tuned NSCs were considered).

Vukobratović and Fajfar (2015) proposed an equation for the direction generation of elastic floor acceleration spectra, for the elastic and inelastic primary systems, from ground motion
spectra. Their equation was based on the superposition of modal responses. The effect of equipment damping ratio was explicitly taken into account. Later in 2017, Vukobratović and Fajfar (2017) added the option of taking into account the inelastic response of the equipment to the proposed equations by increasing its damping (e.g., an equivalent damping of 10% for an equipment ductility of 1.5 was used in the proposed equation). As part of their study, they evaluated floor acceleration spectra for elastic and inelastic SDOF equipment mounted on elastic inelastic SDOF primary structures. They used equipment viscous damping ratios varying from 1% to 7%, equipment ductility values of 1.0 and 1.5, and a primary SDOF system with a period of 0.3 s, viscous damping of 5% and ductility values of 1.0 and 2.0. They conducted response history analyses using a set of 30 ground motions, and showed that equipment inelasticity can lead to a substantial decrease of floor acceleration spectra (with an exception of rigid component) for both elastic and inelastic primary structures. They showed that the effect of equipment damping ratio in the case of inelastic equipment is relatively small.

Obando and Lopez-Garcia (2018) developed constant-strength-reduction (usually denoted as constant-$R$) inelastic displacement floor spectra assuming $R$ values ranging from 3.0 to 8.0, and proposed prediction equations for Inelastic Displacement Ratios (IDRs) of floor spectra. They analyzed the responses of eight buildings (three SAC steel moment frame buildings, three RC moment frame and two RC dual wall-frame buildings designed based on Chilean seismic provisions) subjected to far-field ground motions, which were modeled as a Gaussian zero mean random process. Obando and Lopez-Garcia showed that NSC inelasticity can decrease displacement demands on NSCs tuned to the supporting building modal periods, especially the fundamental period. They concluded that floor spectra’s IDRs are qualitatively similar to those of typical ground spectra in that (i) their values are essentially equal to unity at periods larger than the characteristic period (i.e., the fundamental period of the building in the case of floor spectra); (ii) their values tend to increase at periods increasingly less than the characteristic period. In addition, they showed that, unlike the trend observed for ground spectra’s IDRs, floor spectra’s IDRs do not increase monotonically as NSC period tends to zero. Instead, they exhibit local minima at the modal periods of supporting buildings.

In two separate studies (Kazantzi et al. 2018; Miranda et al. 2018), Miranda and his coworkers, investigated the effect of NSC inelasticity on their seismic demands. They proposed an approach
in which bracing elements of nonstructural elements are designed and detailed to work as seismic fuses that limit forces acting on NSCs and their support(s) and attachment(s). Miranda et al. used the recorded roof acceleration responses of instrumented buildings in California and developed floor spectra for target ductility values of 1.0 (i.e., elastic NSC), 1.5 and 2.0. They showed that a NSC ductility of 1.5 and 2.0 can decrease the maximum value (i.e., at tuning) of the mean component amplification spectrum, $S_{ac}/PFA$, by a factor up to 3.5 and 6.0, respectively. In their study, NSC viscous damping ratio was assumed to be either 2% or 5%. Miranda et al. also showed that the mean value of the maximum displacement demand on a tuned NSC with 2% viscous damping and a ductility of 2.0 is almost 45% of that of the corresponding elastic NSC (i.e., NSC inelasticity provided a mean reduction of 55% in the maximum displacement response of NSCs).

As part of the present study, floor motions obtained from analyzing several elastic/inelastic code-based designed (archetype) building models are used to examine the effect of the inelasticity of NSCs on their seismic demands. The main differences with respect to the study of Miranda et al. are listed next: (i) the floor acceleration motions of code-based designed numerical models are used instead of instrumented buildings responses; (ii) inelastic floor spectra are developed for floor levels other than the roof as well; (iii) the evaluation is conducted for elastic and inelastic supporting buildings (note that majority of instrumented buildings in the US responded elastically, or nearly elastically, see Chapters 2 and 4 for detailed information); (iv) an evaluation of the influence of different parameters (e.g., ground motion type, modal period and lateral-load resisting system) on the value of component response modification factor and floor displacement spectra is conducted (note than in the study conducted by Miranda et al. the mean value of different roof spectra was evaluated regardless of a categorization for the supporting building lateral-load resisting system type, the level of inelastic behavior of supporting building, and ground motion excitation type); (v) in addition to constant-ductility spectra, constant-R spectra are developed.

### 5.4 Analysis methodology, structural models, and ground motions

The analysis methodology used in this study consists of performing response history analyses in which structural models are exposed to different sets of ground motion records. The NSCs under consideration are those that can be represented by SDOF systems with masses that are smaller than the total mass of the supporting structure by a factor of at least 1000. Such light NSCs do not offer
significant dynamic feedbacks to the building. Therefore, as stated by Adam et al. (2013), floor spectra can be adequately developed based on an uncoupled analysis of the supporting building and NSCs (in Appendix 1, the accuracy of this statement is examined on a representative archetype building). For a given building, ground motion, and floor level, the floor acceleration time history is extracted from the finite element model of the supporting building. This floor acceleration motion is then used as input for a separate SDOF analysis program to develop its corresponding floor response spectrum for a pre-defined NSC viscous damping ratio and target displacement ductility value. The viscous damping ratio of interest for NSCs ranges from 2% to 8% of the critical damping. The target ductility of NSCs varies from 1.0 to 8.0. The floor spectral values are computed for 2000 periods equally spaced between 0.005 and 10.0 s.

5.4.1 Structural models

The present study focuses on short-to-midrise buildings, which constitute most of the building inventory in the US. Structural models with heights of one, two, four, six, eight, and 12 stories are utilized in this study. Special steel moment resisting frame (SMRF) and special reinforced concrete cantilever shear wall (RCSW) archetype buildings are studied, which are representative of flexible and stiff lateral-load resisting systems, respectively. These archetype buildings comply with the designed buildings as part of the ATC-63 project (FEMA P695, 2009)). Nonlinear two-dimensional (2D) numerical models of the archetype buildings are developed. All contributions to lateral strength and stiffness from the gravity system are neglected. The global P-Delta effects of gravity loads that are not tributary to the lateral-load resisting elements are incorporated via a leaning column with a zero flexural stiffness. This P-Delta column is linked to the 2D finite element model with axially rigid pin-ended members. To approximate viscous damping of the supporting structure, the Rayleigh approach (based on initial stiffness) assigning a 2.5% viscous damping ratio to $2T_{b1}$ and $T_{b2}$, where $T_{bi}$ is the period of the $i$-th elastic mode of vibration of the building, is implemented. The value of $2T_{b1}$ is selected to limit the viscous damping ratio assigned to the effective fundamental period when the building becomes inelastic. In a separate section, the sensitivity of the elastic floor spectral ordinates to the value of the building viscous damping is investigated for the elastic and inelastic building cases.

The selected SMRF buildings are consistent with the FEMA P695 building group RSA-D$_{max}$. 

5-11
These buildings were designed for a high seismic loading based on the Response Spectrum Analysis approach subjected to the ASCE 7-05 requirements and using a building response modification factor \((R)\) of 8.0 (see FEMA P695 for details of different building groups). The SMRF buildings have a rectangular plan configuration with dimensions of 42.7 and 30.5 m consisting of a three-bay perimeter frame of equal spans (6.1 m) on each side. The representative building plan, which is the same for all SMRF archetypes, is shown in Figure 5-1(a). Nonlinear analysis models for SMRF buildings are based on the concentrated plastic hinge concept. A monotonic moment-rotation backbone curve is used to define the material hysteretic behavior. When defining backbone curves of the nonlinear hinges, a post-yield strain hardening of 3\% is assumed. Other characteristics of the backbone curves are defined according to ASCE 41-13 (2014). Consistent with the approach adopted in FEMA P695, the effects of the composite floor slab are not considered in finite element models. Localized P-M hinges with five different P-M interaction curves for five selected axial force levels, ranging from 0.1 to 0.5 of the columns axial strength, are defined and assigned to the columns ends. The panel zone is not explicitly modeled implying that its shear distortion and shear failure are neglected. Regarding the shear failure of the panel zone, it is assumed that such fractures are adequately controlled by special SMRF design requirements. Consistent with FEMA P695, the bases of the single- and two-story SMRF columns are assumed to be pinned whereas the bases of taller SMRF buildings are fixed. Beam and column sections of the SMRF buildings are ASTM A992 W sections. Material properties for steel sections are specified according to the FEMA P695 with an expected yield strength and expected ultimate strength of 380 and 493 MPa, respectively.

For this study, the RCSW buildings are selected from the FEMA P695 building group with a low axial force and a high seismic design loading (see FEMA P695 for details of different building groups). Proportioning and detailing of RCSWs was based on ACI 318-08 requirements subject to ASCE 7-05 Chapter 14 amendments and adopting an \(R\) factor of 5.0. To generate lower-bound designs without an excessive overstrength, the plan dimensions and length of walls in each direction were varied in each archetype to optimize strength relative to the level of seismic design loading (see Figure 5-1b and Table 5-1). The RCSWs are modeled via nonlinear shell elements based on steel reinforcing and concrete material characteristics provided in FEMA P695. The reinforcing steel details, especially the confining rebars, are modified in some cases with respect
to the FEMA P695 to improve the system global ductility. For the unconfined concrete, the peak expected strength and the corresponding strain are selected as 43.1 MPa and 0.002, respectively; the ultimate strength and the corresponding strain of the unconfined concrete is assumed to be 24.0 MPa and 0.005, respectively. For the confined concrete, the modeling parameters are determined based on the Mander’s model (Mander et al. 1988), which uses the unconfined concrete parameters and the boundary reinforcement details to determine the confined concrete peak stress and strain, and the post-peak behavior parameters. The rebar material is defined using the Bilinear Kinematic Hardening model. The expected yield strength and the ultimate strength of the reinforcing rebars is 460 MPa and 680 MPa, respectively. A tensile strain value of 5% is defined as the limit state associated with rebar buckling and subsequent rebar fracture.

![Diagram](image)

**Figure 5-1** Archetype buildings plan configuration (gravity columns are not shown for clarity)

These archetype buildings were near-optimally designed. In practice, fulfilling different requirements (i.e., construction considerations), may lead to a building with significantly over-designed structural members. To account for this fact, a modified version of each archetype building with a lateral-stiffness and strength increased by a factor of 1.5 is analyzed as well. The primary archetype buildings are denoted as baseline buildings, whereas the additional models are named as overdesigned buildings. For the RCSW family, an additional 12-story building model with an increased fundamental period (i.e., reduced lateral stiffness) is developed as well. This building, which is denoted as the flexible 12-story building, intends to simulate an RCSW with a second modal period situated in the plateau region of the design spectrum.
Table 5-1 illustrates the most salient geometric and modal characteristics of the baseline and overdesigned archetype buildings studied in this paper. As seen, the modal periods of the selected buildings cover different sections of typical design spectra (i.e., periods smaller than the first corner period, plateau, intermediate and relatively long periods). In this table, approximate fundamental period of the buildings, $T_{ab}$, computed based on the ASCE 7-16 Equation 12.8-7 are also presented. As seen, the building models are more flexible than the ASCE 7-16 expectation.

**Table 5-1  Characteristics of the archetype buildings utilized in this study**

<table>
<thead>
<tr>
<th>System</th>
<th># of stories</th>
<th>Height (m)</th>
<th>Plan dimensions (m × m)</th>
<th>$T_{ab}$ (s)</th>
<th>Baseline $T_{b1}$</th>
<th>Baseline $T_{b2}$</th>
<th>Baseline $T_{b3}$</th>
<th>Overdesigned $T_{b1}$</th>
<th>Overdesigned $T_{b2}$</th>
<th>Overdesigned $T_{b3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SMRF</td>
<td>1</td>
<td>4.6</td>
<td>42.7 × 30.5</td>
<td>0.24</td>
<td>0.71</td>
<td>-</td>
<td>0.57</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>8.5</td>
<td></td>
<td>0.40</td>
<td>1.01</td>
<td>0.21</td>
<td>0.80</td>
<td>0.17</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>17.7</td>
<td></td>
<td>0.68</td>
<td>1.67</td>
<td>0.52</td>
<td>0.23</td>
<td>1.35</td>
<td>0.41</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>24.4</td>
<td></td>
<td>0.93</td>
<td>1.87</td>
<td>0.62</td>
<td>0.32</td>
<td>1.52</td>
<td>0.50</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>36.0</td>
<td></td>
<td>1.17</td>
<td>2.30</td>
<td>0.80</td>
<td>0.44</td>
<td>1.86</td>
<td>0.64</td>
<td>0.35</td>
</tr>
<tr>
<td>RCSW</td>
<td>12</td>
<td>54.3</td>
<td></td>
<td>1.61</td>
<td>3.14</td>
<td>1.08</td>
<td>0.61</td>
<td>2.53</td>
<td>0.88</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>4.0</td>
<td>121.9 × 106.7</td>
<td>0.14</td>
<td>0.20</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.6</td>
<td>85.3 × 73.2</td>
<td>0.22</td>
<td>0.49</td>
<td>0.09</td>
<td>-</td>
<td>0.39</td>
<td>0.08</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>14.9</td>
<td>45.7 × 42.7</td>
<td>0.37</td>
<td>0.68</td>
<td>0.11</td>
<td>0.06</td>
<td>0.58</td>
<td>0.09</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>22.3</td>
<td>33.5 × 30.5</td>
<td>0.50</td>
<td>0.90</td>
<td>0.12</td>
<td>0.07</td>
<td>0.76</td>
<td>0.10</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>29.6</td>
<td></td>
<td>0.62</td>
<td>1.13</td>
<td>0.13</td>
<td>0.07</td>
<td>0.80</td>
<td>0.10</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>44.2</td>
<td>32.0 × 30.5</td>
<td>0.84</td>
<td>1.34</td>
<td>0.17</td>
<td>0.08</td>
<td>1.0</td>
<td>0.14</td>
<td>0.06</td>
</tr>
<tr>
<td></td>
<td>12 $^d$</td>
<td>44.2</td>
<td></td>
<td>0.84</td>
<td>1.66</td>
<td>0.30</td>
<td>0.12</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

(a) $T_{ab}$ is the approximate fundamental period computed based on ASCE 7-16 Equation 12.8-7; (b) not applicable; (c) modal periods of the RCSWs are estimated based on the spikes observed in the mean roof floor spectra at the DE level; (d) this building is a more flexible version of the baseline 12-story RCSW.

### 5.4.2 Ground motions

Two suites of ground motion records including a set of spectrum-compatible (SC) and a set of actual far-field (FF) records are used in the response history analyses conducted in this chapter. As illustrated by Anajafi and Medina (2018c), a relatively large variation can be observed about the mean value of structural responses under a set of SC records even when individual records are tightly matched to the same target spectrum. Hence, a sufficient number of SC records should be used in inelastic response history analyses to address this record-to-record variability. This study utilizes a set of 20 SC ground motions, which are generated through matching the individual 5%-damped response spectra of 20 actual far-field records to the target spectrum. The spectral matching is conducted using the wavelet adjustment technique (Al Atik and Abrahamson 2010).
The target spectrum used for generating simulated records (with the characteristics \( S_{DS} = 1.0 \text{ g} \) and \( S_{D1} = 0.55 \text{ g} \)) is similar to the elastic 5%-damped design spectrum implemented in FEMA-P695 for designing the archetype buildings. The spectral misfit (i.e., deviation of a response spectrum ordinate from the target spectrum ordinate) over the period range 0.05-5.0 s for all simulated records is limited to 2% implying a tight spectral matching. Figure 5-2(a) illustrates the 5%-damped ground spectra for the SC records.

The second suite used in this study is the FEMA-P695 FF record set, which is amplitude-scaled based on ASCE 7-16 provisions for different archetype buildings. The scale factor is selected in such a way that the average of the maximum-direction spectra from all the ground motions does not fall below 90% of the target response spectrum for any period within the period range \( T_{b2} \sim 2T_{b1} \). This approach results in different scale factors to be applied to individual ground motions for various archetype buildings. Figure 5-2(b) illustrates the 5%-damped ground spectra of the amplitude-scaled FF records for the eight-story RCSW building, as an example. This set includes 44 ground motions, which were recorded in stiff soils (i.e., NEHRP site class D), have a moment magnitude that varies from 6.5 to 7.6 with an average of 7.0, and closest distances to the fault rupture that vary from 11.1 to 26.4 km with an average of 16.4 km. For the unscaled records of this set, the peak ground acceleration (PGA) values vary from 0.21 to 0.82 g with an average of 0.43 g; and the peak ground velocity (PGV) values vary from 19 to 115 cm/s with an average of 46 cm/s. Approximately 60% of the records are from California earthquakes, and the rest are from outside the US. Detailed information on the FF ground motions can be found in FEMA-P695. It should be noted that these ground motions were referred to as far-field in FEMA-P695 because their recording distance to the fault rupture area was greater than 10 km. However, in several cases, the potential evidence of forward rupture directivity effects (near-field effects) can be observed in the response spectra of these records (see the relatively large spectral ordinates at the long periods shown in Figure 5-2b).
The major part of analyses in this chapter are performed using SC records, and in specific cases, the FF record set is used for the validation purposes. Because SC records are used in the primary computations, the dispersion in structural responses (record-to-record variability) is not as significant as when actual ground motion records are used. Therefore, the mean is selected as the primary descriptive statistics.

### 5.4.3 Parameters and nomenclatures

The most salient parameters evaluated in this study, and the nomenclatures used are presented next:

**NSC** = component-support-attachment system.

**\( T_{bi} \)** = elastic period of the supporting building at the \( i \)-th mode of vibration.

**\( T_c \)** = elastic fundamental period the component-attachment-support system, which is named NSC period for brevity.

**HM** = higher-mode region of a floor spectrum, defined as \( 0 < T_c \leq 1.5T_{b2} \).

**FM** = fundamental-mode region of a floor spectrum, defined as \( 1.5T_{b2} < T_c \leq 10 \) s.

**PGA** = peak ground acceleration.

**PFA** = peak floor acceleration.

**PCA** = peak component acceleration response over a given component period range.
\( \xi_b = \) Rayleigh viscous damping ratio specified to the supporting building’s modal periods, which is called building damping for brevity.

\( \xi_c = \) viscous damping ratio of the component-attachment-support system, which is termed NSC damping ratio for brevity.

\( S_{dc} = \) maximum displacement response of an elastic or inelastic NSC.

\( S_{ac} = \) the maximum force demand on an elastic or inelastic NSC, \( F_c, \) normalized to the NSC operating weight, \( W_c. \) \( S_{ac} \) is denoted as the (pseudo) spectral acceleration response of NSC and is computed from the following equation:

\[
S_{ac} = \frac{F_c}{W_c} \quad (5.1)
\]

\( DMF_c = \) elastic-component damping modification factor, which account for NSC viscous damping ratio other than the prevalent 5%:

\[
DMF_c = \frac{S_{ac,\xi}}{S_{ac,5\%}} \quad (5.2)
\]

\( \mu_c = \) target (demand) displacement ductility of component-support-attachment system, which is called NSC ductility for brevity:

\[
\mu_c = \frac{S_{dc}}{\Delta_{\text{yield}}} \quad (5.3)
\]

where \( \Delta_{\text{yield}} \) is the yield displacement of the NSC.

In this study, the parameters \( R_{cb} \) and \( R_{cc} \) are used to evaluate the effect of the supporting building and the component inelasticity on NSC seismic force demands, respectively. The subscript “cb” refers to the response modification factor of component due to building inelasticity, whereas the subscript “cc” stands for the response modification factor of component due to component inelasticity:

\( R_{cb} = \) reduction or amplification in the force demand on NSC due to building inelastic behavior:

\[
R_{cb} = \frac{[S_{ac}]_{\text{elastic bldg}}}{[S_{ac}]_{\text{inelastic bldg}}} \quad (5.4)
\]

\( R_{cc} = \) reduction or amplification in the force demand on NSC due to NSC inelastic behavior:
\[ R_{cc} = \frac{S_{ac}^{\text{elastic NSC}}}{S_{ac}^{\text{inelastic NSC}}} \]  

(5.5)

\[ C_{cc} = \text{inelastic displacement ratio of NSC}: \]

\[ C_{cc} = \frac{S_{dc}^{\text{inelastic NSC}}}{S_{dc}^{\text{elastic NSC}}} \]  

(5.6)

\[ R_{PFA} = \text{reduction or amplification in PFA demand due to the building inelastic behavior.} \]

Many of the above-mentioned parameters have been used with the same or different names in the literature. For example, different parameters were used to quantify the effect of supporting building inelasticity on the elastic force demands on NSC. Lin and Mahin (1985) used a parameter called amplification factor, which is equivalent to the inverse of the \( R_{cb} \) factor used in the present study. Sewell et al. (1986) used an expression called floor response spectra ratio (FRSR) that is the ratio of floor response spectrum for the inelastic supporting structure normalized by that of the corresponding elastic supporting structure. This ratio is equivalent to the inverse of the \( R_{cb} \) factor. Sankaranarayanan and Medina (2007) defined a parameter denoted as acceleration response modification factor (\( R_{acc} \)) that is equivalent to the \( R_{cb} \).

To quantify the effect of the inelastic behavior of NSC on its seismic force demands, also different parameters were used. Viti et al. (1981) used an expression denoted as peak reduction coefficient. Kazantzi et al. (2018) proposed a parameter called \( R_{\mu \text{comp}} \) that is identical with the \( R_{cc} \) used herein. Lastly, the term inelastic displacement ratio was used in many different works to quantify the effect of supporting building inelasticity on the building displacement demand. However, this term was seldom applied to floor spectra. For example, Obando and Lopez-Garcia (2018) used the symbol \( C_d \) for the inelastic displacement ratio of NSC that is equal to \( C_{cc} \) used in the present study. Miranda et al. (2018) also used the term inelastic displacement ratio for quantifying the ratio of the maximum displacement of an inelastic NSC to that of an elastic NSC in tuning condition.
5.5 Influential parameters on NSC elastic seismic demands

5.5.1 Supporting building type (lateral-load resisting system and period)

From the results of previous studies, it is well understood that the lateral-load resisting system and fundamental period of supporting building significantly influence floor response spectra. Figure 5-3 illustrates 5%-damped roof spectra for all baseline and overdesign archetype buildings. An evaluation of this figure reveals a significant building-to-building variability in the shape and amplitude of floor spectra. RCSWs with relatively short periods, exhibit sharp spikes NSCs in the vicinity of their second modes that coincide with the relatively short period NSCs (i.e., $T_c \leq 0.25$ s). For these buildings, a third-mode spike in floor spectra is not clearly observed because the third mode is relatively small and not excited by ground motions. SMRF buildings exhibit relatively large spikes in the intermediate NSC period region $0.5 \leq T_c \leq 1.0$ s. It is observed that with in a lateral-load resisting system family, with increasing the building fundamental period, spectral ordinates in the vicinity of the fundamental mode tend to decrease; however, the effect of higher mode become highlighted because they situate in the ground spectra constant acceleration region.

![Figure 5-3](image)

*Figure 5-3  Elastic 5%-damped roof spectra for all baseline and overdesigned RCSW and SMRF buildings exposed to DE level SC records*

5.5.2 Supporting building viscous damping

In the primary analyses conducted in this paper, the building Rayleigh viscous damping ratio specified to $2T_{b1}$ and $T_{b2}$ is assumed to be 2.5%. In this section, the sensitivity of the roof floor spectral ordinates to the value of the building viscous damping ratio is investigated for the baseline
six-story SMRF and eight-story RCSW buildings. The buildings are analyses under the DE level (i.e., and inelastic building behavior) and 0.25 DE level (i.e., an elastic building behavior) using the SC ground motions. Hereinafter, structural responses under the 0.25 DE and 1.0 DE ground motions intensity levels are assumed to be representative of the elastic and inelastic building behavior, respectively.

Figures 5-4(a) and (b) illustrate the mean normalized elastic 5%-damped roof spectra for the six-story building responding inelastically and elastically, respectively. Figures 5-5(a) and (b) quantify the sensitivity of the elastic 5%-damped roof spectrum to the value of the supporting building viscous damping for the elastic and inelastic building scenarios. As seen in Figures 5-5(a) and (b), the influence of the building viscous damping on NSC seismic demands is more pronounced for NSC periods in the vicinity of the building modal periods because of the semi-harmonic motions that dominate responses at these NSC periods. In addition, this effect is less pronounced in the inelastic building than in the elastic building. The reason for this latter observation is that the hysteretic damping provided by the building inelasticity is significantly larger than the building viscous damping, and hence, floor response spectra of the inelastic building are less sensitive to the value of building viscous damping. For the inelastic building case, the effect of building viscous damping for NSC periods in the vicinity of the third and fourth modes is considerable whereas this effect in the vicinity of the building fundamental mode is not very significant. This is because the fundamental mode of the building experiences larger inelastic actions (i.e., a larger equivalent viscous damping ratio) than the higher modes, and hence fundamental-mode dominated responses are less sensitive to the value of the building viscous damping. For the inelastic building case, in the vicinity of the second mode, wherein the spike dominates the floor spectrum, the variation in the spectral ordinates due to the considered deviations of building viscous damping from the baseline value of 2.5% is limited to ±18%, whereas for the elastic building case this variation is up to +42%/-30%.
Figure 5-4  Mean normalized elastic 5%-damped roof spectra for the six-story baseline SMRF exposed to the SC records assuming different building viscous damping ratios (a) inelastic building; (b) elastic building

Figure 5-5  Sensitivity of the elastic 5%-damped roof spectra to the value of the supporting building viscous damping for the six-story baseline SMRF exposed to the SC records (a) inelastic building; (b) elastic building

Similar analyses are conducted for the eight-story baseline RCSW building and the results are presented in Figures 5-6 and 5-7.
5.5.3 The level of inelastic behavior of supporting building

5.5.3.1 Elastic roof spectra for inelastic and elastic buildings

Elastic floor response spectra have been extensively studied in the past. The effect of supporting building inelasticity (or the ground motion intensity level) on the elastic floor spectra has been also focus of many previous studies (see e.g., Sewell 1986; Igusa 1990; Adam and Fotiu 2000; Rodriguez et al. 2002; Medina et al. 2006; Politopoulos and Feau 2007; Sankaranarayanan and Medina 2007; Adam and Furtmüller 2008; Chaudhuri and Villaverde 2008; Sullivan et al. 2013;
Wieser et al. 2013; Calvi and Sullivan 2014b; Petrone et al. 2015; Vukobratović and Fajfar 2015; Petrone et al. 2016; Anajafi and Medina 2018b). In most of these studies, SDOF inelastic buildings or MDOF inelastic generic frames were used. One primary goal of the present study is to evaluate the design equations for NSC. These design equations are meant to be used for NSCs mounted on actual code-based designed buildings. Hence, there is a need to reproduce and reevaluate some of the previously well-known behaviors. The present subsection, uses the response of several code-based designed buildings to examine the effect of supporting building inelasticity on elastic floor spectral acceleration responses. Although it is expected that the results of this evaluation only corroborate the previous researchers’ observations, it is still an essential step toward the ultimate goal of this study.

The archetype buildings are exposed to the SC records scaled to different intensity levels varying from 0.25 DE to 1.50 DE to simulate different levels of inelastic behavior of the supporting building. In this section, examples of 5%-damped elastic floor spectra for two baseline archetype buildings exposed to the SC ground motions with different intensity levels are illustrated.

Figure 5-8(a) illustrates the dispersion in the normalized 5%-damped elastic roof spectra for the eight-story baseline RCSW due to record-to-record variability present in the set of SC records at the DE level. As seen, the dispersion in spectral ordinates, especially in the vicinity of the supporting building second mode (i.e., $T_c = 0.13$ s), is significant. Figure 5-8(b) presents mean values of normalized roof spectra for different ground motion intensity levels.

![Image](image-url)

**Figure 5-8** (a) Dispersion in the normalized 5%-damped elastic roof spectra due to the record-to-record variability at the DE level, (b) mean normalized roof spectra for different intensities; for the eight-story baseline RCSW exposed to the SC records

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Figure 5-8(b) illustrates that with increasing the ground motion intensity, at most component periods, especially at tuning situations, the mean normalized roof spectral ordinates markedly decrease. This is consistent with results of many previous studies listed earlier in this section. At longer component periods (e.g., $T_c > 1.2$ s) inelasticity may slightly increase the mean normalized demands. However, these periods are not in the practical range of typical acceleration-sensitive NSCs, and moreover, the mean normalized demands at these relevant component periods do not dominate the peak values of the mean normalized response spectrum. As seen in Figure 5-8(b), the maximum value of the mean spectrum for all intensity levels, except for the 0.25 DE, occurs at component periods in the vicinity of the supporting building second-modal period. Given that ASCE 7-16 equations are meant to provide demands at a DE level, the normalized floor spectra corresponding to the 1.0 DE should be used for the evaluation of these equations. For this intensity level, at the most critical component case, i.e., when the component period is in the vicinity of the second mode of the building, the normalized mean spectral demands exceed the normalized ASCE 7 limit of 4.0 by a factor up to 1.82.

Similar results are shown in Figures 5-9 for the four-story baseline SMRF building. Although results obtained from the evaluation of the four-story SMRF and eight-story RCSW buildings are different in some details, the overall trends regarding the effect of supporting building inelasticity on the PCA responses are consistent.
5.5.3.2 Evaluation of the parameter $R_{cb}$ for elastic floor response spectra

The effect of the supporting building inelasticity on the NSC seismic demands is quantified through a parameter denoted as $R_{cb}$. A larger $R_{cb}$ value implies a larger beneficial effect on NSC force demands due to the supporting building inelastic behavior. An $R_{cb}$ value less than unity illustrates the detrimental effect of the supporting building inelasticity on NSCs responses. In this section, $R_{cb}$ is evaluated for a representative inelastic supporting building case. Structural responses under the 0.25 DE and 1.0 DE ground motions intensity levels are assumed to be representative of the elastic and inelastic building behavior, respectively. Therefore, for a given building floor and NSC period, $R_{cb}$ is the ratio of the normalized floor spectrum ordinate under the DE level ground motion to the corresponding ordinate under the 0.25 DE.

Figures 5-10(a) to (c) illustrate $R_{cb}$ for the elastic roof spectra of the six-story baseline SMRF inelastic building under the SC records for component viscous damping ratios of 0.1%, 1% and 5%. Figure 5-11(a) to (c) depict similar results for the second-floor level of this building. An evaluation of Figures 5-10 and 5-11 illustrates that: (i) record-to-record variability in the $R_{cb}$ values is significant, especially for the low-damping floor spectra. (ii) $R_{cb}$ value in the vicinity of the building fundamental period is significantly larger than that in the vicinity of the higher modes. This is because the higher-mode’s inelasticity is not as significant as the fundamental-mode’s inelasticity. (iii) $R_{cb}$ is larger for a low-damping floor spectrum. For example, the maximum value of the mean $R_{cb}$ in Figure 5-10(a) with a component damping ratio of 0.1% is 2.4 whereas this quantity in Figure 5-10(c), where the component damping is 5%, is limited to 2.0. (iv) $R_{cb}$ values less than unity are observed at some non-tuning situations under individual ground motion records. This behavior is more pronounced for the low-damping floor spectra and in the higher mode region. This is consistent with the observations made by Toro et al (1989); Sankaranarayanan (2007); Sankaranarayanan and Medina (2008). As discussed by Toro et al (1989), detrimental effect of the supporting building inelasticity on NSC demands is due to high-frequency pulses that are generated every time a member of the structure undergoes a stiffness change.

Under some individual ground motions, especially in non-tuning conditions, inelastic structural behavior may cause an increase in elastic demands on NSCs, especially for low-damping short-period ratio NSCs. However, in most cases, building inelasticity decreases demands on NSCs particularly when the period of the component coincides with one of the modal periods of the
primary structure. This is consistent with the results of Sankaranarayanan and Medina (2008). As an important conclusion, the mean value of $R_{cb}$ for all cases are reduced or at worse slightly increased. The de-emphasized adverse effect of building inelasticity on NSC demands in this study with response to some previous works, is most likely because in this study SC ground motions are used that are rich in the high frequency region. Singh et al. (1996) concluded that the amplification in average floor spectral ordinates due to the inelastic behavior of the supporting building decreases when the input energy at the frequencies of the higher modes is significant.

Some previous research works (e.g., Sewell et al. 1987; Chaudhuri and Villaverde 2008; Sankaranarayanan and Medina 2008) reported that the adverse effect of the building nonlinearity on the PCA responses is more pronounced when the nonlinearity is localized (e.g., a weak-story mechanism is formed). This behavior is evaluated for a representative archetype building in subsection 5.5.3.3.

Figure 5-10 Dispersion in the $R_{cb}$ of the elastic roof spectra due to the record-to-record variability of the SC record set for the six-story inelastic baseline SMRF, assuming a component damping ratio of (a) 0.1%; (b) 1% and (c) 5%

Figure 5-11 Dispersion in the $R_{cb}$ of the elastic second-floor spectra due to the record-to-record variability of the SC record set for the six-story inelastic baseline SMRF, assuming a component damping ratio of (a) 0.1%; (b) 1% and (c) 5%
Impact of localized plasticity on the value of the parameter $R_{cb}$ in elastic floor spectra

In the previous section, the effect of the inelastic behavior of the supporting building on NSC force demands was evaluated using the floor acceleration responses of the six-story baseline SMRF building. This archetype building is a well-behaved structure in which plastic hinges are developed at the two ends of the beams at different stories and also at the base of the first-story columns (i.e., a widespread inelasticity). In the present section, the finite element model of the six-story baseline SMRF building is modified in such a way that plastic rotational hinges can develop only at the first-story columns. To this end, a relatively large factor of 20 is assigned to the yield strength of all structural members other than the first-story columns. Other characteristics of the model, including elastic modal periods, are kept the same as in the baseline building. This model simulates a building with a weak first-story mechanism.

Figures 5-12(a) to (c) illustrate the parameter $R_{cb}$ for the elastic roof spectra of the six-story baseline SMRF building for component viscous damping ratios of 0.1%, 1% and 5%, respectively. An evaluation of the results of this section and Section 5.5.3.2 illustrates that the localized plasticity can lead to smaller $R_{cb}$ values with respect to the widespread plasticity. For example, the maximum value of $R_{cb}$ in Figure 5-10(a) for the baseline building is 2.4 whereas the corresponding value for the building with a weak-story mechanism, shown in Figure 5-12(a), is 1.7. It is also observed that the detrimental effect of the building inelasticity on NSC seismic demands (i.e., increasing the seismic demand on NSCs due to building inelasticity) is more significant in the building with a weak first-story mechanism.

Figure 5-12 Dispersion in the $R_{cb}$ of the elastic roof spectra due to the record-to-record variability of the SC record set for the six-story SMRF with a weak-story mechanism, assuming a component damping ratio of (a) 0.1%; (b) 1% and (c) 5%
Figures 5-13(a) to (c) depict similar results for the second-floor level of this building.

![Graphs showing dispersion in the $R_c$ of the elastic second-floor spectra due to record-to-record variability of the SC record set for the six-story SMRF with a weak-story mechanism, assuming a component damping ratio of (a) 0.1%; (b) 1% and (c) 5%]

The conclusions of Sections 5.5.3.2 and 5.5.3.3 corroborate the observations made by previous studies. For example, Chaudhuri and Villaverde (2008) stated that “the peak acceleration response of a linear nonstructural component may increase when the supporting building goes from linear to localized nonlinear behavior but not when the building goes from linear to widespread nonlinear behavior. Sankaranarayanan and Medina (2008) also concluded that amplification in NSC demands caused by the supporting building inelasticity is more pronounced at lower floors, where the contribution of higher modes is more significant, of short structures with a weak-first story and when the component damping is very low.

### 5.5.4 The relative height of the point of attachment of NSC

Figure 5-14 presents the in-structure amplification factor, i.e., PFA/PGA or PCA/PGA for a rigid component, versus the relative heights (i.e., $z/h$) for the eight-story baseline RCSW exposed to SC records with different intensity levels. The overall trend observed in this figure suggests that with increasing the ground motion intensity, the normalized PFA response at all floor levels decreases. This observation corroborates results of many past works listed earlier in this section. Figure 5-14 illustrates that for this building at the DE level the ASCE 7-16 equation overestimates the PFA/PGA responses at all floor levels. For example, at the roof level, the computed PFA/PGA ratio is 1.8, which is smaller than the ASCE 7 limit of 3.0.
Figure 5-14  Mean PFA/PGA response versus relative heights for the eight-story baseline RCSW exposed to a set of SC records scaled to various intensity levels

Figures 5-15(a) and 5-15(b) present, respectively, the higher-mode region mean PCA/PFA and PCA/PGA profiles for the eight-story baseline RCSW exposed to SC records with different intensity levels. Similar graphs are presented in Figures 5-15(c) and 5-15(d) for the fundamental-mode region. An evaluation of the results shown in Figures 5-15(a) to (d) illustrate that PCA/PFA responses irregular change when ground motion intensity increases, whereas, PCA/PGA responses exhibit a deduction when ground motion intensity increases.

As seen in Figure 5-15(a), at the DE level, the ASCE 7-16 value of $a_p = 2 \frac{1}{2}$ in many floor levels tend to underestimate the computed higher-mode PCA/PFA responses. As another observation, unlike the ASCE 7-16 provisions, the value of $a_p$ changes along the building height. Similar results were reported in several previous research works listed earlier in this section. An evaluation of Figure 5-15(b) reveals that at the DE level, PCA/PGA of the second-mode at the roof and middle floors exceeds the ASCE 7-16 prescribed values by a factor of 1.9 and 1.8, respectively. For the first-mode region at the DE level, the ASCE 7-16 equations can capture both PCA/PFA and PCA/PGA responses at most floor levels (see Figure 5-15c and 5-15d)
Figure 5-15  Mean PCA/PFA and PCA/PGA responses versus the relative height for the eight-story baseline RCSW exposed to a set of SC records scaled to various intensity levels

Similar graphs are shown in Figures 5-16 and 5-17 for the four-story SMRF building.

Figure 5-16  Mean PFA/PGA response versus relative height for the four-story baseline SMRF exposed to a set of SC records scaled to various intensity levels
5.5.5 Design implication of the acceleration response modification factor for different relative heights

5.5.5.1 Acceleration response reduction factor for the PFA response

The effect of the building inelasticity on PFA responses is quantified using a parameter denoted as $R_{PFA}$. For a given building floor and a given ground motion intensity, the parameter $R_{PFA}$ is calculated using the following equation:

$$ R_{PFA} = \frac{(PFA/PGA)_{0.25\ DE}}{(PFA/PGA)_{\%\ DE}} $$  \hspace{1cm} (5.7)
The mean $R_{PFA}$ profiles for the four-story and eight-story baseline RCSW exposed to SC records with different intensity levels are illustrated in Figures 5-18(a) and (b), respectively. Similar graphs are shown for the four-story and eight-story baseline SMRF in Figures 5-18(c) and (d). Results consistently show that the value of $R_{PFA}$ is highly dependent on the floor relative height and is generally higher for top floors. For example, for the four-story RCSW the value of $R_{PFA}$ at the roof ($RH = 1.0$) and second floor ($RH = 0.25$) levels is 2.2 and 1.3, receptively. The results of this section will be used in Section 5.11 when evaluation of the accuracy of the proposed equation by ATC-120 project for designing NSCs.

*Figure 5-18*  
*PFA response reduction due to the supporting building inelastic behavior versus the relative height for different archetype buildings*
5.5.5.2 Acceleration response reduction factor for the PCA response

Generally speaking, the maximum (peak) values of floor spectra, which correspond to NSC periods in the vicinity of building modal periods, are used in design. In this section, component acceleration reduction factor (obtained from building inelasticity) corresponding to these maximum spectral ordinates are computed. For example, \( R_{cb} \) for the higher-mode region of a floor spectrum is estimated using Equation (5.8). A similar equation is used to compute \( R_{cb} \) for the first-mode region.

\[
(R_{cb})_{PCA-HM} = \frac{\text{Max}[S_{ac}]_{\text{elastic bldg over HM region}}}{\text{Max}[S_{ac}]_{\text{inelastic bldg over HM region}}} \tag{5.8}
\]

Figures 5-19(a) and (b) illustrate \( R_{cb} \) profiles for the higher-mode and first-mode region of floor spectra of the four-story baseline RCSW building. Similar graphs are presented in Figures 5-19(c) and (d) for the eight-story baseline RCSW. As seen the value of \( R_{cb} \) depends on the floor relative height and has its minimum value at the lowermost floors. Overall \( R_{cb} \) values of the first-mode region are larger than those of higher-mode region.
The results of similar analyses for the four-story and eight-story baseline SMRF building are presented in Figures 5-20.
5.5.6 Influence of the NSC damping ratio on elastic floor spectra

Isolated studies on specific structures have investigated the influence of NSCs viscous damping ratio on their seismic force demands (e.g., Singh et al. 2006; Sankaranarayanan and Medina 2007; Sullivan et al. 2013; Calvi and Sullivan 2014a; Vukobratović and Fajfar 2017). These studies, consistent with the fundamentals of structural dynamics, illustrated that the component damping ratio is an influential parameter on the NSCs elastic seismic force demand, especially for NSCs that are in tune with one of the modal periods of the supporting building.

In the ATC-120 project discussions, primarily by Miranda et al., ensued to develop NSCs design equations based on a component viscous damping ratio smaller than the prevalent 5%. The
present section examines the influence of the value of the component viscous damping on floor spectra of a representative baseline archetype building. A parameter denoted as damping modification factor (DMF_c) is evaluated to quantify the influence of NSC viscous damping on the seismic force demands of NSCs that are attached to elastic and inelastic buildings. For a linear SDOF oscillator, the terminology DMF_c can be defined as the ratio of the seismic force demand corresponding to a damping ratio of \( \xi_c \) to the force demand associated with the nominal 5% viscous damping ratio (see Equation 5-2).

Representative plots of roof spectra DMF_c for various \( \xi_c \) are depicted in Figure 5-21(a). Similar graphs are presented for a mid-high floor in Figure 5-21(b). As seen, with increasing the deviation of \( \xi_c \) from the default 5%, DMF_c values are more influenced by the value of \( \xi_c \), and contain sharper and more pronounced peaks. The influence of \( \xi_c \) on DMF_c is more pronounced for NSCs with periods near the modal periods of the supporting structure (i.e., tuning condition). For example, as seen in Figure 5-21(a), for \( \xi_c = 1\% \) the value of DMF_c at NSC period in the vicinity of the second mode of the supporting building (i.e., \( T_c/T_{b1} = 0.3 \)) is 2.1. This value is significantly larger than the magnitudes of DMF_c for other component periods that can even exhibit a value of 1.0. An evaluation of Figure 5-21 illustrates that for a given \( \xi_c \), the values of DMF_c are nearly constant for period ratios beyond 2.25. This implies that for this normalized period range, the DMF_c parameter is weakly dependent on the NSC period ratio. This observation reiterates the importance of the parameter \( T_c/T_{b1} \), where \( T_{b1} \) is the period of vibration of the \( i \)th mode of the supporting structure, in the quantification of DMF_c.

As can be observed from Figure 5-21, the behavior of DMF_c in the HM and FM regions is dependent on the location of NSC in the supporting building, or in other words, the contribution of different vibration modes. The transition from the HM region to the FM region is associated with a trough, and the effect of damping ratio on the magnitude and shape of DMF_c in this region is minimal, especially for lower-floor locations (see Figure 5-21b). As seen, at the lower-floor locations and in-between the two peaks of DMF_c corresponding to the first two modes of vibration of the supporting building, DMF_c values tend to unity, regardless of the value of \( \xi_c \).
Figure 5-21 DMF of the elastic floor spectra for the (a) roof; (b) 4th floor; level of the eight-story baseline SMRF building exposed to the DE level SC records

In studies that are in-progress by Anajafi et al. (Anajafi and Medina 2018a; Anajafi et al. 2018) buildings with various characteristics including different number of stories, lateral-load resisting systems, modal periods and levels of inelastic behavior, are used to develop prediction equations for floor spectra DMF. To the best of our knowledge, Miranda and his coworkers have been performing analyses using the responses of instrumented buildings to evaluate the effect of component viscous damping on NSCs seismic demands.

5.6 Development of a Generic Floor Response Spectrum using floor response spectra of the archetype buildings

In the seismic qualification testing of NSCs, the input ground motions are usually generated based on a Required Response Spectrum (RRS) provided by the International Code Council Evaluation Services Acceptance Criteria 156 (AC156 2010). The RRS is an elastic 5% spectrum. The equation to compute RRS in the constant acceleration region is the same as that of the ASCE 7-16 equivalent static equation (i.e., $\frac{PCA}{PGA} = 2.5[1 + 2z/h]$ with applying the upper limit of 4.0). Figure 5-22 presents the mean 5%-damped elastic spectra obtained at the roof level of all archetype buildings (baseline and overdesigned configurations) under the DE level SC records along with the RRS proposed by AC156 for the roof level. As seen, many individual computed roof spectra, especially in the constant acceleration region, exceed the RRS, whereas, a version of RRS without applying the upper limit of 4.0, can capture the maximum computed values for most of the studied buildings.
In many occasions when testing a NSC, no information is available regarding the detail of a component’s support and/or its attachment to the supporting building, which can significantly influence the NSC system’s period and damping. Furthermore, the future location (i.e., floor level) of attachment of NSCs might be unknown. Additionally, there are many situations in which the dynamic characteristics of neither the building nor the NSC are known or specified. Therefore, it might be justifiable to advocate for the use of a generic floor spectrum for testing NSCs that is neither building- nor component-specific. In other words, the target force (or acceleration) demand imposed on a NSC during the test should be a representative force for the entire family of NSCs of interest. A reasonable approach for dealing with this problem is to conduct a probabilistic analysis. For such an approach, floor motions from many reliable and realistic numerical building models with different characteristics are needed. The building sample should be large enough to present a fair representative of building inventory. This approach is numerically expensive and very challenging. The rest of this section presents a preliminary discussion for developing a generic floor response spectrum (GFRS) that is neither building- nor component-specific. Based on the simulation results of archetype buildings, the following empirical equation is proposed for the GFRS:

\[
S_{ac}/PGA = \begin{cases} 
2.0 & \text{for } T_c \leq 0.06 \text{ s} \\
66.7T_c - 2.0 & \text{for } 0.06 < T_c \leq 0.11 \text{ s} \\
6.0 & \text{for } 0.11 < T_c \leq 1.0 \text{ s} \\
\frac{1}{T_c} & \text{for } T_c > 1.0 \text{ s}
\end{cases}
\] (5.9)

Figure 5-22  Mean 5%-damped elastic spectra of the roof level of all baseline and overdesigned archetype buildings under the DE level SC ground motions and the AC156 Required Response Spectrum (w/o: without applying the upper limit of 4.0)
The normalized 5%-damped response spectra for all above-ground floors of all archetype buildings under the 20 SC records at the DE level are illustrated in Figure 5-23. Different statistical parameters including mean, 84th percentile, and 95th percentile are calculated over all floor response spectra. Given the fact that the 24 archetype buildings used in this study may not be considered fair representatives of the complete universe of buildings, the 95th percentile is selected to be a more reliable parameter for the evaluation of the proposed GFRS. As seen in Figure 5-23, the proposed GFRS envelops the 95th percentile values. To further verify the adequacy of this GFRS, response spectra obtained from the roof level of the instrumented buildings discussed in Chapters 2 and 4 are plotted in Figure 5-24(a). Figure 5-24(b) plots response spectra for the roof and bottom-half floor levels of the instrumented buildings. As seen, for these cases, the 84th percentile spectrum is in agreement with the proposed GFRS. It should be noted that the relatively large normalized spectral acceleration values observed in the responses of the instrumented buildings belong to the buildings that most likely responded elastically because they were exposed primarily to ground motions below the DE level (see Chapter 4).

![Figure 5-23](image)

*Figure 5-23 Evaluation of the proposed GFRS based on the 5%-damped elastic floor spectra for all floor levels of all studied archetype buildings under individual SC records*
5.7 Constant-ductility inelastic floor spectra

Significant uncertainties are present in the estimation of the floor acceleration motions that excite NSCs at their base, where base refers to the point of attachment of a NSC to the supporting building. These uncertainties are inherent in the characteristics of ground motion, supporting building and NSC itself (Miranda et al. 2018). In other words, in addition to the filtering effect of the soil profile underneath the structure, two additional filters, i.e., the supporting building and NSC itself can significantly influence the NSC seismic demands. Hence it might be more rational to control the NSC demands through incorporating a nonlinear seismic fuse into the NSC system, as the last part of a long and complicated chain (this chain includes fault, site effect, foundation, building and NSC) (Miranda et al. 2018). The ultimate goal of this approach is to reduce the dependence of the seismic forces on the characteristics of the supporting structure (Miranda et al. 2018).

The use of inelastic floor spectra for designing NSC dates back to 1980’ (see the Background section). Inelastic design of NSCs has been the ultimate goal of ASCE 7-16 $F_p$ equation, as it applied a component response modification factor, $R_p$, to reduce the elastic design forces. However, this the value of $R_p$ in ASCE 7-16 has been established based on engineering judgment rather than numerical or experimental studies. Recently, Miranda and his coworkers proposed a capacity design approach for designing the NSC system (i.e., component-attachments-anchors). In this approach, the attachment of NSCs to the supporting building is designed to undergo inelastic
actions, and the other elements of the NSCs (anchors and connections) are designed as force-controlled elements. In other words, attachments (e.g., bracings) are the weakest link in the chain that influences NSC seismic demands. Other parts of the NSC system are designed based on the force demand that corresponds to forming an inelastic mechanism in attachments considering the inherent overstrength of attachments. In their study, Miranda et al. (2018) used the recorded roof floor acceleration motions of 113 instrumented buildings in California. They developed constant-ductility inelastic spectra assuming ductility values of 1.0, 1.50 and 2.0 and a 2% viscous damping for NSCs.

Providing such inelastic fuses in many NSCs can be controversial and challenging. Examples of these NSC are piping systems, wall-anchored equipment, and any other NSCs that are directly attached to a building without a well-behaved bracing system. There are other types of NSC attachments that can provide a ductile behavior. For example, consider a fridge that is attached to a floor using steel angels. In this case, the angles’ inelastic deformation can provide considerable energy dissipation although it is very challenging to quantify this behavior. Complicated nonlinear mechanisms such as rocking, friction, sliding, and inelastic actions in anchors are other sources of seismic energy dissipation in NSCs. In this study, the focus is on “inelastic” behavior of NSCs. Further research including experiments is needed to quantify other NSC nonlinear mechanisms.

In this section, constant-ductility inelastic floor spectra are developed for different floor levels of the elastic and inelastic archetype buildings. The NSC viscous damping ratio is varied from 2 to 8%, and the NSC target ductility ranges from 1.0 to 8.0. A dimensionless parameter denoted as the component response modification factor, $R_{cc}$, is utilized to quantify the NSC force reduction due to its nonlinear behavior. Parameters that can influence the magnitude and behavior of $R_{cc}$, including the NSC ductility and damping, the floor relative height, the supporting building level of inelastic behavior, and ground motion type, are evaluated.

### 5.7.1 Forced vibration of SDOF systems subjected to harmonic base-excitations of different durations

Considering the pseudo-harmonic nature of the floor motions, an evaluation of the response of inelastic SDOF systems exposed to a simple harmonic excitation is an initial step toward understanding the effect of NSC nonlinearity on its seismic demands. Assume that a nonlinear
SDOF system with the natural frequency $\omega_n$, viscous damping ratio $\xi$ and zero initial conditions is exposed to a sinusoidal base excitation of frequency $\omega_b$ with $N$ cycles of loading. The base excitation is given by the equation $\ddot{u}_b(t) = \sin(\omega_b t)$, where $t = 2\pi N/\omega_b$. The SDOF inelastic responses for different numbers of loading cycles are studied next.

As an example, Figure 5-25(a) illustrates the constant-ductility spectra for the sinusoidal base excitation with 10 cycles of loading, assuming different ductility values and a constant viscous damping ratio of 2% for the SDOF oscillator. Figure 5-25(b) presents similar graphs for a viscous damping ratio of 5%. As seen, the SDOF ductile behavior can significantly mitigate the spectral acceleration responses, especially at the tuning condition. Figures 5-26(a) and (b) depict the response modification factor, $R_\mu$, for the inelastic spectra previously presented in Figure 5-25(a) and (b). An evaluation of Figure 5-26 reveals that (i) for a given $\mu$, the maximum value of $R_\mu$ (i.e., the maximum response reduction with respect to the elastic SDOF) is obtained at the tuning condition. (ii) For the non-tuning cases, especially for the short SDOF periods, $R_\mu$ approaches unity regardless of the value of $\mu$. (iii) The beneficial effect of ductility is more highlighted for a low-damped SDOF system. For example, for $\mu = 2.0$, the maximum value of $R_\mu$ for the viscous damping ratio of 2 and 5% is 10.3 and 6.2, respectively. These results at some extent are consistent with results of early studies of Newmark on earthquake ground motions (i.e., broad band excitations).

![Figure 5-25](image)

*Figure 5-25  Normalized constant-ductility spectra for a sinusoidal base-excitation with 10 cycles of loading, assuming a viscous damping ratio of (a) 2%; (b) 5%*
Figure 5-26  Response modification factor of the constant-ductility spectra for a sinusoidal base-excitation with 10 cycles of loading, assuming a viscous damping ratio of (a) 2%; (b) 5%

Figures 5-27(a) and (b) depict inelastic displacement ratio, $C_\mu$, for the sinusoidal base-excitation with 10 cycles of loading, assuming a viscous damping ratio of 2% and 5%, respectively. As seen, at the most critical region, i.e., tuning condition, the inelastic behavior of the SDOF system can significantly decrease the maximum displacement responses of the SDOF system with respect to the elastic case (see values below unity). For the longer periods, $C_\mu$ values tend to unity, regardless of the value of SDOF ductility (this is consistent with the well-known equal displacement principle for the ground spectra). These observations suggest that when a SDOF system is exposed to harmonic motions and is tuned to the dominant frequency of the excitation, its ductile design can be a superior approach. This conclusion reinforces the appropriateness of the inelastic design of floor-mounted tuned NSCs as they are excited by semi-harmonic motions.

However, Figure 5-27 illustrates that for the short SDOF period ratios, inelasticity can lead to a significant increase in the displacement responses. This is while, it was previously observed from the results shown in Figure 5-22 that the inelastic behavior cannot meaningfully decrease force demands on these SDOF systems. This observation suggests that short period ratio SDOF systems (NSCs) should be designed to remain in the elastic range. This issue is more discussed later. It is important to note that the issue of large ductility demands on short-period (rigid) inelastic structures under ground motion excitations has been addressed in many previous works (e.g., see Charney et al. 2012). However, one should be mindful that for the case of NSC, the expression “rigid” is a relative term and the rigidity of a NSC can be evaluated by knowing the ratio of the
NSC period to the modal periods of the supporting building. In other word, a given NSC can be regarded as rigid (i.e., less likely tuned) in a building but as flexible (i.e., more likely tuned) in another building. This is why in this study the expression “short-period ratio” is used in lieu of the term “short-period”.

![Diagram](image)

Figure 5-27  The inelastic displacement ratio for a sinusoidal base-excitation with 10 cycles of loading, assuming a viscous damping ratio of (a) 2%; (b) 5%

Figure 5-28(a) illustrates the variation of the response modification factor with the number of loading cycles for a SDOF system with a 2% viscous damping ratio subjected to a sinusoidal base excitation with a dominant frequency equal the SDOF frequency (i.e., tuning condition). Figure 5-28(b) presents similar graphs for the inelastic displacement ratio. As seen, with increasing the number of loading cycles, the value of $R_\mu$ asymptotically increases, whereas the value of $C_\mu$ asymptotically decreases. These results suggest that the beneficial effect of SDOF inelasticity for floor motion excitations is more significant than that for ground motion because the number of harmonic cycles in floor motions are larger than that in typical ground motions.
Figure 5-28  Variation of the (a) response modification factor; (b) inelastic displacement ratio; with the number of loading cycles for a tuned SDOF system with 2% viscous damping ratio subjected to a sinusoidal base excitation

5.7.2 Constant-ductility inelastic floor spectra for elastic and inelastic supporting buildings

This section utilizes mean inelastic floor spectral values to identify dependencies and behavioral trends. Figure 5-29(a) presents the mean normalized roof spectral acceleration values ($S_{ac}/PGA$) for the baseline two-story SMRF building assuming different NSC target ductility, $\mu_c$, values and a constant component damping ratio, $\xi_c$, of 5%. Figure 5-29(b) depicts the corresponding $R_{cc}$ spectra. Figures 5-29(c) and (d) present the mean displacement spectra and the corresponding inelastic displacement ratios, respectively. Similar graphs are shown in Figures 5-30 through 5-32 for the baseline eight-story SMRF, baseline two- and eight-story RCSW buildings, respectively. The floor spectra can be studied in five different period ratio regions: (i) NSC periods smaller than the considered buildings second-mode (i.e., period ratios smaller than 0.1) in which NSCs are essentially rigid; (ii) and (iii) NSC periods in the vicinity of the second- and first-mode of the supporting building; (iv) NSC periods in between the second- and first-mode of the building, respectively; (v) relatively long NSC period ratios (e.g., $T_c/T_{bl} > 1.5$).
The most salient conclusions from the evaluation of the floor spectral responses at the five previously mentioned period ratio regions are summarized next:

For the relatively short period ratios, the component inelastic behavior is not very effective in reducing spectral acceleration demands, and inelastic displacement responses adversely increase (see $C_{cc}$ values larger than unity). In the vicinity of the second mode, spectral acceleration values decrease when the NSC experiences inelastic behavior. This reduction is achieved at the expense of a slight increase in spectral displacement responses. For components with periods in the vicinity of the first mode, the component inelasticity not only significantly reduces spectral acceleration demands but also decreases the spectral displacement values. In the transition region (i.e., in between the first two modes), the component inelasticity can decrease the spectral acceleration values but at the expense of a relatively large increase in the spectral displacement demands. In
the long-period ratio region, the component inelasticity can decrease spectral acceleration responses by a factor that is fairly close to the component displacement ductility value (i.e., $R_{cc} \approx \mu_c$). In this period ratio region, the magnitude of $R_{cc}$ is weakly dependent on the value of NSC period ratio, and the inelastic displacement responses tend to the elastic displacement responses. These conclusions are consistent with observations pertaining to the well-known equal displacement rule in inelastic ground spectra. As another important observation, in the vicinity of the first mode of the building, $R_\mu$ values are more influenced by the value of $\mu_c$, and contain sharper and more pronounced peaks. An evaluation of Figures 5-30 to 5-32 consistently illustrates that for a $\mu_c$ larger than 3.0 no amplification is observed in spectral acceleration responses with respect to the peak floor acceleration response (i.e., spectral acceleration at a component period ratio of 0). This is consistent with the results of Viti et al. (1981). This observation suggests that designing NSCs for a $\mu_c > 3.0$ can simplify the design in such a way that only PFA should be determined from an inelastic response history analysis of the supporting building and be used to estimate force demands on NSCs as long as the design can accommodate the required component displacement and ductility demands.

For designing NSCs, in addition to the spectral acceleration demands (i.e., force demands), ductility and also displacement demands are of importance. Large ductility demands are associated with significant seismic damage that may impair the component’s functionality. Large displacement demands may cause pounding to the adjacent structural and nonstructural elements (i.e., component-building and component-component interactions may become more critical). An evaluation of the spectral displacement responses of the NSC at the roof level of the studied buildings illustrates that for buildings with a longer period (e.g., eight-story SMRF), spectral displacement responses in tuning conditions can be significantly large. For example, for the eight-story baseline SMRF building, in the vicinity of the first mode of the building, spectral displacement responses for the elastic and inelastic case with a target ductility of 1.5 can exceed 1.75 and 1.0 m, respectively. Hence, using relatively flexible NSCs that can be tuned to the modal periods of these long period building may be very challenging.
Figure 5-30  Characteristics of the inelastic roof spectra for the eight-story baseline SMRF exposed to the DE level SC records assuming a NSC damping ratio of 5%
Figure 5-31  Characteristics of the inelastic roof spectra for the two-story baseline RCSW exposed to the DE level SC records assuming a NSC damping ratio of 5%
Similar analyses are conducted assuming a NSC viscous damping ratio of 2%. The overall behavioral trends are similar to the result obtained when NSC viscous damping ratio was assumed to be 5%. Results illustrate that the influence the NSC inelasticity on the NSC seismic demands is more significant for a low-damping NSC.

5.7.3 Seismic design implication of the parameter $R_{cc}$

While using an inelastic floor spectrum can emphasize the importance of the NSC period, in many situations the period of the component-support-attachment system is unknown or at least associated with significant uncertainties. In this case, designing for the most critical NSC period, which is usually the tuning period, is a rational (but probably conservative) approach. Adopting this approach for an inelastic NSC scenario, needs the determination of the associated $R_{cc}$ value.
(i.e., $R_{cc}$ value corresponding to the critical NSC period). An evaluation of Figures 5-29 to 5-32 illustrates that with the transition of a NSC to the inelastic behavior, the first-mode peak value of the floor spectrum shifts to the left (i.e., to the $T_c/T_{b1}$ values less than unity). Therefore, a more rational approach for determining the component response modification factor, $R_{cc}$, is to compute the ratio of the maximum elastic spectral acceleration to the maximum inelastic acceleration over a given component period ratio range, even though the maximum values do not occur at the same component period ratios.

In this section, two modal period regions, namely higher-mode (HM) and fundamental-mode (FM) regions, are defined, and representative $R_{cc}$ values are introduced for each region. The HM region is considered as $0 \leq T_c/T_{b2} \leq 2.0$, and the FM region is defined as $T_c/T_{b2} > 2.0$. The component response modification factor for the HM period range is computed using the following equation:

$$
(R_{cc})_{HM} = \frac{\max(S_{ac})_{\text{elastic NSC}} \text{ over the HM region}}{\max(S_{ac})_{\text{inelastic NSC}} \text{ over the HM region}}
$$

A similar equation is used to estimate $R_{cc}$ for the FM region. Figures 5-33(a) to (d) illustrate the variation of $(R_{cc})_{HM}$ and $(R_{cc})_{FM}$ with the component ductility value, $\mu_c$, for the roof spectra of the studied archetype buildings assuming different values of component damping, $\xi_c$. As seen, at a given $\mu_c$ and $\xi_c$, the value $(R_{cc})_{FM}$ is significantly larger than the value of $(R_{cc})_{HM}$. For a given $\xi_c$, the value of $(R_{cc})_{FM}$ constantly increases with increasing $\mu_c$, whereas, $(R_{cc})_{HM}$ approaches an asymptotic limit value. Results also illustrate that the value of $R_{cc}$ varies from building to building. This variation is more significant for the FM region. In addition, the value of $(R_{cc})_{FM}$ for the short-period buildings is larger than that for the longer-period buildings. The reverse is true for the $(R_{cc})_{HM}$.
5.7.4 Influence of the NSC viscous damping on the estimated NSC inelastic seismic demands

Figure 5-34(a) illustrates elastic roof spectra assuming different NSC viscous damping ratios for the four-story baseline SMRF exposed to the DE level SC records. Figure 5-34(b) present the corresponding inelastic roof spectra (with a NSC ductility of 2.0). Figures 5-35(a) and (b) illustrate the parameter $DMF_c$ for the roof spectra shown in Figures 5-34(a) and (b), respectively. The results show that elastic spectra are much more sensitive to the value of NSC viscous damping. For example, the value of $DMF_c$ in the vicinity of the building second-mode for a NSC damping ratio of 2% in Figure 5-35(a), i.e., the elastic NSC case, is up to 1.6, whereas the corresponding value for inelastic NSC with a ductility of 2.0 in Figure 5-25(b) is limited to 1.2.
Figure 5-34  Mean roof spectra for the four-story SMRF baseline exposed to the DE level SC records (a) elastic NSC; (b) inelastic NSC with a target ductility of 2.0

Figure 5-35  Roof spectra’s mean $DMF_c$ for the four-story baseline SMRF exposed to the DE level SC records (a) elastic NSC; (b) inelastic NSC with a target ductility of 2.0

An evaluation of Figures 5-34(a) and (b) illustrates that the spikes in the vicinity of the building modal periods shift to the left when NSC becomes inelastic. Hence, an improved way of computing $DMF_c$ would be determining representative $DMF_c$ s for given period ratio regions as the one shown in Equation 5-11 for the higher-mode region:

$$(DMF_c)_{HM} = \frac{\text{max}[S_{ac\xi}]}{\text{max}[S_{ac5\%}]} \text{ over the HM region}$$

(5.11)
A similar equation can be used to determine \( (\text{DMF})_{\text{FM}} \). Figure 5-36 illustrates the variation of the roof spectra’s mean \( \text{DMF}_c \) of the higher-mode and fundamental-mode regions with the component target ductility for the baseline four-story SMRF exposed to the DE level SC records. As seen, for a component damping ratio of 2% when the component transitions to inelastic behavior the value of \( \text{DMF}_c \) for both higher-mode and fundamental-mode regions rapidly decreases and approaches a constant value that is fairly close to the square root of \( \text{DMF}_c \) for the elastic component case. For a component damping ratio of 8%, the effect of component inelastic on the value of \( \text{DMF}_c \) is not as significant as this effect for a component damping ratio of 2%.

![Figure 5-36](image-url)

**Figure 5-36**  Variation of the roof spectra’s mean \( \text{DMF}_c \) with the component target ductility for the four-story baseline SMRF exposed to the DE level SC records

### 5.8 Additional factors that influence the parameters \( R_{cc} \) and \( C_{cc} \)

Results of Section 5.7 illustrate that the component response modification factor is a function of the component target ductility, viscous damping and tuning ratio, as well as the supporting building period and the lateral-load resisting system type (recognized by \( T_{b2}/T_{b1} \)):

\[
R_{cc} = f (\mu_c, \xi_c, T_c/T_{b1}, T_{b1}, T_{b2}/T_{b1})
\]  \( (5.12) \)

This section investigates the other parameters that can be influential on the value of \( R_{cc} \). For the analyses conducted in this section, the NSC viscous damping ratio is fixed at 5%.

#### 5.8.1 Ground motion intensity level (or level of inelastic behavior of supporting structure)

The \( R_{cc} \) values discussed so far correspond to the design earthquake (DE) level at which the archetype buildings are expected to experience inelastic actions. However, if a building is
significantly over-designed or it is exposed to ground motions with intensities well below the DE, it can respond elastically. As illustrated in many previous studies (see e.g., Sewell 1986; Igusa 1990; Adam and Fotiu 2000; Rodriguez et al. 2002; Medina et al. 2006; Politopoulos and Feau 2007; Sankaranarayanan and Medina 2007; Adam and Furtmüller 2008; Chaudhuri and Villaverde 2008; Wieser et al. 2013; Petrone et al. 2015; Vukobratović and Fajfar 2015; Petrone et al. 2016; Anajafi and Medina 2018b), the supporting building inelasticity in many cases can partially mitigate detrimental narrow-band frequencies of floor motions. This behavior usually results in a reduction in the $S_{ac}/PGA$ values, especially near the fundamental period of the supporting building. In other words, one can conclude that lower intensity ground motions can yield larger $S_{ac}/PGA$ values. This section evaluates the sensitivity of the parameter $R_{cc}$ to the ground motion intensity level.

Figures 5-37(a) and (b) illustrate, respectively, roof floor spectra and their corresponding $R_{cc}$ spectra for the six-story baseline SMRF archetype building exposed to the SC records that are scaled to intensity levels ranging from 0.25 DE to 1.50 DE. In this figure, floor spectra for two component target ductility values of 1.0 and 2.0 are presented. Figures 5-38(a) and (b) depict similar graphs for the eight-story baseline RCSW archetype building. As consistently observed from all plots, with increasing the ground motion intensity level (i.e., transition of the supporting structure from elastic to inelastic behavior), $R_{cc}$ in the vicinity of the fundamental period exhibits a reduction. In other words, with increasing the level of inelastic behavior in the supporting building, detrimental narrow-band floor motions (i.e., semi-harmonic motions) are partially filtered out at this tuning NSC period. In Section 5.7.1 it was observed that the more dominant the harmonic characteristic of floor excitation, the larger the effect of component ductility. Therefore, with building inelasticity, the effect of component ductility on the floor spectral acceleration values near the building fundamental period decreases. However, the value of $R_{cc}$ in the vicinity of the second mode is weakly dependent on the ground motion intensity level or the level of inelastic behavior of the supporting building because higher modes experience lower levels of inelasticity than first mode.
Figure 5-37  Characteristics of the inelastic roof spectra for the six-story baseline SMRF exposed to the SC records with different intensities assuming a NSC damping ratio of 5%
Figure 5-38  Characteristics of the inelastic roof spectra for the eight-story baseline RCSW exposed to the SC records with different intensities assuming a NSC damping ratio of 5%.

Figure 5-39(a) and (b) illustrates the variation \( (R_{cc})_{HM} \) and \( (R_{cc})_{FM} \) with \( \mu_c \) for roof spectra of the six-story baseline SMRF and eight-story baseline RCSW building for different ground motion intensities. As seen, for all \( \mu_c \) values, \( (R_{cc})_{FM} \) is larger for a lower ground motion intensity (i.e., an elastic supporting building behavior) than for a higher intensity. However, the value of \( (R_{cc})_{HM} \) is not significantly affected by ground motion intensity (i.e., supporting building inelasticity).
5.8.2 The floor relative height

As previously observed in Section 5.5.4 and also results of many previous studies listed in the Section 5.3, relative height, RH (i.e., the ratio of the height above the base of the point of attachment of the component to the average roof height), has a significant effect on the shape and value of the floor spectra, where shape refers to the variation of spectral acceleration responses with the normalized component period. The current section investigates the effect of relative height on the amplitude of the $R_{cc}$ and $C_{cc}$ for two representative archetype buildings. The evaluation conducted in this section is based on a component target ductility and viscous damping ratio of 2.0 and 5%, respectively. The buildings are exposed to the DE level SC records.

Figures 5-40(a) and (b) respectively present the parameter $R_{cc}$ and $C_{cc}$ for different floor levels of the six-story baseline SMRF building including the ground level (i.e., RH = 0). Figure 5-41 illustrates similar plots for the eight-story baseline RCSW building. The behaviors of $R_{cc}$ and $C_{cc}$ are evaluated in various period regions (i.e., very short period ratios, periods in the vicinity of the building modal periods, periods in between the building modal periods and longer period ratios. As consistently observed in Figures 5-40(a) and 5-40(b), the lower floors (herein RH = 0.33) exhibit the largest $R_{cc}$ and lowest Figures 5-40(a) and (b) respectively present the parameter $R_{cc}$ and $C_{cc}$ for different floor levels of the six-story baseline SMRF building including the ground level (i.e., RH = 0). Figure 5-41 illustrates similar plots for the eight-story baseline RCSW building. The behaviors of $R_{cc}$ and $C_{cc}$ are evaluated in various period regions (i.e., very short
period ratios, periods in the vicinity of the building modal periods, periods in between the building modal periods and longer period ratios. As consistently observed in Figures 5-40(a) and 5-40(b), the lower floors (herein $RH = 0.33$) exhibit the largest $(R_{cc})_{HM}$ and smallest $(C_{cc})_{HM}$ values. The largest value of $(R_{cc})_{FM}$ and smallest value of $(C_{cc})_{FM}$ occurs at the roof level. By the definition, $R_{cc}$ and $C_{cc}$ express the reduction in spectral ordinates due to the NSC inelastic behavior. Therefore, observations of 5-40(a) and 5-40(b) illustrate that the fundamental-mode floor spectral values near the roof are most influenced by the NSC inelastic behavior than those at lower locations. The sensitivity of fundamental-period spectral values to NSC inelasticity at the top floors comes primarily from the predominant contribution of the first mode to the roof response. The same interpretation is true for the higher-mode floor spectral values at the lowermost levels and the contribution of higher modes at these locations.

Results show that the beneficial effects of NSC inelasticity for NSC in tune with the building modal periods, especially the first-mode, periods in above-ground floor spectra are significantly larger than that for any period in ground spectrum. For example, at NSC period in the vicinity of the six-story SMRF building first-mode, the $R_{cc}$ value for the roof spectrum is 4.5 whereas the maximum value of $R_{cc}$ for the ground spectrum is 2.1. The corresponding $C_{cc}$ values for the roof and ground spectra are 0.5 and 0.9 respectively. This observation is consistent with results of Section 5.7.1 wherein it was shown that the beneficial effect of NSC inelasticity increases with an increase in the number of loading cycles (or the dominant the harmonic characteristic of the floor motion). For NSC period ratios in between the building modes, $R_{cc}$ has its minimum value and is significantly smaller than the values of $R_{cc}$ in ground spectrum. At these NSC period ratios, the $C_{cc}$ values at above-ground floors are larger than unity (i.e., an adverse effect of NSC inelasticity). For short period ratios, $R_{cc}$ values at all relative heights including ground and roof levels tend to unity meaning that NSC inelasticity is not effective in reducing force demands on these NSCs. In these cases, $C_{cc}$ values become adversely larger. At the long-period ratio region, the $R_{cc}$ values for all floor levels approaches the $R_{cc}$ of the ground spectra that is almost equal to $\mu_c$. For these NSC period ratios, the value of $C_{cc}$ at all floor levels tends to unity. The two latter observations are consistent with the equal displacement rule.
5.8.3 Ground motion excitation type

In this section, the FEMA-P695 far-field ground motion set (denoted as FF herein), which includes 44 individual records, is used to evaluate the dependence of the parameters $R_{cc}$ and $C_{cc}$ on the type of the ground motion set used in analyzing the supporting building. The FF ground motions are
amplitude-scaled in accordance with the ASCE 7-16 provisions. In this section, results are presented for NSC target ductility values of 1.0 and 2.0 and a fixed NSC damping ratio of 5%.

Figure 5-42(a) illustrates the mean 5%-damped roof spectra for the six-story baseline SMRF building computed based on the floor motions obtained from the simulation of the structure under SC and FF record suites. In this figure, roof spectra for two NSC ductility values of 1.0 and 2.0 are presented. Figures 5-42(b) to (d) depict the corresponding mean $R_{cc}$, $S_{dc}$ and $C_{cc}$ spectra, respectively. Figures 5-43(a) to (d) present similar graphs for the eight-story baseline RCSW building. As observed, the overall shapes of roof acceleration and displacement spectra for the two record suites are analogous for the entire floor spectrum although their magnitudes are different. The amplitudes of the parameter $R_{cc}$ for the two record sets are fairly close in the entire spectrum period range, especially in the vicinity of the modal periods of the supporting buildings, which tend to govern the maximum value of these parameters. The same statement is true for the parameter $C_{cc}$. These observations illustrate that mean value of $R_{cc}$ and $C_{cc}$ is weakly dependent on the type of ground motion set used for analyzing the supporting buildings.
Figure 5-42 Characteristics of the inelastic roof spectra for the six-story baseline SMRF exposed to the DE level SC and FF records a NSC damping ratio of 5%
5.9 A brief summary and an evaluation of ASCE 7-16 $F_p$ equation

5.9.1 The concept of capping floor spectral ordinates

In many studies, the maximum value of the (elastic) floor spectral acceleration responses, denoted as peak component acceleration, PCA, is considered for the evaluation of NSC design equations. This is while the elastic PCA values occur in a relatively narrow-band period region, as shown in Figure 5.44 for the roof level of the studied archetype buildings. In other words, these large PCA responses only affect NSCs very close to tuning, and with a relatively small detuning these values are no longer observed. Therefore, designing NSCs for such demands might be significantly conservative. To take into account this fact in design, in the ATC-120 project, a decision was made...
to cap the floor spectral ordinates at a specific value. Other reasons for the capping were stated as following (i) the limited amount of time that the building and a NSC are likely to be in tune, (ii) the likely reduction that more realistic hysteresis loops than those used in research analyses would show, (iii) changing of both building and NSC periods during the earthquake event (NIST GCR 18-917-43).

To determine a force level at which the floor spectra can be capped, a database of shake-table tested NSCs through the OSHPD OSP program (which will be discussed later in this section) was used. The fundamental period of these tested NSCs and instrumented building-directions introduced in Chapter 2 of this dissertation were evaluated to estimate the likelihood of a NSC being in resonance with one of the first three modes of its supporting building, assuming neither the NSC period nor the building periods are known. It was accepted to have a 10% probability of forces exceeding those predicted by equations. It was shown that there is about 10% probability of a flexible NSC being in the range $0.85 \leq T_c / T_{bl} \leq 1.15$. In other words, the NSC period ratio region $0.85 \leq T_c / T_{bl} \leq 1.15$ was shown to be associated with 10% probability of exceeding the cap value. The 84th percentile of 5%-damped roof spectra of a group of instrumented building was considered as the baseline spectrum for predicting the design equations, and the cap was established at the force level near the force values associated with the tuning ratios of 0.85 and 1.15 (NIST GCR 18-917-43). The capping was applied only to elastic NSC and inelastic NSCs with a target ductility of 1.25, for which the floor spectra spikes are sharp. Table 5-2 illustrates the $PCA/PFA$ values before and capping.
Table 5-2  Design PCA/PFA values proposed by ATC-120 Project for flexible NSCs at Roof and suspended floors (NIST GCR 18-917-43)

<table>
<thead>
<tr>
<th>NSC category</th>
<th>Assumed NSC ductility</th>
<th>Peak value of $S_{ac}/PFA$ from the 84th percentile</th>
<th>Reduction from capping</th>
<th>PCA/PGA used for design equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic</td>
<td>1.0</td>
<td>5.6</td>
<td>1.4</td>
<td>5.6/1.4=4.0</td>
</tr>
<tr>
<td>Low</td>
<td>1.25</td>
<td>3.4</td>
<td>1.2</td>
<td>3.4/1.2=2.8</td>
</tr>
<tr>
<td>Moderate</td>
<td>1.5</td>
<td>2.2</td>
<td>Not applied</td>
<td>2.2</td>
</tr>
<tr>
<td>High</td>
<td>≥2.0</td>
<td>1.4</td>
<td>Not applied</td>
<td>1.4</td>
</tr>
</tbody>
</table>

The approach developed by ATC-120 is utilized in the present study when evaluating the proposed NSC design equations. Figure 5-45(a) illustrates the 5%-damped elastic roof component amplification factor, $S_{ac}/PFA$, computed for the roof level of all (baseline and overdesigned) SMRF and RCSW archetype buildings in the vicinity of the third mode on the buildings (it should be noted that in this figure, for each response spectrum, NSC periods are normalized to the third mode period of the corresponding supporting buildings). Figures 5-45(b) and 5-45(c) present similar graphs for the second-mode and first-mode region of the floor spectra. As seen capping reduces the PCA/PFA response from 3.3 to 2.7 (i.e., 18% reduction), from 5.0 to 3.3 (i.e., 34% reduction) and from 5.6 to 4.1 (i.e., 27% reduction), in the third-mode, second-mode and first-mode region of the floor spectra.

![Graphs](image)

Figure 5-45  The 5%-damped elastic roof component amplification spectra for the baseline and overdesigned SMRF and RCSW archetype buildings exposed to DE level SC records

### 5.9.2  Practical range of fundamental period of typical mechanical an electrical NSCs

In most previous studies, the maximum values of spectral acceleration responses over the entire spectrum periods are introduced as the PCA response and used for the evaluations. This is while modal periods of tall buildings, at which PCA might occur, could be well beyond the period range of typical NSCs. Therefore, such an approach could be significantly conservative. This section uses the results of testing on NSCs to estimate the most practical range of NSC period.
One of the largest numbers of qualification testing in the commercial building industry has been performed for California hospitals, due to the enforcement of code requirements by OSHPD. Different types of NSCs have been tested as per AC156. During these tests, NSCs are first subjected to low level broadband or sine sweep input motions to determine their natural frequencies. The tested NSCs include elevator equipment, chillers, equipment with hazardous contents, cooling towers, transformers, electrical substations, air handling units, motorized surgical lighting systems, exhaust and smoke control fans (a complete list of the tested NSCs can be found in (NIST GCR 17-917-44 2017). A histogram showing the relative distribution of NSCs horizontal natural frequency, which was developed based on test data available from shake table testing of equipment for the OSHPD OSP program, is illustrated in Figure 5-46(a) and (b) for fixed-based and isolated mounted NSCs. In these figures, the fundamental frequency for the two primary axes (side-to-side and front-to-back) of tested components are included. Frequency data is provided for a total of 1,115 mechanical and 540 electrical units under test (UUTs). Items within the categories of architectural NSCs were not included in this program.

![Histogram showing the relative distribution of NSCs horizontal natural frequency](image)

(a) NSCs rigidly mounted on the floor  
(b) NSCs with spring isolators mounted on the floor

**Figure 5-46** The horizontal natural frequency distribution of a unit under test (UUT) developed based on test data available from OSHPD OSP program (NIST GCR 17-917-44 2017).

The distribution shown in Figure 5-46 is used to herein to roughly estimate the lower and upper limits of fundamental periods of typical architectural and mechanical NSCs. It is important to note that only the largest and smallest units of a product family are usually tested to obtain an OSHPD OSP preapproval. Therefore, the data shown in Figure 5-46 is representative of the NSCs with lowest and highest frequencies of a family, and it does not depict the actual distribution of the universe of NSCs frequencies. According to Figure 5-46(a) and (b), the fundamental period of 91 and 99% of the UUTs is less than or equal to 0.25 s (4.0 Hz) and 0.5 s (2.0 Hz), respectively. Only
three units exhibit a natural period larger than 1.0 s (1.0 Hz). Based on these observations and engineering judgment, four NSC period ranges are demarcated for this study: (i) $T_c \leq 0.5$ s; (ii) $0.5 < T_c \leq 1.0$ s; (iii) $1.0 < T_c \leq 2.0$; (iv) $T_c > 2.0$ s. The maximum value of the mean normalized spectral acceleration responses, $S_{ac}/PGA$, over a given NSC period range is selected as the representative $PCA/PGA$ for that particular NSC period range. From results of Figure 5-46, it can be conservatively postulated that typical mechanical and electrical equipment situate in the period range 0-1.0 s.

The mean elastic 5%-damped $PCA/PGA$ profiles for the baseline SMRF buildings exposed to the DE level SC records are shown in Figure 5-47 assuming different NSC period ranges. As seen, some of relatively larger $PCA/PGA$ responses belong to the relatively large NSC periods of 1.0-2.0 s, that is beyond the period range of typical NSCs. Similar graphs are shown in Figure 5-48 for the baseline RCSW buildings. In the rest of this section, $PCA$ is considered to be the maximum value of a floor spectrum over the period range 0-1.0 s.

![Computed PCA/PGA profiles for the baseline SMRF buildings exposed to DE level SC records assuming different NSC period ranges (an elastic NSC with a 5% damping)](image)

Figure 5-47  Computed PCA/PGA profiles for the baseline SMRF buildings exposed to DE level SC records assuming different NSC period ranges (an elastic NSC with a 5% damping)
Figure 5-48  Computed PCA/PGA profiles for the baseline RCSW buildings exposed to DE level SC records assuming different NSC period ranges (an elastic NSC with a 5% damping)

5.9.3 Evaluation of ASCE 7-16 $F_p$ equation assuming an elastic NSC

This section summarizes the results of evaluations conducted on the ASCE 7-16 $F_p$ equation. It is assumed that NSCs are elastic with a 5% viscous damping ratio. The spectral acceleration responses obtained from all archetype buildings exposed to the DE level SC records are used. The ASCE 7-16 equation is evaluated based on the responses of individual buildings. In other words, the statistical analysis is not performed across different building responses. As a result, dispersion in responses are not as significant as those shown in Figures 5-45, and hence, the mean responses under the SC records are used for the evaluation. The evaluation is conducted with and without applying the capping over the NSC period ratio region $0.85 \leq T_c / T_{bl} \leq 1.15$. Only NSC periods smaller than 1.0 s (i.e., the practical range of typical electrical and mechanical NSCs based on Section 5.9.2) are considered. Figure 5-49(a) illustrates the evaluation of the ASCE 7-16 structure amplification factor. As seen, the ASCE 7-16 estimate is consistently higher than the computed responses for all archetype buildings and all floor levels. Figures 5-49(a) and 5-49(b) illustrate the evaluation of the normalized $PCA$ responses without and with the discussed cap. As shown many individual data points exceed the ASCE 7-16 estimation for normalized $PCA$
demand. As an important observation, the capping has reduced the maximum value of \( \frac{P_{CA}}{P_{GA}} \) from 12.0 to 8.0.

![Graph](image)

(a) in-structure amplification factor  
(b) component amplification factor without capping  
(c) component amplification factor with capping

*Figure 5-49 Evaluation of ASCE-7 \( F_p \) equation’s amplification factors using the responses of baseline and overdesigned archetype buildings exposed to the DE level SC records (elastic NSC with 5% damping).*

### 5.9.4 Evaluation of \( R_p \) factor provide by ASCE 7-16 \( F_p \) equation

The inelastic design of NSCs based on a target ductility implies using the displacement-based design approach. The current ASCE 7-16 provisions, designing buildings and NSCs are based on the force-based design approach. In these provisions, the seismic force demands are computed through an elastic design approach. Then a response reduction factor is applied to the elastic force demands to account for the energy dissipation through the inelastic mechanisms such as hysteretic behavior of materials. In the current ASCE 7-16 \( F_p \) design equation, a component response modification factor, \( R_p \) (which is equivalent to \( R_{cc} \) evaluated in this study), is used to primarily account for NSC inelasticity. The value of \( R_p \), which varies in a wide range of 1.0 to 12.0 for different NSC types, has been prescribed based on engineering judgment rather than experimental and numerical models (it should be noted that \( R_p \) larger than 6.0 are used in ASCE 7-16 for distributed NSCs that are not in the scope of this research).

At present, there is no clear understanding of the consequences of designing NSCs using these \( R_p \) factors. In other words, the ductility and displacement demands associated with these \( R_p \) values have not been quantified. This is particularly important given than for rigid NSCs (i.e., NSCs with an associated of \( a_p =1.0 \)) the use of an \( R_p \) is allowed in ASCE 7-16 provisions. This section evaluates the ductility and displacement demands on NSCs designed for different values of \( R_p \).
this end, constant-$R_{cc}$ inelastic spectra at the roof level of four representative archetype buildings under the DE level SC records are developed and discussed. For the study conducted in this section, viscous damping ratio of NSCs is assumed to be 5%. In Section 5-8 it was observed that with a reasonable level of component ductility (e.g., 2 to 4.0), no meaningful reduction is achieved in the spectral acceleration response of rigid (or very short-period ratio) NSCs. Constant-$R_{cc}$ spectra inelastic can better clarify the consequences of designing inelastic rigid NSCs.

Figure 5-50(a) illustrates the 5%-damped constant-$R_{cc}$ inelastic roof spectra for the two-story baseline SMRF building exposed to the DE level SC records assuming different component response modification factor, $R_{cc}$, values ranging from 1.0 to 4.0. Figures 5-50(b) to (d) depict the associated component ductility demands, $\mu_c$, displacement demands, $S_{dc}$, and inelastic displacement ratios, $C_{cc}$, respectively. Figures 5-51 through 5-53 present similar graphs for the eight-story baseline SMRF, two- and eight-story baseline RCSW buildings, respectively.

An evaluation of $\mu_c$ and $C_{cc}$ values consistently reveals that (i) for NSCs with period larger than the fundamental period of the supporting buildings, the value of $\mu_c$ is weakly dependent on the NSC period and is close to the value of $R_{cc}$. For example, for component tuning ratios larger than 1.5, the value of component ductility demand associated with an $R_{cc}$ of 4.0 remains nearly constant an equal to 3.5. For this NSC period range, the inelastic displacement tends to the elastic displacement regardless of the value of $R_{cc}$. These observations are consistent with the equal displacement principle for ground spectra, which has been extensively studied in the literature. (ii) For NSCs tuned to the fundamental mode of the supporting building, with increasing the value of $R_{cc}$, not only $\mu_c$ remains in a reasonable range (e.g., limited to 2.0 for an $R_{cc}$ of 4.0 for the two-story SMRF shown in Figure 5-50b) but also inelastic displacement ratios decrease (e.g., for an $R_{cc}$ of 4.0, the value of $C_{cc}$ is 0.5 in Figure 5-50d). (iii) For NSCs tuned to the higher modes of the supporting buildings, with increasing the value of $R_{cc}$, the value of $\mu_c$ (and also $C_{cc}$) increases. For example, for a NSC in tune with the second mode of the two-story SMRF building, for an $R_{cc}$ value of 4.0, the value of $\mu_c$ exceeds 10.0 and its corresponding $C_{cc}$ is larger than 3.50. This latter observation combined with the observation made in (ii), suggests that NSC inelasticity is more effective for NSCs tuned to the supporting building fundamental mode. (iv) for NSC periods in between the modal periods of the supporting buildings, component inelasticity is associated with
relatively large ductility and displacement demands. This is because, e.g., according to Figure 5-50(a), the yield strength level (i.e., the elastic spectral acceleration) in this region is already relatively low, and a further reduction of the yield strength by an $R_{cc}$ factor is associated with significant drawbacks. (v) For NSCs with relatively small normalized periods (e.g., smaller than the two-story SMRF building second-mode wherein NSCs essentially behave as rigid for this specific building), using an $R_{cc}$ as low as 1.50 is associated with an $\mu_c$ value as large as 20.0. For larger $R_{cc}$ values, the value of $\mu_c$ excessively increases. Similar trends are observed in the values of $C_{cc}$. These observations suggest that allowing rigid NSCs to experience inelastic behavior has the potential of resulting in significant, and in most cases unrealistic, displacement ductility demands. This is consistent with the results of many studies on rigid (short period) SDOF systems under ground motion excitations (e.g., see Charney et al. 2012).

![Figure 5-50](image-url)

(a) Mean normalized spectral acceleration
(b) Mean NSC ductility demand
(c) Mean spectral displacement
(d) Mean inelastic displacement ratio

*Figure 5-50  Characteristics of constant- $R_{cc}$ inelastic spectra for for the roof level of the two-story baseline SMRF building exposed to DE level SC records assuming a NSC damping of 5%*
(a) Mean normalized spectral acceleration
(b) Mean NSC ductility demand
(c) Mean spectral displacement
(d) Mean inelastic displacement ratio

Figure 5-51  Characteristics of constant- \( R_{cc} \) inelastic spectra for the roof level of the eight-story SMRF building exposed to DE level SC records assuming a NSC damping of 5%
Figure 5-52  Characteristics of constant- $R_{cc}$ inelastic spectra for the roof level of the two-story RCSW building exposed to DE level SC records assuming a NSC damping of 5%. 
Results of this section suggest that rigid NSCs should be designed to undergo inelastic behavior. Although the absolute displacement demands on inelastic rigid NSCs are not significant, the imposed ductility demands could be excessively large. It would be very challenging or even impractical to accommodate these large ductility demands and avoid significant component damage, and in many cases, maintain functionality.

5.9.5 Summary

The results presented in this dissertation regarding the evaluation of ASCE 7-16 $F_p$ mostly corroborate results of many previous studies available in the literature that when using generic frames reported potential shortcomings associated with the ASCE 7 $F_p$ equation. This dissertation also presents a few new findings regarding the influential parameters on NSCs seismic demands and also shortcomings of this equation (i.e., the importance of 3D effects discussed in Chapter 2,
the drawbacks of using $R_p$ for rigid NSCs, and the quantification of the influential parameters using the responses of code-based design buildings that was seldom addressed in the literature).

In has been shown in this dissertation (and many previous studies) that two substantially different filters influence demands on NSCs. The ground excitation at the base of a building is first filtered by the supporting building in such a way that the characteristics of the induced motions (i.e., floor acceleration motions) at the base of NSCs are substantially different than those of the ground motion. The second filter is the NSC itself whose characteristics can amplify or decrease floor acceleration motions. The flowchart provided in Figure 5-54 illustrates a schematic view of these two filters. As seen, the filtering effect of the supporting building is governed, at different extents, by many different and complicated parameters. The second filter (i.e., NSC), is characterized by tuning ratio, level of inelasticity and viscous damping. The way that ASCE 7-16 incorporates these two filtering effects is provided next.

The ASCE 7-16 $F_p$ equation can be written in the following normalized format:

$$\frac{PCA}{PGA} = \frac{PFA}{PGA} \times \frac{PCA}{PFA} = \left(1 + \frac{2z}{h}\right) \times \frac{a_p}{R_p} \times I_p$$

$$0.75PGA \times I_c \leq \frac{PCA}{PGA} \leq 4.0PGA \times I_c$$

NSC response modification factor varying from 1.0 to 12.0

As seen, assuming an importance factor of unity, the seismic design acceleration on NSCs is a simple multiplication of the in-structure amplification factor, $(1 + 2z/h)$, and $(a_p/R_p)$ that is essentially the amplification factor caused by the component. This means that except for the “floor level” none of the parameters controlling the filtering effect of the supporting buildings is explicitly incorporated into the current ASCE 7-16 $F_p$ equation. Furthermore, as shown in this study, the estimation of the ASCE 7-16 for the in-structure amplification factor is consistently higher than the computed in-structure amplification factors for the archetype buildings exposed to the DE level SC records.
The ASCE 7-16 $F_p$ equation uses the term $a_p/R_p$ to incorporate the effect of NSC tuning, damping and ductility, and inherent component overstrength factor generated in the design process, although in the ASCE 7-16 commentary discussions are not provided as to how these effects are combined (personal communication with Bachman, R. E.). In the Introduction section of this chapter, this issue was discussed in more detail. In the present study, it is shown that applying $R_p$ for rigid components can result in significantly large ductility demands. Unlike the current ASCE 7-16 approach, $a_p/R_p$ changes along the building height. This parameter also depends on component tuning ratio, and the ASC 7-16 threshold of 0.06 s (that is an absolute value) warrants modification.

5.10 The ATC-120 proposed equations for designing acceleration sensitive NSCs

Alternative equivalent static equations were proposed for designing acceleration-sensitive NSCs as part of a recent project sponsored by the Applied Technology Council (ATC-120 project). The effects of the supporting building characteristics, specifically inelastic behavior and fundamental period, and the NSC inelasticity were incorporated into the design equations. The present section discusses the basis for developing these equations and apply them to compute design PCA values.
for different archetype buildings studied. Later in Section 5.11, results of evaluation of the adequacy of these equations (many conducted in a period post-completion of the data-generation component of the ATC-120 project), using the responses of baseline and overdesigned archetype buildings exposed to the DE level SC records, are presented.

### 5.10.1 Introducing the ATC-120 proposed equations

In the ATC-120 Project, it was recognized that the force demand on NSCs is as a function of the parameters listed below (NIST GCR 18-917-43):

\[
\frac{F_c}{W_c} = f \left( \frac{PGA}{SFRS}, T_{bn}, \mu_b, \xi_b, IRR, DIA, \frac{z}{h}, T_c, \mu_c, \xi_c, \Omega_{oc} \right) \times I_c \quad (5.13)
\]

where \(PGA\) is the ground shaking intensity; \(SFRS\) is the seismic force-resisting system of the building; \(T_{bn}, \mu_b\) and \(\xi_b\) are the supporting building’s \(n\)-th modal period, ductility demand, and inherent viscous damping, respectively. \(IRR\) stands for the building configuration (such as plan and vertical irregularities). The term \(DIA\) refers to the floor diaphragm [in-plane] rigidity; \(z/h\) is the ratio of the vertical location of NSC within the building with respect to the building overall height. \(T_c, \mu_c, \xi_c\) and \(\Omega_{oc}\) are NSC period, ductility, inherent viscous damping and overstrength factor, respectively.

It should be noted that in the ATC-120 document, instead of the subscripts “c” used in this chapter for the nonstructural component in the proposed equations, the subscript “comp”, and instead of the subscript “b” for the supporting building, the subscript “bldg” was used.

In this project, the effects of different parameters on NSC seismic demands were identified and quantified using the responses of a wide variety of instrumented and archetype buildings. Attempted was made to refine Equation (5.13) through eliminating the parameters that are not significantly influential on NSC seismic demands. For example, it was shown that the inherent building damping at the DE level has a relatively small effect on the maximum value of NSC acceleration responses over the entire floor spectrum range. Some parameters were identified as important, however, because of the complexity and lack of information were taken out from Equation (5.13). For example, it was acknowledged that the in-plane floor diaphragm flexibility and building torsional responses can significantly influence NSC demands. However, “given the complexity of the issue, it was decided not to include the effects of diaphragms [and torsional...
responses] in the proposed nonstructural design equation”. For NSCs, various literature sources were investigated, and it was found that inherent viscous damping in NSCs could be less than 5%. Therefore, in the initially proposed equation, a term denoted as $B_\beta_{\text{comp}}$ was incorporated to account for NSC viscous damping ratios other than 5%. In the next step, a 2% inherent component damping was tentatively selected as the underlying basis of the NSC design equation. However, because of the several concerns (e.g., the insufficient research on viscous damping for many NSC, particularly at high shaking levels of interest, and the issue that use of 2% is inconsistent with the underlying 5% assumption of the equations in ASCE/SEI 7-16 Chapter 15 for nonbuilding structures), it was decided to continue with the traditional assumption of 5% inherent component damping (NIST GCR 18-917-43). The need for additional research, and account for potentially lower levels of damping in some components (e.g., rigid mechanical equipment) was highlighted in this document. Finally, the following equations were proposed for designing acceleration sensitive NSCs (NIST GCR 18-917-43):

$$\frac{F_c}{W_c} = PGA \times \left[ \frac{PFA/PGA}{R_{\mu b}} \right] \times \left[ \frac{PCA/PFA}{R_{poc}} \right] \times I_c$$  \hspace{1cm} (5.14a)

$$0.75PGA \times I_c \leq \frac{F_c}{W_c} \leq 5.0PGA \times I_c$$  \hspace{1cm} (5.14b)

It should be noted that Equation (5.14b) presents the lower and upper limits for Equation (5.14a). As seen, the proposed equation is essentially multiplication of four terms: (i) $PGA$ that is the peak ground acceleration at the DE level, equal to $0.4S_{DS}$. (ii) $I_c$ that is the component importance factor. (iii) $(PFA/PGA)/R_{\mu b}$ that is essentially the in-structure amplification factor (i.e., amplification of the maximum upper floor acceleration with respect to the ground). (iv) $(PCA/PFA)/R_{poc}$ that is the component amplification factor. The in-structure and component amplification factors are described in more details next.

In the in-structure amplification factor the term $PFA/PGA$ is the ratio of the maximum floor acceleration to the maximum ground acceleration assuming that the building responds elastically. The $PFA/PGA$ presented in Equation (5.14a) is computed using the following equation (NIST GCR 18-917-43):

$$\frac{PFA}{PGA} = 1 + a_1 \left( \frac{z}{h} \right) + a_2 \left( \frac{z}{h} \right)^{10}$$  \hspace{1cm} (5.15)
where \( a_1 = \min(1/T_{ab}, 2.5) \) and \( a_2 = \max(1 - (0.4/T_{ab})^2, 0) \). Note that \( T_{ab} \) is the approximate fundamental translational period of the building per ASCE/SEI 7-16 Equation 12.8-7. Equation 5.16 was derived based on a study conducted on instrumented buildings in the US. Given that these buildings most likely responded elastically, the term \( R_{\mu b} \), which is equivalent to the term \( R_{PFA} \) used in this dissertation, was incorporated to account for the effect of building nonlinearity. In other words, the term \( R_{\mu b} \) is the floor acceleration reduction factor, which accounts for the effect of building global ductility on the in-structure amplification factor, and is equal to \((1.1R/\Omega_0)^{1/2}\) where \( R \) and \( \Omega_0 \) are the Response Modification Coefficient and the Overstrength Factor of the supporting building from ASCE/SEI 7-16 Table 12.2-1. The value of \( R_{\mu b} \) needs not be taken as less than 1.0. As seen, the proposed approach incorporates the effect of building inelasticity on NSCs seismic force demands through modifying the PFA response. The discussions presented in the previous sections of this chapter reveal that the effect of building nonlinearity on the PCA responses is still present, but it is different than that on the PFA responses because PCA responses are more dominated by the building modes of vibration than the PFA. However, a decision was made as part of the ATC-120 project to use the proposed format of the \( F_c/W_c \) because it was believed to be easier to understand and implement in engineering practice.

The component amplification factor, \( PCA/PFA \), is incorporated to account for the component flexibility, inherent viscous damping, and ductility, and NSC vertical location (see Table 5-3 for the values of component amplification factor for different NSC scenarios). As seen, the proposed \( PCA/PFA \) values for component ductility of 2.0 and larger is the same for all floor levels.

Lastly, \( R_{ poc } \) is the inherent component overstrength factor and is assumed to be 1.3 (based on engineering judgment). This factor is incorporated to account for the fact that NSCs, similar to the supporting buildings, are generally overdesigned in practice. This factor can reduce the design forces to a lower level and counteract the effect of overdesigning in practice.

An evaluation of the proposed \( PCA/PFA \) values presented in Table 5-3 for elastic and inelastic NSCs can illustrate the component response modification factor, \( R_{ce} \), inherent in this approach. For example, for a tuned elastic roof-mounted NSC, the proposed \( PCA/PFA \) value is 4.0, whereas the corresponding value for a NSC with a target ductility of 2.0 is 1.4. This implies an
\( R_{cc} = 4/1.4 = 2.9 \). The corresponding \( R_{cc} \) value at the ground level is \( 2.5/1.4 = 1.8 \). Multiplying these values by \( R_{poc} \) illustrates the overall \( R_{comp} \) incorporated into the proposed design equations.

<table>
<thead>
<tr>
<th>Location of Component</th>
<th>Possibility of Being in Resonance with Building</th>
<th>Component Ductility Category</th>
<th>Assumed ( \mu_c )</th>
<th>( PCA/PFA )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>More Likely (^1)</td>
<td>Elastic</td>
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<td>2.5</td>
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<tr>
<td></td>
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<td>2.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Moderate</td>
<td>1.5</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>High</td>
<td>( \geq 2 )</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>Less Likely</td>
<td>Any</td>
<td>--</td>
<td>1.0</td>
</tr>
<tr>
<td>Roof or Elevated Floor</td>
<td>More Likely</td>
<td>Elastic</td>
<td>1</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Low</td>
<td>1.25</td>
<td>2.8</td>
</tr>
<tr>
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<td>1.5</td>
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<td>( \geq 2 )</td>
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<tr>
<td></td>
<td>Less Likely</td>
<td>Any</td>
<td>--</td>
<td>1.0</td>
</tr>
</tbody>
</table>

1. The expressions “more likely and less likely to be in resonance with the building” were used in lieu of the terms “flexible and rigid”, respectively.

5.10.2 Estimation of the seismic force demands on NSC located at different floor levels of the archetype buildings based on the ATC-120 equations

Equation 5.14 is used to estimate the normalized seismic design force (\( PCA/PGA \)) on NSC located at different floor levels of the studied archetype buildings. Four different primary-secondary scenarios are considered: (i) elastic NSC–elastic supporting building; (ii) elastic NSC–inelastic supporting building; (iii) inelastic NSC–inelastic supporting building; (iv) inelastic NSC–elastic supporting building. The following assumptions are adopted when using Equation 5-14: for the cases in which NSCs are inelastic, the \( PCA/PFA \) values corresponding to the component ductility value of 1.5 or 2.0 is used; NSC are considered to be more likely in tune with the supporting building modal periods; for the sake of consistency with the PCA demands obtained from the archetype buildings, the term \( R_{poc} \) is taken out from the design equation; the SC records are used for the evaluations. With these assumptions, the design \( PCA/PGA \) profiles for different SMRF and RCSW archetype buildings are computed and shown in Figures 5-55 and 5-56, respectively. In a subsequent section, the estimated design values are evaluated with the responses obtained from analyzing the archetype buildings under the SC record set.
Figure 5-55 ATC-120 proposed design PCA/PGA profiles for different primary-secondary system assumptions for the SMRF buildings (for the inelastic NSC a ductility of 2.0 is assumed)
As seen in Figures 5-55 and 5-56, the proposed upper limit of $PCA/PGA = 5.0$ is not reached if a NSC is designed for a target ductility of 2.0, regardless of supporting building behaving elastically or inelastically. For the component ductility of 1.5, if the building is elastic, in a few cases the upper limit of 5.0 is reached. For the elastic component, in most cases the upper limit is reached. Another important observation is the discrepancy in the $PCA/PGA$ responses at the lower floor levels in the case of inelastic NSC–inelastic building. As seen in these cases, the value of the
PCA/PGA at the above-ground floors is smaller than that at the ground level, which is not consistent with expectations. The primary reason for this discrepancy is using a constant $R_{\mu b}$ for all relative heights whereas, strictly speaking, the value of $R_{\mu b}$ should be smaller for lower floor levels than the upper floors. This latter statement is discussed in detail next. If the value of $R_{\mu b}$ for the lower floor levels is modified, the mentioned discrepancy is mitigated.

5.11 Assessment of the ATC-120 equations for different primary-secondary system scenarios using the responses of archetype buildings

This section uses the floor spectra results of baseline and overdesigned archetype buildings exposed to the DE level SC records to evaluate the accuracy of the proposed equations by ATC-120. The equations are evaluated for the four previously mentioned primary-secondary scenarios, and potential modifications and improvements are proposed.

5.11.1 Assessment for the elastic NSC-inelastic building scenario

This section evaluates the ATC-120 equations for the elastic NSC-inelastic building scenario. A component viscous damping ratio of 5%, and the archetype buildings are exposed to the DE level SC records to simulate an inelastic building behavior. The evaluation is performed assuming $T_c \leq 1.0$ s (i.e., the range considered as the practical period range for typical NSCs).

First, the adequacy of the proposed upper limit of $PCA/PGA = 5.0$ is evaluated. Figure 5-57 illustrated the 5%-damped elastic roof spectra of all baseline and archetype buildings exposed to the DE level SC records. Results for all individual SC records are presented. A similar capping approach as the one developed by ATC-120 Project is adopted. The roof spectra are normalized to the first three modal periods of the individual buildings once at a time. Roof spectra are presented in Figure 5-57 (a) to (c) for three different regions, namely third-, second-, and first-mode regions. For each modal region, the 84th percentile spectrum is capped at the ordinate corresponding to either $0.85 T_{bi}$ or $1.15 T_{bi}$ (the one that is larger). Results show that for the critical modal region, i.e., second-mode region shown in Figure 5-57b, the applied cap is obtained at $S_{ac}/PGA = 5.9$ that is larger than the proposed upper limit of 5.0. For other modal regions, the applied caps are obtained at normalized ordinates smaller than 5.0. Based on the obtained results an alternative upper bound could be $S_{ac}/PGA = 6.0$.
Figure 5-57  The 5%-damped elastic normalized roof spectra for the baseline and overdesigned SMRF and RCSW archetype buildings exposed to the DE level SC records

The ATC-120 equation is evaluated based on the responses of individual buildings. In other words, the statistical analysis is not performed across different building responses. As a result, dispersion in responses are not as significant as those shown in Figures 5-57, and hence, the mean responses under the SC records are used for the evaluation. The evaluation is conducted with applying the capping over the NSC period ratio region \(0.85 \leq \frac{T_c}{T_{b1}} \leq 1.15\). Only NSC periods smaller than 1.0 s (i.e., the practical range of typical electrical and mechanical NSCs based on Section 5.9.2) are considered. The PCA adopting the abovementioned strategy is computed for all floor levels of the archetype buildings. For a given building and a given floor, the value of the computed PCA is normalized to the corresponding design value proposed by the ATC-120 equations (i.e., Equation 5.14a and 5.14b). The results are shown in Figures 5-58(a) to (d) for different groups of archetype buildings.
As seen in Figure 5-58(a) to (d), the ATC-120 equation at the upper floor levels can, in most cases, capture the computed $PCA/PGA$ responses. However, at the mid-height floor levels, the estimation of the ATC-120 equation can be less than the computed responses (i.e., by a factor of 1.6 at some cases). This observation is because of the assumed distribution for the term $(PFA/PGA)/R_{\mu b}$. Simulation results presented in this dissertation illustrate that under the DE level ground motion (i.e., inelastic building case), the $PFA/PGA$ response at the middle floor of the archetype buildings are fairly close to those at the roof (e.g., see Figures 5-14 and 5-16), whereas, in the proposed distribution by ATC-120, design values at mid-height floors are significantly lower than that at the roof. (see Figures 5-56 and 5-57 the design value for the inelastic buildings). This discrepancy occurs because the $PFA/PGA$ equation was developed based on the

Figure 5-58  Evaluation of the ATC-120 equation using the responses of archetype buildings; (elastic NSCs with 5% damping and periods smaller than 1.0 s; inelastic building)
responses of elastic (or nearly-elastic) instrumented buildings, and a constant $R_{\mu b}$ was applied to reduce the elastic $PFA/PGA$ values at all floor levels, whereas results shown in Figure 5-18 for archetype models illustrate that $R_{\mu b}$ at the lower floor levels could be significantly smaller than that at the roof. From this point of view, the ATC-120 proposed equations are primarily based on the response of elastic (or nearly elastic) buildings. To overcome this discrepancy, the following modification is proposed to the equation used for the estimation of the $R_{\mu b}$ to be used in Equation 5.14a:

$$
(R_{\mu b})_{\text{modified}} = 1 + \left(\frac{z}{h}\right)^{0.75}.[(R_{\mu b})_{\text{ATC-120}} - 1]
$$

(5.17)

The evaluation of the modified ATC-120 equation for the SMRF and RCSW buildings is illustrated in Figure 5-59(a) to (d). As seen, almost for all cases, the computed ratios are limited to 1.30. Hence, the modified equation can be accepted by engineering judgment (the exceptions are the mid-height floor levels of the overdesigned four- and six-story SMRF and the roof level of the single- and two-story SMRF, shown in Figure 5-59b). These values are obtained assuming the NSCs behave elastically. There was a belief as part of the ATC-120 project that most NSCs will experience some level of energy dissipation (equivalent ductility capacity) through nonlinear mechanisms other than viscous damping. The results of this dissertation illustrate that a mild level of NSC inelasticity (i.e., ductility demands as low as 1.25) can significantly reduce the NSCs force demands with respect to the elastic NSC case. Therefore, it is reasonable to accept exceedances observed in Figure 5-59.
5.11.2 Assessment for the inelastic NSC-inelastic building scenario

In this section, the modified ATC-120 equation, referred to as $\text{PCA}_{\text{modified}}$, is evaluated for the inelastic NSC-inelastic building scenario through using the roof spectra of baseline and overdesigned archetype buildings under the DE level SC records. In this section, a component ductility of 2.0 is assumed. The modified equation is still based on a NSC viscous damping ratio of 5%. However, the evaluation is conducted for component damping ratios of 2 and 5% to investigate the potential improvement to the proposed equation for inelastic NSCs with viscous damping ratios smaller than 5%.

Figures 5-60a to d depict the computed inelastic 5%-damped roof spectra normalized to the $\text{PCA}_{\text{modified}}$ for different groups of the studied archetype buildings. As seen in Figures 5-60a and
the modified design PCA can capture the computed responses for the baseline SMRF and RCSW buildings. For the overdesigned buildings at tuning conditions, the modified equation is exceeded by a factor up to 25%. For the flexible 12-story RCSW building (see Figure 5-60c), the exceedance is up to 40%. Figures 5-61(a) to (d) present a similar evaluation for the 2%-damped roof spectra. As seen in this case, at the tuning condition the modified equation is exceeded for different buildings by factors up to 25% for the baseline buildings and 70% for other buildings. This observation illustrates that, for NSCs with damping ratios other than 5%, if the objective is to limit the ductility capacity to 2.0 (the target ductility examined in this section), a component DMF, as those discussed in Section 5.7.4 needs to be incorporated to the design equations. For example, for the considered inelastic NSC–inelastic building scenario, assuming a NSC damping ratio of 2%, a DMF of 1.26 could be incorporated into the modified PCA equation. With applying this modification, the design PCA can capture computed responses for all baseline buildings; the maximum exceedance for the other buildings in the most critical case (i.e., the for flexible 12-story RCSW) is 35%. As an alternative, the deviation of NSCs viscous damping from the baseline 5% value could be compensated by accepting an increase in the NSC ductility demand. Consider PCA design value (i.e., the NSC strength) for an inelastic NSC is determined assuming a viscous damping ratio of 5% and a target ductility of 2.0. If the NSC viscous damping ratio is less than 5%, given that the strength of the NSC is constant, it will experience a ductility demand higher than the target. To quantify the amount of this increase in the NSC ductility demand, constant-strength inelastic 2%-damped spectra (with the strength equal to design PCA5% value) can be used, which is part of a study being conducted by the authors. Another important point is that in the ATC-120 project, the concept of capping floor spectral ordinates was applied to elastic and low-to-moderately inelastic NSCs but not for a NSC with a target ductility of 2.0. Results shown in Figure 5-60, more specifically 50-60d, illustrate the presence of relatively sharp spike for the cases that exceed the proposed PCA value. It might be reasonable to apply the floor spectral ordinate capping for these cases as well.
Figure 5-60  Evaluation of the ATC-120 equation using the roof spectra of archetype buildings; (inelastic NSC with a ductility of 2.0 and damping of 5%; inelastic building)
Figure 5-61  Evaluation of the ATC-120 equation using the roof spectra of archetype buildings; (inelastic NSC with a ductility of 2.0 and damping of 2%; inelastic building)

5.11.3 Assessment for the inelastic NSC-elastic building scenario

This section presents an evaluation of the ATC-120 modified equation for the inelastic NSC-elastic building scenario using the roof spectra of the baseline archetype buildings at the DE level. In this study, the term elastic building refers to the archetype buildings under a 0.25 DE level ground motion. For the elastic building case, the building ductility term $R_{\mu_b}$ of Equation 5.14 is set to 1.0 unity.

Figure 5-65(a) illustrates the computed inelastic 5%-damped roof spectra of the elastic baseline SMRF buildings normalized to the ATC-120 modified equation. Figure 5-65(b) illustrates the results for the inelastic 2%-damped roof spectra. The results of similar evaluations for the elastic baseline RCSW buildings are presented in Figure 5-66(a) and (b). As seen, the proposed equation
can capture the peak computed responses for all studied cases. The one exception is the eight-story RCSW shown in Figure 5-66b where the NSC damping ratio is 2%. For this case, applying the spectral ordinate capping approach can significantly mitigate the observed sharp spike.

**Figure 5-62** Evaluation of the ATC-120 equation using the roof spectra of baseline SMRFs assuming a NSC damping ratio of (a) 5%; (b) 2% (inelastic NSC with a target ductility of 2.0; elastic building)

**Figure 5-63** Evaluation of the ATC-120 equation using the roof spectra of baseline RCSWs assuming a NSC damping ratio of (a) 5%; (b) 2% (inelastic NSC with a target ductility of 2.0; elastic building)
5.11.4 Assessment for the elastic NSC-elastic building scenario

A 5% NSC damping ratio is used in this section. This primary-secondary scenario is studied herein only for the sake of completeness of this section, whereas it is understood that this is a rare case in practice. In other words, designing a primary-secondary elastic system might be in contradiction with the fundamentals of earthquake engineering in which seismic fuses are incorporated to limit the earthquake-induced forces in structural and nonstructural components.

Figure 5-64 presents the normalized 5%-damped elastic roof spectra for the four-, six-, eight-, and 12-story bassline SMRF and RCSW buildings. As expected, PCA values for this primary-secondary scenario are larger than those of other scenarios in which NSC and/or supporting buildings were allowed to become inelastic. The spectral spikes in the vicinity of the higher modes of the buildings are very sharp suggesting that for these cases the spectral capping approach can significantly reduce PCA demands, whereas, the spike in the vicinity of buildings fundamental mode is broader.

For short-period SMRF buildings the maximum value of the roof spectral ordinates (i.e., PCA) occurs in the vicinity of the supporting building fundamental-mode, however, these periods are beyond the practical period range considered in this study for NSCs (i.e., $T_c \leq 1.0$ s). Therefore, for the SMRF building, the sharper spikes are always used for the evaluation of the proposed equation, and hence, the spectral capping has a significant effect on the computed demands on NSCs. For example, for the roof level of the four-story SMRF building, the spectral capping can reduce PCA/PGA from 9.6 to 5.6. For RCSW buildings, at the roof level, the spikes in the vicinity of the building first-mode dominates the floor spectra. These spikes, which are in the practical range of NSCs considered herein, are relatively broad, and hence the reduction obtained in PCA via the spectral capping at the roof level of these RCSW buildings is not as significant as that at the roof level of SMRF buildings. At lower floors spikes in the vicinity of higher mode controls, and the effect of spectral capping is significant for both SMRF and RCSW buildings.
Figure 5-64  **Mean 5%-damped elastic roof spectra for all baseline SMRF and RCSW) archetype buildings (four-, six-, eight-, and 12-story) exposed to 0.25 DE level SC records (elastic NSC with 5% damping; elastic building)**

Figure 5-65 illustrates the evaluation of the ATC-120 equation for the elastic NSC-elastic building scenario for some selected SMRF archetype buildings. In Figure 5-65a, the results are presented without applying the spectral capping on the computed PCA responses, whereas in Figure 5-65b, the spectral capping is performed. Similar results are presented in Figure 5-66 for the selected RCSW archetype buildings.

As seen, for the SMRF buildings, when the spectral capping is applied on PCA responses, the proposed equation can capture the computed demands in all studied cases. For the RCSW, the computed responses consistently exceed the proposed equation. It is important to note that the computed responses are based on a building viscous damping ratio of 2.5%. RCSW buildings even when exposed to relatively small intensity ground motion (e.g., 0.25 DE used in the evaluations conducted in this section), may exhibit significantly larger damping ratios provided by the system cracking. On the other hand, the effect of viscous damping of the supporting building on PCA responses is significant when supporting building responds elastically. In the building viscous damping ratio is considered to be 5%, the computed PCA values shown in Figure 5-66 may be reduced by a factor of 1.3.
5.12 Appendix I. Importance of component-building-interaction

As mentioned in Section 5.4, in this study component-building-interaction is neglected. It is well-known that this assumption is adequate for light NSCs and conservative for heavy NSCs. In this section, as an example, the effect of the dynamic interaction between an elastic tuned NSC and the 2-story RCSW overdesigned by a factor of 3.0 is investigated. Firstly, the building is analyzed under the spectrum compatible record set. The roof acceleration motions are extracted and used as
input for a SDOF program. The mean value of the maximum acceleration responses of the NSC under the record set is obtained. This model is referred as to the decoupled model. Secondly, the NSC is modeled as a SDOF system attached to the roof level through a spring-dashpot system, and the mean value of the maximum acceleration responses is directly obtained from the primary model. This model is the coupled model.

The period and viscous damping ratio corresponding to the fundamental mode of the building are 0.22 s and 5%, respectively. The building mass, \( m_{\text{bldg.}} \), is 10080 kips. The NSC viscous damping ration, \( \xi_c \) is 5%. The ratio of the component mass to the building mass, \( \rho_c = m_c/m_b \), for using in the coupled model is varied from 10\(^{-5}\) to 0.01. The stiffness and damping coefficient for the NSC are

\[
\begin{align*}
    k_c &= \frac{4\pi^2 m_c}{T_c g} \\
    c_c &= 2\xi_c m_c \omega_c
\end{align*}
\]

Table 5-4 presents a comparison between the response of the NSC obtained from the uncoupled and coupled models. As seen, with increasing the NSC mass ratio, the acceleration response of NSC decreases. This is consistent with the behavior of a tuned-mass-damper (TMD) in which the experienced acceleration responses of the mass damper reduces with increasing it mass ratio. Table 2 illustrates that ignoring the building-component interaction for NSCs with masses larger than 0.001 of the studied building can lead to an overestimation of the NSC acceleration response at least by 16%.

<table>
<thead>
<tr>
<th>( \rho_c )</th>
<th>( m_c ) (kips)</th>
<th>( \text{max}(\text{accel}_c)/\text{PGA} )</th>
<th>NSC response reduction due to coupling effect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Uncoupled model</td>
<td>Coupled model</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>10.14</td>
<td>Not applicable</td>
</tr>
<tr>
<td>( 10^{-5} )</td>
<td>0.1</td>
<td>10.14</td>
<td>8.69</td>
</tr>
<tr>
<td>( 10^{-3} )</td>
<td>10.1</td>
<td>10.14</td>
<td>8.48</td>
</tr>
<tr>
<td>( 10^{-2} )</td>
<td>100.8</td>
<td>10.14</td>
<td>7.28</td>
</tr>
</tbody>
</table>

Table 5-4: the mean value of the maximum acceleration responses of the NSC for the coupled and uncoupled cases (the building model is exposed to the 20 SC records)
5.13 Conclusions

Numerous studies have been conducted in the past three decades on the quantification of seismic demands on acceleration-sensitive nonstructural components (NSCs). However, many of these works have assumed a linear-elastic behavior for NSCs and/or their supporting buildings. The studies that have focused on the inelastic supporting buildings have been mostly based on simplified single-degree-of-freedom (SDOF) models or multi-story generic frames with the assumption of an elastic NSC behavior and NSC damping ratio of 5%. In the current ASCE 7-16 equivalent static equations for designing NSCs, the effect of the supporting building inelasticity is not explicitly incorporated. The effect of NSC inelasticity is taken into account by a response modification factor, $R_p$, which is based on engineering judgment rather than experimental or numerical studies. At present, there is no clear understanding of the consequences of designing NSCs using these $R_p$ factors (these consequences include NSC inelastic displacement and ductility demands). Recently as part of ATC-120 project, improved design equations for NSCs were developed. Additional terms incorporated into these equations with respect to the current ASCE 7-16 $F_p$ equation are basically the terms reflecting the effect of building inelasticity and fundamental period. The terms reflecting the component amplification factor and ductility are also modified. This final version of these equations is based on a NSC viscous damping ratio of 5% for both elastic and inelastic NSCs.

The discussion ensued in the ATC-120 meetings inspired the authors to conduct the present study. This study primarily evaluates seismic force and displacement demands on acceleration-sensitive NSCs assuming four different primary-secondary scenarios: (i) elastic NSC–elastic supporting building; (ii) elastic NSC–inelastic supporting building; (iii) inelastic NSC–inelastic supporting building; (iv) inelastic NSC–elastic supporting building. The supporting buildings used to generate floor acceleration motions are code-based designed SMRF and RCSW buildings with different heights varying from 1 to 12-story. An overdesigned version of the archetype buildings is also evaluated. Input ground motion excitations are a set of 20 spectrum compatible record and a set of 44 far-filed records. This chapter basically consists of three different part:

The first part uses the elastic responses of NSCs mounted on elastic or inelastic primary buildings. The effects of supporting building widespread inelasticity, localized inelasticity and
Viscous damping ratio on the elastic floor spectra are quantified. Results of this part corroborate the results of many previous studies conducted on understanding the effect of supporting building characteristics on NSC seismic demands using generic frames. It is observed that primary building inelasticity in most cases has the effect of reducing demands on NSCs, especially for tuned NSCs. For some non-tuning NSCs, the building inelasticity can increase demands on NSCs. This effect is more highlighted for low-damping NSCs and for buildings with a weak-story mechanism (i.e., localized plasticity). In this part, the adequacy of ASCE 7-16 equivalent static equations for designing NSCs are evaluated, and their potential shortcomings are identified.

In the second part, constant-ductility and constant-\( R \) inelastic floor spectra are developed for different floor levels of the archetype buildings assuming different NSC viscous damping ratio and target ductility values. The results show that even a mild level of NSC inelasticity can significantly decrease its force and displacement seismic demand in tuning conditions. At short period ratio NSCs, the component inelasticity leads to significantly large ductility demands, and is not recommended. Component modification factor and inelastic displacement ratio for inelastic floor spectra are quantified and compared with those of the ground spectra. Results show that component response modification factor is a function of component tuning ratio, viscous damping ratio, and target ductility, and at a lesser extent, of the supporting building level of inelastic behavior, lateral load resisting system, and fundamental period, the ground motion characteristics and the vertical location of NSC within the building. The largest beneficial effect of NSC inelasticity is obtained when the component damping is low, the supporting building responds elastically, the component is tuned to the fundamental mode of the building, and it is mounted on the roof floor.

The last part of this chapter presents an evaluation of the recently proposed equation by ATC-120 for designing acceleration-sensitive NSCs. This evaluation is conducted for the four previously mentioned primary-secondary scenarios and was performed primarily after the analyses that formed part of the ATC-120 project were finalized. The floor acceleration motions obtained from different floor levels of the baseline and overdesigned archetype buildings are used for the conducted evaluation. Potential improvements to the ATC-120 equations are proposed. The most important recommended improvement is regarding the \( R_{\mu b} \) value. The proposed value for this parameter is the same for all floor levels. Results of this study suggest that the value of \( R_{\mu b} \) may need to reduce from top to bottom floors. If the objective is to meet the criteria adopted when
developing these equations (i.e., the limiting the NSC ductility to the predefined values), the following modifications are also recommended:

(i) increasing the upper limit of 5.0 to 6.0. This upper limit is reached primarily when the NSC is elastic; (ii) incorporating an additional parameter (i.e., damping modification factor, DMF) to account for the NSC viscous damping ratios other than 5%. Based on the preliminary evaluations conducted in this study, and also some studies performed as part of the ATC-120 project, the value of this parameter for an elastic NSC with 2% damping is 1.6 whereas its value for an inelastic NSC with a target ductility of 2.0 and viscous damping ratio of 2% is the square root of 1.6 (i.e., 1.26). However, an alternative solution is to ignore modifications listed under (i) and (ii) and accept an increase in the ductility demands on NSCs with respect to the target values. This approach needs accommodating a larger NSC ductility demand. Adopting such an approach implies that no component remains in the elastic behavior range. This approach is further justifiable if one considers the following discussion:

If the force-based design approach is adopted, because of the 3D effects discussed in Chapter 2 of this dissertation, additional factors greater than 1.0 (in some instances as large as 1.3) may need to be applied on the baseline equations proposed by ATC-120. Applying the parameters incorporating 3D effects, NSC viscous damping deviation from the nominal 5%, simultaneously, can lead to very large design values, especially for elastic NSCs. For example, for an elastic NSC with 2% damping mounted on a floor with a flexible diaphragm system, two amplification factors of 1.5 and 1.3 should be simultaneously incorporated into the design equations. In other words, and additional amplification factor of 1.95 could be applied. This may significantly increase construction costs. However, as an alternative, NSCs can be designed for the baseline equations without any further amplification due to 3D effects and/or deviation of NSC damping from 5%, if the ductility demand of NSCs for these cases can be controlled. If it is beyond admissible values prescribed in the design criteria, the design forces should be increased.

5.14 Acknowledgments

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statements of interpretations contained in this publication. No warranty is offered with regard to the results, findings, and recommendations contained herein, either by the National Institute of Standards and Technology, the Applied Technology Council, its directors, members or employees. These organizations or individuals do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information included in this publication.

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5.15 References


Chapter 6

Challenges in Modeling the Superstructure Viscous Damping in Base-isolated Buildings
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Chapter 6

Challenges in Modeling the Superstructure Viscous Damping in Base-isolated Buildings Mass Isolation System for Seismic Vibration Control of Buildings


6.1 Abstract

In this chapter, the drawbacks of the improper modeling of the superstructure viscous damping in base-isolated (BI) buildings are addressed. Six different approaches are investigated for constructing the superstructure viscous damping matrix. These scenarios are based on the potential combinations of three viscous damping models (i.e., Mass-proportional, MD, Stiffness-proportional, KD, or Rayleigh damping, RD) and two methods of computing the coefficients multiplying the mass and stiffness matrices (i.e., based on either non-isolated, NI, or, BI periods). Test-bed structural models with different dynamic characteristics are used to simulate their responses when exposed to a set of far-field records. Results illustrate that for all studied BI configurations, even with a superstructure viscous damping as low as 2%, structural responses, especially higher-mode dominated responses such as floor spectral accelerations at short periods, are strongly dependent on the method of modeling the superstructure viscous damping. Applying either the KD-NI or KD-BI approaches results in a reliable estimate of first-mode dominated responses whereas the other four scenarios may underestimate these responses up to 60%. However, when the isolation system damping is relatively high, the KD approaches assign significant damping to the higher modes that spuriously suppresses high-frequency responses. The RD-BI approach can provide a more reasonable estimate of floor spectral accelerations at short periods. It is shown that a proposed modified RD approach can conservatively capture the upper-bound estimates of both first- and higher-mode dominated responses of BI buildings.
Keywords: Base isolation; Rayleigh damping; Higher-mode effects; Modal analysis; Modal mass participation ratio; Modal damping.

6.2 Introduction

In a base isolated (BI) building, while the superstructure is typically designed to remain in the elastic or near-elastic range during a design earthquake, supplementary dampers (e.g., lead rubber bearing, viscous dampers, etc.) provide the major source of seismic energy dissipation. Other sources of energy dissipation such as \( (i) \) foundation damping due to soil nonlinearity and radiation of seismic waves; \( (ii) \) friction and slippage in steel connections; \( (iii) \) opening and closing of micro-cracks in concrete members; \( (iv) \) stressing of nonstructural components (e.g., partition walls, mechanical equipment, fireproofing, etc.); \( (v) \) friction between the structure and nonstructural components; and similar mechanisms are usually represented by means of an equivalent linear viscous damping model (Petrini et al. 2008; Chopra 2015). This type of damping is usually assumed to be between 1 and 5% for different structures, which is mainly based on experts’ opinion, engineering judgment, and limited system identification studies of buildings exposed primarily to small-amplitude vibrations, e.g., ambient vibration. In most cases, the equivalent linear viscous damping is incorporated into the equations of motion via a damping matrix of constant coefficients multiplying a vector of velocities of the superstructure’s degrees of freedom.

In BI buildings, supplementary dampers are modeled either directly as nonlinear elements or as an equivalent nonlinear viscous damping, the latter is the focus of this chapter. As a matter of fact, experimental studies on large-scale shaking tables have illustrated that nonlinear isolation systems (e.g., lead rubber bearing and high damping natural rubber bearing) can be simulated to some degree of accuracy by a linearly viscoelastic model (Tsai and Kelly 1988). In a BI building, because of the disparity between the high damping of the isolation system and the relatively low damping of the superstructure, the resultant damping of the combined system is non-classical (or non-proportional). In other words, in a BI building, the global damping matrix of the system, \( \mathbf{C} \), is not a direct superposition of the global stiffness, \( \mathbf{K} \), and mass, \( \mathbf{M} \), matrices. The \( \mathbf{C} \) matrix can be constructed by directly assembling the damping matrices of the superstructure and the isolation system. The term superstructure in this chapter refers to the section of the BI structure above the isolation layer that is equivalent to the non-isolated (fixed-base) building in the case of a BI system.
(see Figure 6-1). First, assuming that similar damping mechanisms are distributed throughout the superstructure, the classical damping applies, and the superstructure viscous damping matrix, $C_s$, can be constructed using the procedure described later in this section. The damping contribution of the isolation system (the seismic energy dissipating devices) is then assembled into $C_s$ to obtain the global damping matrix for the combined system, $C$. The presence of a non-classical global damping matrix, $C$, implies that modal periods, damping ratios, and modal vectors depend on $M$, $K$, and $C$. As a result, the modal equations are coupled, and classical modal analysis, which neglects the off-diagonal terms of the generalized damping matrix, is not applicable (Chopra 2015).

The mathematical model of an $n$-story linear-elastic shear building equipped with a linear-viscous isolation system is schematically illustrated in Figure 6-1(b). In this model, the isolation system is simulated as an additional degree of freedom with a mass equal to the typical story mass of the building. The equation of motion for the BI building exposed to ground motion excitations can be expressed as:

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = -Mr\ddot{x}_g(t), \quad (6.1)$$

where

$M = \text{diag}(m^b, m^s_1, \ldots, m^s_n)$,

$C = \begin{bmatrix} c^b + c^s_1 & -c^s_1 \\ -c^s_1 & C^s \end{bmatrix}_{(n+1)(n+1)}$, \quad $K = \begin{bmatrix} k^b + k^s_1 & -k^s_1 \\ -k^s_1 & K^s \end{bmatrix}_{(n+1)(n+1)}$,

$c^b = 2\xi^b\omega^b \left( m^b + \sum_{i=1}^n m^s_i \right)$.

$M$, $C$, and $K$ are the global mass, damping and lateral stiffness matrices of the BI building, respectively. $x = [x^b x^s_1 x^s_2 \cdots x^s_n]$ is the displacement vector of the system relative to the ground. Superscripts $s$ and $b$ refer to the superstructure and base-isolation system, respectively. $r$ is the influence vector; because displacements are measured relative to the ground, $r$ is a column vector of ones. $\ddot{x}_g$ is the ground acceleration. $m^b$ is the isolated raft mass, and $m^s_i$ is the superstructure mass at the $i$-th story. $c^b$ and $k^b$ are the isolation system damping and lateral stiffness coefficients, respectively. $\omega^b$ and $\xi^b$ are the isolation system central frequency and equivalent viscous damping
ratio assuming that the superstructure is laterally rigid with respect to the isolation system. $c_1^s$ and $k_1^s$ are the superstructure viscous damping and lateral stiffness coefficients at the first story, respectively. $C^s$ and $K^s$ are the damping and stiffness coefficient matrices of the superstructure, respectively. The method of constructing $C^s$ and its influence on the structural responses in the BI buildings are the focus of the present study.

Two different approaches are generally used to construct a classical damping matrix (e.g., the superstructure viscous damping matrix, $C^s$ in a BI building). One approach is based on the superposition of modal damping matrices. The alternative approach, which is the focus of the present study, is based on the superposition of mass and stiffness matrices. Mass-proportional damping (MD), Stiffness-proportional damping (KD), and Rayleigh damping (RD) are the well-known formats of this approach, the latter is widely utilized in modeling multi-degrees-of-freedom buildings.

The MD and KD models need to assign a specific superstructure damping ratio, $\xi^s$, to a single mode (generally to the first mode), whereas, in the Rayleigh approach specific damping ratios should be assigned to two structural modes of vibration. Equations 6.2(a) to (c) express the superstructure viscous damping matrix for the MD, KD, and RD, respectively:

$$C_{MD}^s = \alpha_m M^s \quad \text{where} \quad \alpha_m = 2\xi^s \omega_1.$$  \hspace{1cm} (6.2a)

$$C_{KD}^s = \beta_k K^s \quad \text{where} \quad \beta_k = 2\xi^s / \omega_1.$$  \hspace{1cm} (6.2b)
\[
C_{RD}^s = \alpha_m M^s + \beta_k K^s \quad \text{where} \quad \alpha_m = 2\xi^s\omega_i\omega_j/\left(\omega_i + \omega_j\right) \quad \text{and} \quad \beta_k = 2\xi^s/\left(\omega_i + \omega_j\right). \tag{6.2c}
\]

where \(\alpha_m\) and \(\beta_k\) are the constant coefficients of unit \(s^{-1}\) and \(s\), respectively. \(\omega\)'s can be selected as either the superstructure frequencies or the BI building frequencies. As the results of this study illustrate, this selection significantly influences a BI building seismic responses. If the KD or MD approaches are used, \(\xi^s\) is the viscous damping ratio of the (non-isolated) superstructure, shown in Figure 6-1(a), at its fundamental mode. If the RD approach is used, \(\xi^s\) is the superstructure viscous damping ratio at its first two modes (\(\xi_1^s = \xi_2^s\)). Hereinafter, \(\xi^s\) is briefly called “superstructure viscous damping ratio”. In addition, the superstructure fundamental period is called superstructure period for brevity. With these approaches, the specified damping ratio to the \(n\)-th mode of vibration of the superstructure is

\[
\xi_n = \frac{\alpha_m}{2\omega_n} + \frac{\beta_k\omega_n}{2}. \tag{6.3}
\]

Overall, six different superstructure viscous damping scenarios are considered in the present study. Table 6-1 illustrates the nomenclatures used for these scenarios.

<table>
<thead>
<tr>
<th>Damping model</th>
<th>Non-isolated frequencies</th>
<th>Base-isolated frequencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass-proportional damping</td>
<td>MD-NI</td>
<td>MD-BI</td>
</tr>
<tr>
<td>Stiffness-proportional damping</td>
<td>KD-NI</td>
<td>KD-BI</td>
</tr>
<tr>
<td>Rayleigh damping</td>
<td>RD-NI</td>
<td>RD-BI</td>
</tr>
</tbody>
</table>

### 6.3 Literature Survey

Consensus does not exist on the approach adopted for modeling the superstructure viscous damping in BI buildings. For example, the studies presented in (Chimamphant and Kasai 2016; Anajafi and Medina 2018a; Anajafi and Medina 2018c) and a numerical model provided in a reference structural dynamics book (Chopra 2015) used the KD approach based on the superstructure modal frequencies (i.e., KD-NI). Becker et al. (2015) utilized the KD method based on the isolated building second mode (i.e., a version of KD-BI). Some other studies such as those in (Komodromos et al. 2007; Moretti et al. 2014) applied the RD approach (without specifying...
whether it was based on BI or NI modes). Structural dynamics reference books (e.g., Clough and Penzien 1993; Chopra 2015) routinely state that to compute the coefficients of RD for a non-classically damped system, the modal periods of the combined system should be used. This statement implies that for a BI building, the isolated modal periods rather than those of the NI superstructure alone should be used for constructing the global $C$ matrix (i.e., RD-BI should be used). Many research works (e.g., Tsai and Kelly 1993; Pourzeynali and Zarif 2008; Kilar and Koren 2009; Kilar et al. 2011; Chen et al. 2013; Fallah and Zamiri 2013; Chen et al. 2014) used the RD-BI scenario when studying the behavior of BI buildings. The effect of the method of modeling the superstructure viscous damping on structural responses of BI buildings has not been studied to the level of detail in which this issue has been addressed in NI structures. The reader is referred to example studies in (e.g., Léger and Dussault 1992; Bernal 1994; Medina and Krawinkler 2004; Hall 2006; Charney 2008; Zareian and Medina 2010; Erduran 2012; Jehel et al. 2014; Chopra and McKenna 2016) for information on modeling the viscous damping in NI structures. The lack of additional attention to modeling the superstructure viscous damping on BI structures most likely originates from the notion that energy dissipation provided by the superstructure viscous damping is negligible when compared to the significantly higher energy dissipation expected from the supplementary dampers associated with the isolation system. However, it has been shown through a number of studies that the improper modeling of the superstructure viscous damping can lead to an underestimation of seismic demands in BI buildings.

Hall (2006), through a simplified analytical model of an example BI building (with bilinear isolators assuming a design period of 2.5 s and a rigid superstructure with a viscous damping ratio of 5%), investigated the drawbacks of using the RD method to model the superstructure viscous damping in BI buildings. Hall showed that the RD-NI approach could specify an unrealistically high damping ratio to the first mode of the isolated structure owing to the MD term and the relatively stiff superstructure. Hall recommended using the KD method based on the secant shear stiffness of the isolation system. Ryan and Polanco (2008) evaluated the viscous damping ratio specified to the first mode of BI buildings by the RD and KD approaches. They studied 60 different BI building configurations with linear isolation systems. The 60 models were developed based on different combinations of the fundamental period (i.e., 0.3, 0.5 and 1.0 s) and viscous damping ratio of the superstructure (i.e., 2 and 5%), and isolation system period (i.e., 2.0 and 4.0 s). In their
study, a constant damping ratio of 15% was assigned to the isolation system. Ryan and Polanco illustrated that the RD approaches, especially RD-NI, impart considerable damping to the first mode of the isolated structure beyond the energy dissipation provided by the isolation system. They also conducted response history analysis, using a single ground motion record, on two example BI buildings with linear and nonlinear isolation systems. They showed that a BI model with a rigid superstructure could provide a lower-bound estimate of the first-mode dominated responses of a multi-degree-of-freedom BI building. Selecting this model as the baseline, Ryan and Polanco showed that using the RD approach can lead to an underestimation of the roof displacement responses of the studied BI buildings by 10-25%. To overcome this deficiency, they proposed applying the KD-NI approach.

Few research works have used the responses of tested BI buildings to evaluate the appropriateness of the approach adopted for modeling the superstructure viscous damping. Pant and Wijeyewickrema (2012) compared the seismic responses (i.e., maximum floor displacements, floor accelerations and story shear forces) of a 0.4-scaled three-story BI reinforced concrete building obtained from a shake-table test under two ground motion records with those obtained from the numerical models of the building. The building was isolated using bearings whose responses can be represented with a bilinear hysteretic model. Pant and Wijeyewickrema performed numerical simulations using a viscous damping ratio ranging from 1% to 5% and examined 28 different viscous damping matrices for each viscous damping ratio considered. The parameters to construct the 28 different viscous damping matrices were the damping model (i.e., RD, KD or MD), the stiffness matrix of the system (i.e., the initial or the updated stiffness of the BI building at each step), and the modal periods for computing the coefficients \( \alpha_m \) and \( \beta_k \) (i.e., different NI or BI modes). They concluded that (i) the RD approach when \( \alpha_m \) and \( \beta_k \) are calculated using the modal periods of the BI structure based on the post-elastic stiffness of the isolators (i.e., a version of RD-BI) is associated with the least error in estimating peak floor acceleration responses; (ii) the KD-NI method results in the least error in estimating floor displacement and story shear forces. Pant et al. (2013), using the models consistent with those used in Pant and Wijeyewickrema (2012), concluded that the RD-BI model where damping coefficients are calculated based on the modal periods of the BI structure with post-elastic stiffness of the isolation system results in a relatively small error in peak displacement and acceleration responses.
but could not capture the frequency content of the floor accelerations reasonably well. They showed that the KD approach where $\beta_k$ is computed from the period of the BI building with the post-elastic stiffness of the isolation system is able to capture the frequency content of the floor accelerations, in addition to providing reasonable estimates of the peak displacement and floor acceleration responses. In another study, Dao and Ryan (2013) developed numerical models for a full-scale five-story reinforced concrete building tested at E-Defense shake table. The building was isolated by triple pendulum bearings. Dao and Ryan conducted response history analyses under two ground motion record. They reported that the KD-BI modeling approach could underestimate short-period floor spectral accelerations and also produce large errors in peak story drift responses. They recommended using a modified RD-BI scenario in which additional dampers were connected between the base and the roof.

As seen, apparent contradictions or inconsistencies exist in the results of the aforementioned studies regarding the appropriate method for modeling the superstructure viscous damping in BI buildings. Two of the main goals of the present study are to address the reasons behind these inconsistencies and propose a damping modeling approach to reliably estimate both higher- and first-mode dominated responses of BI buildings. In the context of this study, a higher-mode dominated response relates exclusively to short-period floor spectral accelerations for the building models studied.

### 6.4 Contributions of this Study

Most of the aforementioned works in Section 6.3 focused on the effect of the improper modeling of the superstructure viscous damping in BI building on first-mode dominated responses and rarely on higher-mode dominated responses. The lack of studies addressing the effect of modeling the superstructure viscous damping on higher-mode responses might be due to the notion that the contribution of the higher modes in BI buildings, because of their relatively low mass participation, is not significant. For example, Ryan and Polanco (2008) stated that “the details of how Stiffness-proportional damping is applied to the superstructure will affect the higher mode damping ratios [of BI buildings], but these modes generally have low participation”. As another example, Chopra (2015) stated that “the higher modes [of BI buildings] are essentially not excited by the ground motion—although their [ground] pseudo-accelerations are large—because their modal static
responses are very small”. However, in the early works of Kelly et al. (e.g., see Kelly and Tsai 1985; Tsai and Kelly 1988) significant attentions were devoted to the evaluation of higher-mode dominated responses of BI buildings. In several other simulated numerical (e.g., see Fan and Ahmadi 1990; Juhn et al. 1992; Alhan and Gavin 2004; Huang et al. 2007; Isaković et al. 2011), tested (e.g., see Dao and Ryan 2013), and instrumented (e.g., see Nagarajaiah and Xiao-hong 2000; Nagarajaiah and Sun 2001) BI buildings, the significance of the high frequency content of floor acceleration responses is evident.

Drawing a definitive conclusion regarding the use of the superstructure viscous damping scenario based on experimental studies could be very challenging because experimental tests are not always representative of actual field conditions, but perhaps more importantly, modeling assumptions could bias this conclusion. In other words, discrepancies between the results of experiments and numerical models can occur due to a variety of phenomena and modeling assumptions (e.g., soil-structure interaction effects, the contribution of the gravity load resisting system and nonstructural elements to the overall building response, the hysteretic models adopted for structural components and isolation bearings, the real distribution of damage in structural components, etc.) some of which are very challenging to identify and quantify. Furthermore, some of the aforementioned conclusions in Section 6.3 were based on the responses of only a few BI buildings exposed to a few ground motions. However, the results of the present study illustrate the significance of the record-to-record and building-to-building variabilities.

From the hypothesis presented in this study, it is evident that there is a need to evaluate the sensitivity of seismic demand parameters of BI buildings (i.e., first- and higher-mode dominated responses) to the adopted superstructure viscous damping model. A fundamental question is whether or not the extent of this sensitivity is consistent across all studied BI buildings and response quantities. To address this question, a comprehensive numerical study is performed as part of this study with a wide variety of BI structures with different dynamic characteristics. Six different strategies are adopted for modeling the superstructure’s viscous damping. These six strategies are based on combinations that pair one of three damping models (i.e., MD, KD or RD) with two different methods of selecting modal periods (i.e., the NI or BI modal periods) for computing the coefficients multiplying the $\mathbf{M}^s$ and $\mathbf{K}^s$ matrices. More than 200 different BI building configurations are studied. The parameters varied to develop these models are the number
of stories, the superstructure period and viscous damping ratio, and the isolation system period and damping ratio. For each configuration, variations in response quantities due to the adoption of the superstructure viscous damping strategy are investigated. Furthermore, the appropriateness of the modal analysis predictions for approximating the seismic behavior of BI buildings is evaluated. Finally, a superstructure viscous damping model is proposed that can conservatively capture both the higher- and first-mode dominated responses of BI buildings.

6.5 Predictions of Modal Analysis for Base-isolated Buildings

6.5.1 Difference between the assigned first-mode damping by a superstructure damping model and the target damping

Because the global damping matrix of a BI building is non-proportional, Classical modal analysis is not, strictly speaking, applicable. However, in many instances (e.g., Ryan and Polanco 2008; Chopra 2015), the results of Classical modal analysis are used to roughly approximate the overall behavior of BI systems. This section uses Classical modal analysis to estimate the difference between the assigned first-mode damping ratio of BI buildings and the target first-mode damping (i.e., the isolation system damping). In Section 5.5.2, the results of the conducted Classical modal analysis are compared with those of Generalized modal analysis, which is the proper method for non-classically damped systems.

In the first mode of a BI building (i.e., the isolation mode), the superstructure moves essentially as a rigid body without large relative superstructure motions to be damped. Hence, the superstructure viscous damping has a relatively small effect on the damping of a BI building in the first mode. Hence, one of the basic premises in this study is that the damping specified to the first mode of a BI building should be similar to the isolation system damping, denoted as the target first-mode damping herein.

Modal damping ratios of the BI buildings, ignoring the off-diagonal terms of the transformed matrix (i.e., assuming a classically damped system), \( \Phi_i^T \mathbf{C} \Phi_i \), can be estimated as

\[
\xi_i = \frac{\Phi_i^T \mathbf{C} \Phi_i}{2 \omega_i (\Phi_i^T \mathbf{M} \Phi_i)}.
\]  

(6.4)
where \( \Phi_i \) and \( \omega_i \) are the \( i \)-th mode shape and \( i \)-th circular frequency of the BI building, respectively. In this case, the effective mass at the \( i \)-th mode of vibration of the BI building can be obtained by Equation (6-5):

\[
(m_i)_{\text{eff.}} = \frac{(\Phi_i^\top M r)^2}{(\Phi_i^\top M \Phi_i)} .
\]  

(6.5)

In a general form (i.e., the RD approach), the global damping matrix of a BI building with a superstructure having identical story mass, lateral story stiffness and distribution of viscous damping along the height is:

\[
C = \begin{bmatrix}
    c^b + \alpha_m m^s + 2\beta_k k^s & -\alpha_m m^s - 2\beta_k k^s & 0 & \ldots & 0 \\
    -\alpha_m m^s - 2\beta_k k^s & \alpha_m m^s + 2\beta_k k^s & -\beta_k k^s & \ldots & \vdots \\
    0 & -\beta_k k^s & \ddots & \ldots & 0 \\
    \vdots & \ddots & \ddots & \ddots & \vdots \\
    0 & \ldots & 0 & \alpha_m m^s + 2\beta_k k^s & -\beta_k k^s \\
\end{bmatrix}_{(n+1) \times (n+1)}
\]  

(6.6)

Assuming a rigid superstructure, the first-mode shape of a BI building is \( \Phi_1 = \text{ones}(1, n + 1) \). Therefore, the transformed global \( C \) and \( M \) matrices are:

\[
\Phi_1^\top C = [c^b, 0, \text{ones}(1, n - 1)\alpha_m m^s]_{n+1} ,
\]  

(6.7a)

\[
\Phi_1^\top C \Phi_1 = c^b + (n - 1)\alpha_m m^s ,
\]  

(6.7b)

\[
\Phi_1^\top M \Phi_1 = (n + 1) m^s .
\]  

(6.7c)

Substituting Equations 6.7(b) and 6.7(c) into Equation 6.4, the assigned viscous damping to the first mode of the BI building is

\[
\xi_1 = \frac{c^b + (n - 1)\alpha_m m^s}{2\omega_1 (n + 1) m^s} = \frac{c^b}{2\omega_1 (n + 1) m^s} + \frac{(n - 1)\alpha_m}{2\omega_1 (n + 1)} .
\]  

(6.8)

For a rigid superstructure, the first term in Equation 6.8 is basically the target damping, \( \xi^b \), and hence, the second term is the additional (unwanted) damping caused by the superstructure viscous damping modeling approach, denoted as \( \xi_1^{\text{error}} \) herein:

\[
\xi_1^{\text{error}} = \frac{(n - 1)\alpha_m}{2\omega_1 (n + 1)} .
\]  

(6.9)

As seen, this error term is independent of the coefficient \( \beta_k \) meaning that the KD term does not impart an extra damping to the first mode of the BI building. As an example, the details of
computing $\xi_1^{\text{error}}$ for the first mode of a BI building with a rigid superstructure for which the RD approach is used to construct $\mathbf{C}^s$ is provided next. For the RD scenario, $\alpha_m = 2\xi^s \omega_1 \omega_2 / (\omega_1 + \omega_2)$. Hence, if $\omega_1$ and $\omega_2$ are the NI superstructure frequencies, Equation (9a) can be used to estimate $\xi^{\text{error}}$ (note that for the rigid superstructure, $\omega^s_1 \rightarrow \infty$). If $\omega_1$ and $\omega_2$ are the BI frequencies, Equation (6.10b) can be used to compute this parameter (in this case, $\omega_2$, i.e., the second frequency of the BI system, which is essentially the fundamental frequency of the superstructure, approaches infinity).

$$\xi_1^{\text{error}} = \lim_{\omega_1 \rightarrow \infty} \frac{(n - 1) 2 \xi^s \omega_1^s \omega_2^s / (\omega_1^s + \omega_2^s)}{2 \omega_1 (n + 1)} = \infty,$$  \hspace{1cm} (6.10a)

$$\xi_1^{\text{error}} = \lim_{\omega_2 \rightarrow \infty} \frac{(n - 1) 2 \xi^s \omega_1 \omega_2 / (\omega_1 + \omega_2)}{2 \omega_1 (n + 1)} = \lim_{\omega_2 \rightarrow \infty} \frac{\xi^s \omega_2}{(\omega_1 + \omega_2)} \frac{(n - 1)}{(n + 1)}
= \frac{\xi^s (n - 1)}{(n + 1)} \hspace{1cm} (6.10b)$$

The $\xi^{\text{error}}$ parameter for the six different superstructure viscous damping strategies used in this study assuming a rigid superstructure (i.e., $\omega^s_1 = \infty$) is computed and shown in Table 6-2.

<table>
<thead>
<tr>
<th>$\xi_1^{\text{error}}$ parameter for six different superstructure viscous damping strategies in a BI building when the superstructure is rigid</th>
<th>KD</th>
<th>MD</th>
<th>RD</th>
</tr>
</thead>
<tbody>
<tr>
<td>NI modes used</td>
<td>0</td>
<td>$\infty$</td>
<td>$\infty$</td>
</tr>
<tr>
<td>BI modes used</td>
<td>$\xi^s (n - 1) / (n + 1)$</td>
<td>$\xi^s (n - 1) / (n + 1)$</td>
<td>$\xi^s (n - 1) / (n + 1)$</td>
</tr>
</tbody>
</table>

Results of this section, summarized in Table 6-2, suggest that the KD approach, regardless of whether the coefficient multiplying $\mathbf{K}^s$ is based on the NI or BI fundamental frequency, can reliably specify the target damping to the fundamental mode of the BI building. Using the MD or RD approaches and computing the coefficient multiplying $\mathbf{M}^s$ based on the NI modes, might result in assigning significantly large damping to the fundamental mode of the BI building, which are beyond the damping provided by the isolation system, and consequently the fictitious mitigation of the first-mode dominated responses. Using the MD or RD methods based on the BI modes leads to an error in the specified first-mode damping that, for a given number of stories, is a function of
the superstructure viscous damping ratio in its fundamental mode, \( \xi_s \). As seen in Table 6-2, for these methods, the error approaches \( \xi_s \) as the number of stories increases.

### 6.5.2 Generalized Modal Analysis

In a BI building, the disparity between the relatively high damping of the isolation system and low damping of the superstructure implies that the combined system damping is non-classical. In other words, because the system damping is not uniformly distributed throughout the combined system, the global square matrix, \( \Phi^T C \Phi \), is not diagonal, and the equations of motion are coupled. The damping matrix is included in the characteristic equations resulting in complex-valued mode shapes. For a non-classically damped system, a solution to the Eigenvalue problem can provide eigenvalues and eigenvectors (Chopra 2015):

\[
\lambda A \mathbf{K} + B \mathbf{K} = 0,
\]

where

\[
\mathbf{K} = \{ \lambda \Phi \}, \quad A = \begin{bmatrix} 0 & M \\ M & C \end{bmatrix}_{2n' \times 2n'}, \quad B = \begin{bmatrix} -M & 0 \\ 0 & K \end{bmatrix}_{2n' \times 2n'}.
\]

\( \lambda \) is an eigenvalue and \( \mathbf{K} \) is the associated eigenvector of \( 2n' \) elements, where \( n' = n + 1 \) is the total number of degrees of freedom in the BI building. The lower \( n' \) elements of \( \mathbf{K} \) represent the desired modal displacement, \( \Phi \), and the upper \( n' \) elements represent the modal velocity, \( \lambda \Phi \). The modal frequencies, \( \omega_n \), and modal damping ratios, \( \xi_n \), can be computed by Equations (6.12a) and (6.12b), respectively:

\[
\omega_n = |\lambda_n|.
\]

\[
\xi_n = \frac{-\text{Re}(\lambda_n)}{|\lambda_n|}.
\]

For a non-classically damped system the effective mass at the i-th mode is (Song et al. 2006):

\[
(m_i)_{\text{eff.}} = -2\text{Re} \left\{ \frac{(\Phi_i^T K r)(\Phi_i^T M r)}{\lambda_i a_i} \right\}.
\]

where \( a_i = \Phi_i^T (2\lambda_i M + C) \Phi_i \). In this section, generalized modal analysis is conducted for BI buildings with isolation periods of 1.2, 1.8 and 2.4 s. These BI buildings are based on a six-story linear shear-building model with a superstructure period of 0.6 s. The superstructure has identical
story mass, story stiffness and distribution of viscous damping along the height. An additional mass of the same value as the typical story mass is added right above the isolators. For each isolation system period, four different combinations of superstructure and isolation system damping ratios are considered. Hence, overall, 12 different BI building models are developed. Table 6-3 presents the results of Generalized modal analysis for the considered BI buildings in terms of the modal periods and modal mass participation ratios (MMPRs). In this table, the modal analysis results for the baseline NI six-story structure are also presented.

Table 6-3 Modal periods and modal mass participation ratios (MMPRs) for the six-story building with a NI superstructure period of 0.60 s

<table>
<thead>
<tr>
<th>Superstructure mode number</th>
<th>Non-isolated building</th>
<th>Base-isolated building</th>
<th>Base-isolated building</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T (s)</td>
<td>MMPR</td>
<td>T (s)</td>
</tr>
<tr>
<td>1</td>
<td>0.60</td>
<td>0.870</td>
<td>0.30</td>
</tr>
<tr>
<td>2</td>
<td>0.20</td>
<td>0.089</td>
<td>0.16</td>
</tr>
<tr>
<td>3</td>
<td>0.13</td>
<td>0.027</td>
<td>0.12</td>
</tr>
<tr>
<td>4</td>
<td>0.10</td>
<td>0.010</td>
<td>0.09</td>
</tr>
<tr>
<td>5</td>
<td>0.08</td>
<td>0.004</td>
<td>0.08</td>
</tr>
<tr>
<td>6</td>
<td>0.07</td>
<td>&lt;1.0E-03</td>
<td>0.07</td>
</tr>
</tbody>
</table>

The modal damping ratios of the BI buildings are evaluated next. As observed in Equation (6-2c), in the RD approach, the coefficients $\alpha_m$ and $\beta_k$ are defined to assign the same modal damping ratio, $\xi$, to the $i$-th and $j$-th structural modes of vibration. These modes are usually selected as the first mode and a higher mode at which the cumulative MMPR exceeds a relatively large predefined value (e.g., 90 or 95%). With this approach, the specified modal damping ratio over the frequency range that includes the majority of the modal mass participation is limited to $\xi$. For a BI building, selecting the first two modes is usually sufficient to achieve a high cumulative MMPR, and hence, this practice is used for the primary analyses conducted in this paper. Later, the effect of this selection on the structural responses of BI buildings is discussed. Figures 6-2(a) to (c) depict the modal damping ratios of three representative BI buildings calculated based on Equation (6-12b). These three BI buildings are modeled based on the combinations of upper and lower bound parameter values for the isolation system and superstructure viscous damping ratios. Table 6-4 illustrates the damping ratios specified to the first three modes of different BI building models considered.
An evaluation of the results reveals that the predictions of Generalized and Classical modal analyses in terms of modal damping ratios, modal periods and MMPPRs are comparable. Results illustrate that with increasing the isolators damping ratio, the difference between the modal damping ratios obtained from the two approaches increases; for the critical case (i.e., the superstructure’s and isolation system’s damping ratios of 2\% and 30\%, respectively), the difference between the estimates for the first-, second- and third-mode damping ratio using these methods is up to 1, 3 and 18\%, respectively. It is also observed that with increasing the isolation system’s damping, estimates of modal periods based on the two approaches slightly diverge; the difference for the first-mode periods is up to 3\%, and for the higher-modes is limited to 1\%. Finally, the estimations of the two approaches for the MMPPRs in the critical case (i.e., an isolation system period and damping ratio of 1.2 s and 2\%, respectively) differ by only 1\%.

![Graphs showing modal damping ratios for three selected BI configurations assuming different approaches of modeling the superstructure’s viscous damping](image)

**Figure 6-2** Modal damping ratios for three selected BI configurations assuming different approaches of modeling the superstructure’s viscous damping

**Table 6-4** Modal damping ratios for the 12 BI configurations assuming different approaches of modeling the superstructure’s viscous damping

<table>
<thead>
<tr>
<th>Damping scenario</th>
<th>T^b = 1.2 (s)</th>
<th>T^b = 1.8 (s)</th>
<th>T^b = 2.4 (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Modal damping %</td>
<td>Config.</td>
<td>Modal damping %</td>
</tr>
<tr>
<td></td>
<td>( \xi_1 ) ( \xi_2 ) ( \xi_3 )</td>
<td>( \xi_1 ) ( \xi_2 ) ( \xi_3 )</td>
<td>( \xi_1 ) ( \xi_2 ) ( \xi_3 )</td>
</tr>
<tr>
<td>MD-NI</td>
<td>4.6 1.6 1.0</td>
<td>6.3 1.3 0.8</td>
<td>7.7 1.1 0.7</td>
</tr>
<tr>
<td>KD-NI</td>
<td>1.6 4.5 7.6</td>
<td>1.8 4.3 7.5</td>
<td>1.9 4.1 7.4</td>
</tr>
<tr>
<td>RD-NI</td>
<td>3.9 2.3 2.7</td>
<td>5.1 2.0 2.5</td>
<td>6.3 1.9 2.4</td>
</tr>
<tr>
<td>RD-BI</td>
<td>3.0 1.3 0.8</td>
<td>3.2 0.9 0.5</td>
<td>3.3 0.7 4.0</td>
</tr>
<tr>
<td>MD-BI</td>
<td>1.8 7.9 14.6</td>
<td>1.9 11.4 21.7</td>
<td>3.2 2.4 3.7</td>
</tr>
<tr>
<td>RD-BI</td>
<td>2.8 2.6 3.5</td>
<td>3.0 2.5 3.6</td>
<td>3.2 2.4 3.7</td>
</tr>
<tr>
<td>MD-NI</td>
<td>18.6 9.1 5.1</td>
<td>24.3 6.6 3.6</td>
<td>28.6 5.4 3.0</td>
</tr>
<tr>
<td>KD-NI</td>
<td>11.1 16.3 21.6</td>
<td>13.2 14.2 20.3</td>
<td>17.6 4.3 2.1</td>
</tr>
<tr>
<td>RD-NI</td>
<td>16.7 10.9 9.3</td>
<td>21.5 8.6 7.8</td>
<td>24.9 7.3 7.2</td>
</tr>
<tr>
<td>RD-BI</td>
<td>14.6 8.4 4.6</td>
<td>16.8 5.7 2.8</td>
<td>17.6 4.3 2.1</td>
</tr>
<tr>
<td>MD-NI</td>
<td>11.6 24.9 39.2</td>
<td>13.5 32.0 56.0</td>
<td>14.2 40.2 73.5</td>
</tr>
<tr>
<td>KD-NI</td>
<td>14.0 11.7 11.5</td>
<td>16.3 9.6 10.7</td>
<td>17.2 8.5 10.4</td>
</tr>
</tbody>
</table>
The discussion presented in Section 6.5.1 regarding the target first-mode damping is used to investigate which superstructure viscous damping scenario provides a reasonable estimate of the BI building damping in the first mode. The most salient observations from Table 6-4 regarding the first-mode damping ratio are summarized below:

(i) The damping specified to the first mode by the KD-NI and KD-BI approaches in all configurations is close to the first-mode target damping. This is consistent with the analytical solution presented in Section 6.5.1.

(ii) If the isolation system and superstructure viscous damping ratios are both relatively small (i.e., 2%), the KD scenarios result in a damping ratio specified to the first mode that is slightly lower than the target (i.e., 0.8-0.9 times the target value). In this case, other damping scenarios (i.e., MD and RD) result in larger first-mode damping ratios that are 1.4-2.3 times the target values for the studied configurations. As a matter of fact, the MD component of the RD approach causes an increase in the assigned damping ratio to the first mode of BI buildings.

(iii) For the intermediate values of the isolation damping (i.e., $\xi_b = 15\%$), only the KD scenarios assign a reasonable damping ratio to the first-mode. This observation is also consistent with the results of Section 6.5.1.

(iv) When the damping ratio of the isolation system is as much as 30% and significantly higher than that of the superstructure, at the BI period of 1.2 s (i.e., an isolation degree, $R$, of 2.0), for all damping scenarios, the specified first-mode damping is lower than the target (i.e., 0.7-0.8 of the target values). In this case, the superstructure is relatively flexible with respect to the isolation system, and the assumption of $c^b = 2\xi^b(m^b + \sum_i m_i^s)\omega^b$ is no longer valid. With increasing the BI period to 1.8 and 2.4 s, this deficiency is not observed.
(v) If the superstructure viscous damping is higher than that of the isolation system (a rare case in practice), the KD scenarios specify a reasonable damping ratio to the first-mode, whereas, the MD and RD methods significantly overestimate this parameter.

(vi) In general, for a given superstructure viscous damping scenario, the greater the separation between the fundamental periods of the NI superstructure and the isolation system, the more the specified first-mode damping ratio exceeds the target damping of the isolation system (this was also reported by Ryan and Polanco (2008)). For example, for the BI configurations with a superstructure and isolation system damping ratios of 5 and 15%, respectively, at the isolation period of 1.2, 1.8 and 2.4 s, the first-mode damping prediction of the RD-NI is 16.7, 21.5 and 24.9%, respectively (note that the target damping for the three cases is 15%). As another example, for the BI configurations with superstructure and isolation system damping ratios of 2%, the first-mode damping ratio predicted by the RD-BI corresponding to the three BI periods are 2.8, 3.0, and 3.2%, respectively.

Under the premise that the second mode of the BI building is controlled by the superstructure mass and stiffness, this mode is, for practical purposes, the first superstructure mode. Hence, one can expect that the damping ratio of the second mode of a BI building be comparable with the superstructure viscous damping ratio at its first mode. This premise is used to evaluate the assigned damping ratio to the second mode of the BI buildings by different damping scenarios. The most salient observations from Table 6-4 regarding the higher-mode damping ratios of the BI buildings are summarized below:

(i) As consistently seen, the adopted approach for constructing the superstructure’s damping matrix has a significant influence on the modal damping ratios assigned to higher modes. For example, for the building with an isolation system period of 2.4 s, superstructure and isolation system damping ratios of 5 and 15%, respectively, the damping ratio specified to the second mode by different approaches varies from 5.7 to 32%. Hence, it is expected that variation in the higher-mode dominated responses due to the superstructure viscous damping model be significant.

(ii) The KD scenarios, which can satisfactorily provide a first-mode damping ratio close to the target value, assigns a relatively high damping to the higher modes (in some modes the damping
ratio is larger than unity meaning an over-damped system). This could potentially result in a fictitious mitigation of higher-mode dominated responses.

(iii) The MD approaches provide lower-bound estimates of the second-mode damping in all BI configurations, which in some cases is smaller than the target second-mode damping. When the isolation damping ratio is as low as 2%, the second-mode damping assigned by the RD approaches is fairly close to the target. For relatively high values of the isolation system damping ratio (i.e., 15 and 30%), none of the damping scenarios evaluated results in a second-mode damping that is close to the target. For example, for the configuration with isolation system damping and period of 30% and 1.8 s, and a superstructure damping ratio of 2%, the resulting second-mode damping by different damping scenarios ranges from 11 to 21.1%, which is significantly larger than the target value of 2%. When the isolation system damping is relatively high, the coupling effect between the first-mode (i.e., isolated mode) and second mode (i.e., superstructure fundamental mode) becomes more dominant, and as a result, the isolation-system damping increases the second-mode damping (as illustrated in Section 6.6, this apparent increase in damping ratio is not always effective for reducing higher-mode dominated responses).

The two aforementioned premises regarding the first-mode and second-mode damping ratios of a BI building serve as the basis for the evaluation of the structural response estimation of the considered viscous damping models. If one approach specifies a modal damping ratio higher that these premises, it can result in an underestimation of demands controlled by that specific mode of the BI building. With these premises, the results of this section suggest that either the KD-NI or KD-BI can result in a more reliable estimate of first-mode dominated responses, whereas other methods can underestimate these responses. The KD methods may not be able capture the high-frequency responses of BI buildings. In this case, the RD approaches can assign a more reasonable damping ratio to the higher modes. In the next section, the correlation between results from modal analysis (i.e., MMPRs and damping ratios) and structural responses is evaluated using response history analyses.

6.6 Relationship Between Modal Analysis Results and Dynamic Responses

As observed in Table 6-3, the higher-mode mass participations for all three BI configurations are relatively low (i.e., MMPR less than 1%). Hence, a fundamental question herein is whether or not
the contribution of higher modes to response quantities of the BI buildings is significant. In BI buildings modal equations are coupled, and as shown by Kelly (1999), with increasing the isolation system damping, this coupling effect increases. In other words, with increasing the isolation system damping, the contribution of the coupling terms to the dynamic responses of superstructure increases. This coupling effect can increase the seismic responses of the superstructure. From this discussion, one can conclude that when the isolation system damping is significant, MMPRs may not be adequate to evaluate the importance of higher-mode dominated responses such as floor spectral accelerations at short periods. In this section, response history analyses are conducted to investigate whether or not the predictions of modal analysis using the various approaches to model the superstructure’s viscous damping are consistent with their corresponding results from response history analyses. For instance, an assessment of the significance of dynamic responses at modes associated with relatively high damping ratios is conducted. Moreover, an evaluation of the ability of low MMPRs for higher modes to serve as reliable predictors of the relative contribution of higher modal responses to the total response of the system is performed.

For the response history analyses, 20 ground motions recorded at sites located at distances greater than or equal to 10 Km from the closest fault rupture zone (usually referred to as far-field) are selected. Figure 6-3 depicts the 2%-damped ground spectra of the selected ground motion records. In this figure, pseudo-spectral acceleration ordinates, $S_a$, are normalized to the peak ground acceleration (PGA) of the individual corresponding records (herein after, the term spectral acceleration is used in lieu of pseudo-spectral acceleration, for brevity). For the response history analyses, the Direct Time Integration method is used for solving the equations of motion.

![Normalized 2%-damped ground pseudo-spectral acceleration for the 20 selected ground motion records](image)

For a given structural model and ground motion, the acceleration response at different floor levels is obtained and used as input for a SDOF analysis program to develop its corresponding
elastic floor spectrum. Figures 6-4(a) to (c) illustrate the mean normalized 2%-damped roof spectral acceleration (FSa/PGA) responses for three BI structures with the same superstructure and isolation periods of 0.6 and 1.8 s, respectively, but different superstructure and isolation system damping ratios. Floor spectra are generally used for designing acceleration-sensitive nonstructural components (NSCs). Recent studies suggest that using 2% viscous damping ratio is a more rational choice than the prevalent 5% at least for typical electrical and mechanical equipment (Anajafi and Medina 2018b). Therefore, in this study, floor response spectra are developed based on a 2% NSC damping ratio. An evaluation of the result illustrates that for a floor spectrum period, the variation between the predicted values of FSa/PGA obtained from different superstructure viscous damping scenarios, especially for periods in tune with the modal periods of BI buildings, is significant. The KD-BI and MD-BI approaches lead to the minimum or maximum FSa/PGA values in the vicinity of the higher-modes, respectively. The MD-NI and KD-NI methods result in the minimum or maximum FSa/PGA value in the vicinity of the fundamental period of the BI building, respectively. As consistently seen in Figures 6-4(a) to (c), the MD approaches, especially MD-BI, does not significantly suppress the floor acceleration spectral ordinates in the vicinity of the higher modes. These observations are consistent with the results of Section 6.5.2 (i.e., the MD methods assign a second-mode damping ratio that is smaller than the target; whereas the KD approaches result in second-mode damping greater than the target). When the isolation system damping ratio is relatively low (i.e., Figures 6-4a and c), the MD scenarios significantly mitigate the floor acceleration spectral values in the vicinity of the fundamental mode. This was also predicted by the modal analysis results where it was shown that MD methods overestimate the first-mode damping of the BI buildings. Similar interpretations can be made for the trend observed in the modal damping and floor spectral values obtained from the BI models developed based on the RD and KD scenarios.

Based on the assigned second-mode damping ratios, either the RD-NI or RD-BI approach can be regarded as a more reasonable approach for capturing higher-mode dominated responses. This statement can be also rationally validated based on the spectral values in the vicinity of the second mode corresponding to the RD approaches: when the isolation system damping is relatively low (e.g., 2%), the superstructure and isolated raft are essentially decoupled. In this case, given the relatively high stiffness of the superstructure with respect to the isolation system, the
superstructure behaves like a semi-rigid body (or close to a SDOF system). This results in the mitigation of higher-mode responses. When, in addition to the low damping of the isolation system, the superstructure is highly-damped (e.g., 10% as considered in Figure 6-4c), the superstructure relative deformations (e.g., story drifts) decrease even more, and the superstructure behaves as a perfect rigid body. In this case, higher-mode effects (i.e., spikes in the short period range of the floor spectra) are significantly mitigated. An evaluation of Figures 6-4(a) to (c) reveals that RD approaches satisfactorily meet these expectations (i.e., the higher-mode spike in 6.4a, b and c are respectively, just mildly, not significantly, and completely mitigated).

A comparison of the floor acceleration response spectra of the two BI configurations presented in Figures 6-4(a) and (b) reveals the presence of an apparent (fictitious) damping ratio at the second mode of the BI buildings that is relatively high. As previously observed in Table 6-4, the second-mode damping ratio of the BI configuration with superstructure and isolation system damping ratios of 2% (i.e., the configuration studied in Figure 6-4a) are significantly lower than that of the BI with superstructure and isolation system damping ratios 2% and 30%, respectively (i.e., the configuration studied in Figure 6-4b). For example, for the RD-BI method, the second-mode damping ratio of the configurations in 6.4(a) and (b) are 2.5 and 12.2%, respectively (i.e., significantly different), whereas, the FSa/PGA values in the vicinity of the second mode for these buildings are 1.5 and 1.8, respectively (i.e., closely to one another). The system with a significantly larger (apparent) second-mode damping ratio exhibits a larger second-mode response. This observation suggests that increasing the isolation system damping can impart seismic energy to the higher modes although the damping ratio assigned to the higher-modes is apparently high. This
is consistent with the findings of Kelly and Tsai (1985) through mathematical and experimental studies of a five-story steel BI building with low- and high-damping (lead) bearings.

The abovementioned interpretation is consistent with the results presented in Figures 6-4(b) and (c). As observed in Table 6-4, the second-mode damping ratios predicted by the RD-BI method for these two configurations are fairly close (i.e., 12.2 and 9.5%, respectively). However, Figures 6-4(b) and (c) illustrate that their corresponding second-mode responses are substantially different (i.e., 3.0 for the high-damping versus 1.0 for the low-damping isolator). Hence, one can argue that when the isolation system damping is relatively high, the damping ratios assigned to the higher modes become fictitiously high. In the BI building, large values of higher-mode damping ratios are reliable (or effective estimators of the presence of higher-mode effects), only if they are the result of high damping ratios in the superstructure but not in the isolation system. As another important observation, for BI buildings evaluated in this study, the MMPRs of higher modes do not exceed 1% but still, because of the coupling of the modes, their contribution is evident in roof spectra. Therefore, the relatively low MMPR of higher modes can be misleading in terms of providing a quantitative measure of the importance of higher modes in the response of BI buildings.

Floor acceleration response spectra are typically used for the estimation of seismic demands on acceleration-sensitive NSCs, their supports and attachments to the main structure. In the present study, the maximum value of the FSa/PGA in the period range [0-0.5] s, wherein most typical NSCs are situated, is referred to as the peak component acceleration (PCA) of the short-period region. The maximum value of the FSa/PGA over the periods larger than 0.5 s is denoted as the PCA of the long-period region. The PCA/PGA of the short-period region versus the height for different BI buildings are depicted in Figures 6-5(a) to (c). As consistently seen, the adopted superstructure viscous damping model significantly influences the variation of PCA demands along the height.
Similar graphs are shown in Figures 6-6(a) to (c) for the PCA of the long-period region. As seen, except for when the isolation system damping is significantly larger than that of the superstructure (i.e., Figure 6-6b), the superstructure viscous damping scenario can significantly affect the PCA/PGA profiles of the long-period region.

6.7 Variation in Different Structural Response Parameters of Base-isolated Buildings Due to the Selected Superstructure Viscous Damping Model

In this section, response history analyses using BI buildings with different characteristics exposed to 20 far-field records are performed. The objective is to identify relevant superstructure and isolation system characteristics for which peak seismic demands on BI buildings are strongly dependent on the choice of the superstructure viscous damping model. A total of 207 (i.e., 90 three-
story, 81 six-story and 36 12-story) BI building models are developed based on different combinations of the following structural parameters:

\[ n = (3, 6, 12); \text{number of stories} \]

\[ R = T^b / T^s = (2.0: 1.0: 6.0) \text{ where } T^b_{\text{max}} \leq 6.0 \text{ s}; \]

\[ T^s = (0.1n, 0.2n); \text{ for the 12 story building only } 0.1n \text{ is used} \]

\[ \xi^s = (0.02, 0.05, 0.10); \]

\[ \xi^b = (0.02, 0.15, 0.30). \]

Note that the coefficient \( R \) is the ratio of the base-isolation system period, \( T^b \), to the superstructure fundamental period, \( T^s \). For each individual BI building, the superstructure viscous damping matrix, \( C^s \), is constructed using the six introduced viscous damping scenarios (i.e., for each of the 207 BI buildings, six different configurations are analyzed). Hence, for each individual building, six different values are obtained for a given engineering demand parameter (EDP) corresponding to the six different superstructure viscous damping models. Variation in different EDPs caused by the superstructure viscous damping model is identified and quantified. Due to space limitations in this paper, only responses of six-story BI buildings are presented given that similar trends were observed for the three- and 12-story BI buildings.

Three EDPs are considered: mean peak base drift (PBD), mean peak floor acceleration (PFA) and mean peak short-period component acceleration (PCA) responses under the 20 selected records. Hereinafter, the term “mean” is omitted both text and figures for brevity. To evaluate the sensitivity of an EDP to the superstructure viscous damping scenario, the values of the EDP obtained from different superstructure viscous damping scenarios are normalized to the EDP value corresponding to the KD-NI approach (i.e., the most reasonable approach to capture first-mode dominated responses based on Section 6.5). Figures 6-7(a) and (b) illustrate an evaluation of the PBD for several six-story BI configurations. The isolation system period and damping ratio, and the superstructure viscous damping scenario are the parameters varying in these figures. The superstructure period in Figures 6-7(a) and (b) are 0.6 and 1.2 s, respectively whereas the superstructure viscous damping ratio has been fixed at a value of 2% in both figures. The results of a similar evaluation for the superstructure viscous damping ratio of 10% are depicted in Figures 6-8(a) and (b).
As consistently seen in Figures 6-7 and 6-8, for all BI buildings, the KD-NI and MD-NI approaches result in the upper- and lower-bound PBD responses, respectively. Other observations are listed next: (i) the results of the KD-NI and KD-BI approaches are fairly close, especially when the superstructure viscous damping ratio is as low as 2%. (ii) The MD-NI and RD-NI methods can underestimate the PBD response up to 60% and 55%, respectively, for the considered BI buildings; the most critical value of the underestimation (i.e., 60%) occurs for the system with a superstructure viscous damping ratio of 10%, and an isolation system period ratio and damping ratio of 5 and 2%, respectively. (iii) The RD-BI and MD-BI provide fairly close PBD responses, which in the most critical case are 15% below the expected response. These observations are consistent with the trend identified in the first-mode damping ratio obtained from the modal analysis in Section 6.5.2; as observed, the RD and MD approaches specify a damping ratio to the first-mode that is higher than the target damping ratio. The first-mode response underestimation...
caused by the RD and MD models increases with increasing the superstructure viscous damping ratio. In addition, for a constant superstructure viscous damping ratio and a constant isolation system period, increases in the isolation system damping results in responses for all methods that tend to approach the estimation of the KD-NI method.

Figures 6-9 and 6-10 present a similar evaluation for the PFA responses of the BI buildings.

![Figure 6-9](image1.png)  
**Figure 6-9** Normalized peak floor acceleration responses for six-story BI buildings with a superstructure viscous damping ratio of 2% and an NI period of (a) 0.60 s and (b) 1.20 s

![Figure 6-10](image2.png)  
**Figure 6-10** Normalized peak floor acceleration responses for six-story BI buildings with a superstructure viscous damping ratio of 10% and an NI period of (a) 0.60 s and (b) 1.20 s

As seen in Figures 6-9 and 6-10, with increasing either the isolation system period or damping, the variation in responses obtained using different superstructure viscous damping models tends to increase. For the practical range of isolation system period and damping ratio of interest (i.e., $R = 3.0-5.0$ and $\xi^b = 15-30\%$), the normalized PFA responses obtained from different damping scenarios could vary from 0.75 to 1.60 for a given building configuration, which illustrates a significant response sensitivity. In many cases, the MD-NI approach results in upper-bound estimates of the PFA responses. Unless the isolation system damping is as low as 2%, the KD-BI
results provide the lowest estimate of the PFA demands. These results are consistent with the results obtained from the evaluation of PBD responses. Generally speaking, a larger or smaller base-drift response is associated with a smaller or larger floor acceleration response, respectively. Therefore, it is reasonable to infer that the superstructure viscous damping model associated with the lowest PBD response would result in the highest PFA response.

Figures 6-11 and 6-12 illustrate the normalized short-period ($T_{NSC} = 0-0.5$ s) PCA responses for different aforementioned configurations of the six-story BI structure. As seen, in general, the minimum or maximum estimates of the short-period PCA belong to the KD-BI or MD-BI scenarios, respectively. In general, with increasing the isolation system damping, differences between the mean estimates of PCA using the various methods decrease. This trend is more evident in Figure 6-11(a) where the superstructure viscous damping ratio is as low as 2% and the superstructure period is 0.6 s.

![Figure 6-11](image1.png)  
*Figure 6-11 Normalized peak component acceleration responses for six-story BI buildings with a superstructure viscous damping ratio of 2% and an NI period of (a) 0.60 s and (b) 1.20 s*

![Figure 6-12](image2.png)  
*Figure 6-12 Normalized peak component acceleration responses for six-story BI buildings with a superstructure viscous damping ratio of 10% and an NI period of (a) 0.60 s and (b) 1.20 s*
An evaluation of the results depicted in Figures 6-7 through 6-12 illustrates that the PCA, which is the EDP most sensitive to the higher modes in this study, is also the EDP most sensitive to the selection of superstructure viscous damping scenario. An evaluation of the effects of different structural parameters on the sensitivity of EDPs to the selection of superstructure damping model shows that with increasing the isolation system damping while keeping the other two parameters constant, the variation in EDPs decreases. This is consistent for all BI configurations and all EDPs. On the other hand, an increase in the isolation system period while keeping the two other parameters constant, causes an amplification in the variation of EDPs. Results also illustrate that with increasing the superstructure damping, the variation in EDPs increases.

6.8 Sensitivity of EDPs to the Superstructure Viscous Damping Ratio

In this section, the sensitivity of EDPs to the value of the superstructure viscous damping ratio, $\xi^s$, is investigated. To this end, $\xi^s$ is varied from 1 to 10% with increments of 1%. Herein, the KD-NI approach (i.e., the preferred one in terms of the first-mode dominated responses) and the RD-BI approach (i.e., commonly used method in practice and also the preferred one in terms of the higher-mode responses) are considered. The mean structural responses corresponding to a given $\xi^s$ are normalized to the mean responses corresponding to a $\xi^s = 2\%$, which was used as the baseline in Sections 6-5 to 6.7 of this chapter.

Figure 6-13(a) illustrates the sensitivity of the normalized PBD response of the BI configurations with a non-isolated superstructure period of 0.6 s and different isolation characteristics to the value of $\xi^s$ when the KD-NI approach is used. As seen in this figure, for variations in $\xi^s$ from 1 to 10%, the variation in the PBD response is limited to 10%. In this case, for larger isolation period ratios, the PBD responses are weakly dependent on the value of the $\xi^s$. If the RD-BI approach is used, as illustrated in Figure 6-13(b), variations in PBD responses are significantly higher. For example, for the most critical configuration (i.e., $\xi^s = 10\%$, $R = 2.0$, and $\xi^b = 2\%$), the normalized PBD varies from 0.71 to 1.08. These observations illustrate that PBD responses are less sensitive to the value of $\xi^s$ when the KD-NI approach is used for constructing the superstructure viscous damping matrix. The variation in PBD responses occurred when adopting the RD-BI approach results from the error generated in the first-mode damping of the BI
buildings (as observed in Section 6.5, this error is a function of $\xi^s$). Overall, the sensitivity of the PBD responses to the value of $\xi^s$ decreases with increasing either the isolation system period and/or the isolation damping ratio because the isolation system parameters become more dominant than those of the superstructure.

![Figure 6-13](image)

**Figure 6-13** Normalized peak base drift responses for six-story BI buildings with an NI period of 0.60 s and different superstructure viscous damping ratio (a) KD-NI; and (b) RD-BI approaches

Figures 6-14(a) and (b) illustrate the sensitivity of the PFA responses to $\xi^s$ when the KD-NI or RD-BI methods are used, respectively. As seen, in general, when the KD-NI approach is used, with increasing either the isolation system damping or period ratio, the variation in PFA responses increases. For example, for a constant $R$ of 5.0, at $\xi^b$ of 2%, the normalized peak drift response varies from 0.96 to 1.03; however, for a $\xi^b$ of 30%, this parameter ranges from 0.77 to 1.10. As seen in Figure 6-14(b), when the RD-BI is used, the PFA responses are more dependent on the value of $\xi^s$. The results of similar analyses as those performed for PBD and PFA responses are illustrated in Figure 6-15 for the short-period PCA responses. An evaluation of Figures 6-15(a) and (b) reveals that when the RD-BI approach is used, the PCA responses are more strongly dependent on the value of $\xi^s$. The influence of $\xi^s$ on PCA decreases with increasing the isolation system damping ratio. A reverse trend, with the trend less pronounced, is observed when the isolation system period ratio increases. As another observation, in general, the effect of $\xi^s$ is more highlighted on the PCA demands than the PFA and PBD demands.
Figure 6-14 Normalized peak floor acceleration responses for six-story BI buildings with a NI period of 0.60 s and different superstructure viscous damping ratios (a) KD-NI; and (b) RD-BI approaches.

Figure 6-15 Normalized short-period peak component acceleration responses for six-story BI buildings with an NI period of 0.60 s and different superstructure viscous damping ratios (a) KD-NI; (b) RD-BI

An evaluation of the results of Sections 6.7 and 6.8 suggests that for the range of \( \xi_s \) considered, EDPs are more sensitive to the method adopted for modeling the superstructure viscous damping than the value of the target damping ratio, especially when the KD-NI is used for constructing the superstructure damping matrix and first-mode dominated responses are of interest.

6.9 Sensitivity of Epistemic Uncertainties to Ground Motion Characteristics

In the evaluation conducted in the previous sections, mean EDPs were computed based on individual responses from 20 far-field records. In this section, EDPs are evaluated for individual records to be able to assess the sensitivity of these epistemic uncertainties to the ground motion characteristics. As an example, Figure 6-16(a) illustrates normalized PDB responses (normalization is performed with respect to the responses of KD-NI approach) corresponding to
different superstructure viscous damping scenarios for a representative BI building exposed to the set of 20 ground motions. It can be readily observed that the underestimation of different damping scenarios significantly depends on the characteristics of the ground motion records. For example, the PBD underestimation of the RD-NI under records No. 10 and 16 is 11% and 33%, respectively (i.e., significantly different). Figure 6-16(b) illustrates a similar behavior for the PCA responses. These observations highlight the need of an adequate set of ground motions and using the statistical measures such as mean to draw a conclusion regarding the appropriateness of the superstructure damping scenario. Similar trends are observed for PFA responses (the results are not shown herein for brevity). For a given record, the coefficient of variation (COV) for EDPs obtained from different superstructure viscous damping scenarios are computed using Equation (6.14).

\[
\text{COV}_{\text{EDP}} = \frac{1}{\mu_{\text{EDP}}} \sqrt{\sum_{i=1}^{6} (\text{EDP}_i - \mu_{\text{EDP}})^2 / 5.}
\] (6.14)

where \( \mu_{\text{EDP}} = \frac{1}{6} \sum_{i=1}^{6} \text{EDP}_i \) is the average of the values of an EDP obtained from the six superstructure viscous damping scenarios considered. Figure 6-16(b) illustrates COV of different EDPs at each record for a representative BI building.

As seen in Figure 6-16(c), the value of the parameter COV is very sensitive to the ground motion characteristics. For example, for the PFA, the COV at record No. 5 is 0.03 whereas at record No. 11 is 0.29. This observation illustrates that the influence of the choice of the superstructure viscous damping model on structural responses varies from record to record. In other words, the sensitivity of the EDPs to the superstructure viscous damping model depends on
the characteristics of the input ground motion excitation. As another observation, on average, the variation in the PCA responses is larger than the two other EDPs.

6.10 Recommendations for Modeling the Superstructure Viscous Damping

In the previous sections, it was observed that none of the superstructure viscous damping models evaluated can result in a reliable estimation of both higher-mode and first-mode dominated responses of BI buildings. Seismic demand estimation based on the KD-NI approach is satisfactory for first-mode dominated responses but not for higher-mode dominated responses. A reverse statement is true for the RD-BI method in which modal damping is specified to the first two modes of the isolated building. These versions of the KD and RD approaches underestimate the first- or higher-mode dominated responses, respectively. In this study, the underestimation produced by the RD-BI method for all cases is up to 35%, and for the cases with structural parameters within a practical range (e.g., $\xi^s \leq 5\%$ and $\xi^b \geq 15\%$), this underestimation is limited to 10%. The main reason for these underestimations is the relatively high damping imparted to the respective modes of the BI system. It was also observed that the first-mode dominated response obtained from the RD-BI approach is strongly dependent on the value of $\xi^s$. The underestimation made by the KD-NI for the higher-mode dominated responses is significant (in some cases this modeling approach can practically damp out higher-mode effects). A potential solution for these deficiencies is to use the RD-BI with a $\xi^s$ of 2% (instead of the prevalent 5%), and compute $\alpha_m$ and $\beta_k$ based on a lower frequency than $\omega_1^b$ (e.g., selecting a frequency equal to $\frac{1}{2}$ times the first-mode frequency of the BI building). In fact, specifying $\xi^s$ to $\omega_1^b/2$ instead of $\omega_1^b$ results in a lower damping ratio assigned to $\omega_1^b$, which in turn increases the amplitude of first-mode dominated responses to counteract the underestimation inherent to the RD-BI approach.

It was previously shown that the RD-BI approach can also reasonably capture higher-mode dominated responses. This approach for the BI buildings with a high-damping isolation system results in near upper-bound values of higher-mode dominated responses for all the models considered in this study. However, for the low-damping isolators, the estimates of higher-mode dominated response obtained with this method are well below the upper-bound estimate (i.e., the estimate based on the MD-BI approach). Although it was shown that these estimates are reliable, if one is interested in conservatively using the upper limits to quantify higher-mode dominated
responses, using the RD-BI method and computing $\alpha_m$ and $\beta_k$ based on $\omega_1^b/2$ and $\omega_n^b$ may be considered. This method is denoted in this study as the modified RD-BI (MRD-BI) approach. It is well understood that higher-modes can excite typical acceleration-sensitive NSCs attached to buildings. The integrity and operability of NSCs are of paramount importance, especially in critical facilities such as hospitals, emergency response centers, shelters, etc. Hence, using the upper-limit estimates discussed in this section could be justifiable because these values are still significantly smaller than the corresponding values used for designing NSCs in fixed-base buildings. The upper-limit value of the normalized 2%-damped floor spectra across all BI buildings studied herein is equal to 4.2, whereas numerical studies have shown that for fixed-base buildings these values can be larger by factors of up to 4.0 (Anajafi and Medina 2018b).

As an example, Figures 6-17(a) to (c) illustrate the mean FSa/PGA for representative BI buildings when the superstructure viscous damping matrix is constructed based on either the KD-NI (which results in upper-bound estimates for first-mode dominated responses), MD-BI (which results in an upper-bound estimate for higher-mode dominated responses), or the MRD-BI scenarios. As seen, in all BI configurations, the estimation provided by the MRD-BI approach in the vicinity of the first and higher modal periods of the BI buildings is comparable to the upper-bound values, especially when the isolation system period is significantly larger than that of the superstructure, which is usually the case in practice. Similar results are obtained for BI configurations with other isolation system period and damping ratios.

**Figure 6-17** Normalized 0.02-damped roof spectra (FSa/PGA) for the six-story BI structure with a NI period of 0.6 s and an isolated period of 1.8 s assuming different methods for modeling the superstructure’s viscous damping.
6.11 Conclusions

This study addresses challenges encountered in modeling the superstructure viscous damping in linear-viscous base-isolated (BI) buildings. Six different approaches are investigated for constructing the superstructure viscous damping matrix, $C_s$. These approaches are based on the potential combinations of three damping models (i.e., Stiffness-proportional, KD, Mass-proportional, MD and Rayleigh damping, RD) and two methods of computing the coefficients multiplying the superstructure mass and stiffness matrices (i.e., based on the non-isolated, NI, or base-isolated, BI modes) when computing the $C_s$ matrix. BI building models with different properties (i.e., various fundamental periods and damping ratios for the NI superstructure, and different fundamental periods and damping ratios for the isolation system) are studied. Classical and Generalized modal analyses are conducted to estimate the expected dynamic responses of BI buildings. Response history analyses using 20 far-field ground motion records are conducted to identify and quantify variation in different engineering demands parameters (EDPs), including peak base drift (PBD), peak floor acceleration (PFA), and peak component acceleration (PCA) responses, due to the superstructure viscous damping modeling. Results illustrate that the approach implemented to model the superstructure viscous damping can significantly impact the first- and higher-mode dominated responses of BI buildings. This sensitivity is more pronounced for higher-mode dominated responses (e.g., short-period floor spectral accelerations).

Two basic premises regarding the first-mode and second-mode damping ratios of a BI building serve as the basis for the evaluation of the structural response estimation of the considered viscous damping models. One of the basic premises in this study is that the damping specified to the first mode of a BI building should be similar to the isolation system damping, denoted as the target first-mode damping herein. The second premise is that the second mode of the BI building is controlled by the superstructure mass and stiffness, and hence, for practical purposes, the second-mode damping of a BI building should be comparable with the first superstructure mode damping, denoted as the target second-mode damping. With these premises, if a given approach specifies a damping ratio to a BI building vibration mode higher than the respective target damping ratio, this approach can result in an underestimation of demands controlled by that specific mode of the BI building.
The results of the Classical and Generalized modal analysis methods in terms of modal mass participation ratios (MMPRs) and modal damping ratios are fairly close, especially when the isolation system damping is relatively low. Modal analysis illustrates that the KD-NI and KD-BI approaches assign a first-mode damping ratio to the BI building that is close to the target first-mode damping (i.e., the isolation system damping). In other words, KD methods provide the upper-limit estimate of first-mode dominated responses. Other superstructure damping methods result in first-mode damping larger than the target, which in turn spuriously mitigates first-mode dominated responses. The RD and the MD methods may underestimate the magnitude of the first-mode dominated EDPs up to 30 and 60%, respectively. This underestimation is more critical for buildings with either a lower isolation system damping or a longer isolation system period. The MD-BI results in the lowest second-mode damping ratio and consequently provides upper-bound estimates of higher-mode dominated responses. The KD approaches assign relatively high damping ratios to the higher modes, which results in a significant mitigation of higher-mode dominated responses. Except for when the isolation system damping is relatively high, the RD-BI method provides a second-mode damping ratio that is near the target damping of the second mode of the BI building, which is controlled by the superstructure first-mode damping.

For BI buildings with a high-damping isolation system (e.g., 30%), although the modal analysis shows a significantly large damping assigned to the higher modes, this damping is apparent rather than effective. In other words, high-damping isolation systems impart energy to the higher modes of the BI building because the modes of vibration are coupled. Overall, in BI buildings, the high values of the higher-mode damping ratios are reliable indicators of the expected presence of significant higher-mode dominated responses when they are controlled by a high damping ratio specified to the superstructure and not the isolation system. It is also shown that the relatively low MMPRs of higher modes (e.g., 1%) in BI buildings with a high-damping isolation system does not imply that higher mode effects are negligible because of the coupling of vibration modes. However, for the low-damping isolation system, the superstructure is essentially decoupled from the rest of the structure and behaves similarly to a semi-rigid body with relatively small higher-mode effects.

The influence of the choice of the superstructure viscous damping, \( \xi_s \), model on structural responses varies from record to record. The influence of the value of the superstructure viscous
damping on the considered EDPs is not as significant as the influence of the method selected to model the superstructure viscous damping.

It was observed that none of the superstructure viscous damping models evaluated can result in a reliable estimation of both higher-mode and first-mode dominated responses of BI buildings. Seismic demand estimation based on the KD-NI approach is satisfactory for first-mode dominated responses but not for higher-mode dominated responses. A reverse statement is true for the RD-BI method in which modal damping is specified to the first two modes of the isolated building. These versions of the KD and RD approaches underestimate the first- or higher-mode dominated responses, respectively. In this study, the underestimation produced by the RD-BI method for all cases is up to 35%, and for the cases with structural parameters within a practical range (e.g., $\xi_s \leq 5\%$ and $\xi_b \geq 15\%$), this underestimation is limited to 10%. The main reason for these underestimations is the relatively high damping imparted to the respective modes of the BI system. It is also shown that the underestimation made by the KD-NI for the higher-mode dominated responses is significant (in some cases this modeling approach can practically damp out higher-mode effects). A potential solution for these deficiencies is to use the RD-BI with a $\xi_s$ of 2% (instead of the prevalent 5%), and compute $\alpha_m$ and $\beta_k$ based on a lower frequency than $\omega_1^b$ (e.g., selecting a frequency equal to $\frac{1}{2}$ times the first-mode frequency of the BI building). In fact, specifying $\xi_s$ to $\omega_1^b/2$ instead of $\omega_1^b$ results in a lower damping ratio assigned to $\omega_1^b$, which in turn increases the amplitude of first-mode dominated responses to counteract the underestimation inherent in the RD-BI approach.

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6.13 References


Chapter 7

Comparison of the Seismic Performance of a Partial Mass Isolation Technique with Conventional TMD and Base-isolation Systems Under Broad-band and Narrow-band Excitations
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Chapter 7

Comparison of the Seismic Performance of a Partial Mass Isolation Technique with Conventional TMD and Base-isolation Systems Under Broad-band and Narrow-band Excitations


7.1 Abstract

In the present study, a partial mass isolation (PMI) technique is proposed. This approach, through isolating different portions of masses at different stories, can provide a building with multiple inherent vibration suppressors without the need to add extra masses. Optimization of the PMI system’s parameters is conducted for reference structural models with 6, 12, and 20 stories to minimize root-mean-square inter-story drift responses under Kanai-Tajimi filtered Gaussian white noise excitations, while parameter constraints are specified to control isolated components’ (ICs) responses. The seismic performance of the PMI system under excitations with different frequency contents (representing different soil profiles) is compared to that of conventional tuned mass damper (TMD) and base-isolation (BI) systems as baseline configurations. Simulation results indicate that the PMI system with extreme isolated mass ratios of 5% or 90% exhibits dynamic behaviors identical to those of an equivalent TMD or an ideal BI system, respectively. Meanwhile, this technique can resolve some of the inherent difficulties associated with the implementation of TMD (e.g., weight restriction) and BI (e.g., problems due to the superstructure flexibility, overturning moments, and heavy loads) in high-rise buildings.

Keywords: Mass damper; Base isolation; Partial mass isolation; Optimization; Modified Kanai-Tajimi (K-T) filter; Soil conditions.
7.2 Introduction

Tuned mass damper (TMD) and base isolation (BI) systems are two of the most commonly utilized modern seismic protection techniques involving passive control strategies. These techniques provide building designers with a means to adjust structural periods and damping to substantially mitigate the detrimental effects of earthquake ground motions. However, these techniques can suffer limitations that prevent their application, especially to high-rise buildings. For instance, the excessive superstructure flexibility and heavy loads experienced by high-rise buildings can prevent the effective implementation of BI systems, while the large auxiliary mass generally required in TMDs may present significant practical and architectural constraints.

A TMD, initially introduced by Frahm in 1909 (Den Hartog 1956) and then extended by many others, is a well-known passive control device consisting of an auxiliary mass-spring-dashpot system. TMDs are generally designed to oscillate at the same frequency as a primary system but in an opposite phase to attenuate undesired dynamic vibrations induced by wind or earthquake excitations. Since a TMD’s effectiveness is highly dependent on its weight, this system generally requires a heavy mass, especially in high-rise buildings, which may cause practical and architectural problems. In low-frequency buildings, a TMD requires occupying a large space, usually at a top floor, to accommodate large mass damper’s drift responses (i.e. the relative deflection of the lumped TMD mass with respect to its attachment point). Other shortcomings of the TMD system, including its performance sensitivity to the tuning frequency, its performance dependency on the input excitation frequency content and the primary structure dynamic characteristics, are well addressed in the literature (Constantinou et al. 1998; Wang and Lin 2005; Marano et al. 2010; Sgobba and Marano 2010; Tributsch and Adam 2012).

One of the first reports of using the BI technique in the modern era dates back to more than 130 years ago when John Milne isolated a wooden house from the ground by mounting the building on ball bearings (Naeim and Kelly 1999). BI systems can decouple the dynamic responses of a building from the horizontal components of ground excitations by interposing low-horizontal stiffness bearings at the isolation interface (Skinner et al. 1993; Naeim and Kelly 1999). Isolator bearings shift the fundamental frequency of a building away from the dominant frequencies of typical earthquake excitations, protecting the entire building and its potentially vulnerable contents from detrimental effects caused by system resonance. The frequency shift (i.e., period lengthening)
in BI system is usually associated with significant drift responses at the isolation interface that impose large global displacement responses on the superstructure. To limit these excessive responses, supplementary damping devices (e.g., lead rubber bearings, viscous dampers, etc.) are generally incorporated into the isolation system. These supplementary dampers can also lead to the further mitigation of superstructure inter-story drift responses. Alternatively, the global seismic displacement demands present in BI systems can be reduced by incorporating a TMD system or one of its variants attached to the base slab (Palazzo and Petti 1994; Tsai 1995; Palazzo and Petti 1999; Taniguchi et al. 2008; Adam et al. 2017; Di Matteo et al. 2017). For example, Adam et al. (2017), through experimental and numerical analysis of a three-story base-isolated building, illustrated that a tuned liquid column damper (TLCD) system could protect the building against excessive displacements without impairing the efficiency of the BI technique. This objective was achieved by tuning the TLCD frequency to the fundamental frequency of the isolated building.

In recent years, researchers have extensively studied different aspects of BI systems (e.g., Jangid and Datta 1994; Hall et al. 1995; Ramallo et al. 2002; Hong and Kim 2004; Ariga et al. 2006; Constantinou et al. 2006; Li and Wu 2006; Jangid 2007; Providakis 2008; Kilar and Koren 2009; Huang et al. 2010; Branco and Guerreiro 2011; Anajafi et al. 2013; Chen et al. 2013; Shi et al. 2014). Nowadays, it is well acknowledged that the most suitable candidates for the BI technique are low- to mid-rise buildings situated on dense soils with high-frequency motions (Jangid and Datta 1994; Hall et al. 1995; Ariga et al. 2006; Li and Wu 2006). However, the BI system has been used recently in Japan even for rather tall buildings and low-frequency ground motions. As of 2015, approximately 170 isolated high-rise buildings, ranging from 60 to 180 m tall, have been constructed in Japan (Becker et al. 2015). The first base-isolated structure with a height of over 60 m in Japan is the Sendai MTI 18-story building with a height of 84.9 m (1999). Another example is the residential Thousand Tower reinforced concrete (RC) building with 41 stories above the ground and a height of 135.0 m (Komuro et al. 2005). Some studies have illustrated that, unlike the US seismic code provisions, the Japanese code provisions facilitate the implementation of the BI technique in high-rise buildings. For example, Becker et al. (2015) evaluated a base-isolated 32-story RC building with a fixed-base period of 2.57 s under Japanese and US seismic design codes. They showed that assuming the Japanese design loads, the isolation system met all the design criteria whereas the US provision design loads resulted in large overturning moments and
consequently large tensile and compressive forces beyond the limits recommended by the bearings manufacturer. Despite its popularity in Japan, there is no consensus on the effectiveness of BI systems in high-rise structures. The concerns about the application of BI systems in high-rise buildings primarily arise from the relatively long fundamental period of the fixed-base superstructure and the heavy loads imposed on the isolator bearings. Considering the practical range of isolation-system parameters, the ratio of the isolated building’s fundamental period to the fixed-base superstructure’s fundamental period in high-rise structures is typically below two, hence, the isolation technique is less beneficial than for shorter buildings (Becker et al. 2015). Furthermore, P-delta effects and large overturning moments resulting from the high center of gravity in high-rise buildings may lead to difficulties in the design and operation of isolation systems (Ziyaeifar and Noguchi 1998; De Silva 2005).

While a TMD primarily applies to mitigate the wind-induced vibrations (Lin et al. 2001; Saaed et al. 2015), BI systems cannot benefit buildings against wind excitations. This limitation has its roots in the difference between the nature of earthquake and wind loads; unlike the earthquake induced-loads, which are transmitted from the ground to the structure, wind loads directly apply to the building rendering the BI technique practically ineffective. In moderate and strong wind excitations, base-isolated buildings can be even exposed to undesirable vibrations arising from the inserted flexibility at their base, which have been comprehensively addressed in the literature (Kelly and Chitty 1980; Chen and Ahmadi 1992; Liang et al. 2002).

In the present paper, the authors propose a partial mass isolation (PMI) approach that, by isolating portions of masses at different stories, can provide a building with multiple inherent vibration suppressors partially resolving the abovementioned deficiencies associated with the application of conventional TMD and BI systems. In a PMI system, the lower and upper bounds of the isolated mass ratios (i.e., the mass of the isolated components to the total mass of the story) may be dictated by many parameters, such as the structural skeleton’s weight, nonstructural components’ details, the architectural layout of a building, construction and economic considerations, etc. In this study, regardless of such concerns, PMI configurations with extreme isolated mass ratios (IMRs) of 5% and 90% are considered. Decoupling a large portion of a story mass such as 90% may not be realistic because of associated structural and architectural constraints, but it is considered herein to fully characterize the behavior of the proposed system.
and provide a comprehensive view of its potential applications and limitations. Given the flexibility of the PMI system to isolate different sections of the total story mass (e.g., as light as an architectural double-skin façade or as heavy as a section of a floor system including its contents), the authors are currently conducting a parallel study to evaluate the seismic effectiveness of the PMI system with different IMR values in a wide range (i.e., 5%-90%) (Anajafi and Medina 2017).

In this chapter, the seismic effectiveness of the PMI technique is examined in structural models with different fundamental frequencies. A parametric study approach is utilized to optimize the PMI configurations with the extreme IMRs of 5% and 90%. The response quantity to be optimized is the sum of root-mean-square inter-story drift responses of the superstructure under stochastic excitations. To assess the effectiveness of these PMI configurations, conventional TMD and BI systems are also presented as baseline configurations.

### 7.3 Local mass isolation technique

A partial/local isolation technique can be considered as an appropriate alternative for the BI technique in tall buildings or in situations in which isolating the whole structure is not cost-effective. In this technique, instead of isolating an entire building at the base, specific structural systems, components, floors or stories can be selectively isolated. Different types of partial isolation systems included in the literature are generally “upper-story isolation”, “mega-sub-controlled”, and “floor isolation” systems (Figure 7-1). Utilizing the TMD-related principles, the mentioned local isolation systems are capable of converting isolated components into inherent mass dampers to overcome the traditional TMDs’ weight limitations.

![Figure 7-1 Schematic models of different seismic isolation systems.](image-url)
In an upper-story isolation approach, seismic isolators are installed at a higher level unlike a conventional BI system in which isolators are installed at the base. This technique separates roof story or several upper stories from the rest of a building to provide a heavy mass damper for the non-isolated part underneath the isolation interface (Ziyaeifar and Noguchi 1998; Villaverde and Mosqueda 1999; Villaverde 2002; Tsuneki et al. 2009; Chey et al. 2010, 2013; Johnson et al. 2015). A “mega-sub-controlled” building approach, proposed by Feng and Chai (1997), consists of a mega structure as the main structural frame and several isolated multistory substructures. This technique can convert a flexural tall building, where a typical BI is not suitable, into several shear sub-buildings that can benefit from the seismic isolation technology. Meanwhile, isolated substructures can function as multiple TMDs for the entire building without imposing any unnecessary extra weight to the building.

A floor isolation technique is typically applied to protect local contents (e.g., sensitive equipment in a specific room) in a building. In this system, only a particular part of a floor, which requires a higher level of seismic protection than a traditionally-designed structure, is decoupled from the rest of a building through a secondary isolated raised floor (Hamidi and El Naggar 2007; Liu and Warn 2012; Lu et al. 2013; Jia et al. 2014). This version of the partial isolation system, which is also known as a content-protection system, has been rarely designed to reduce seismic responses of an entire building. However, when main floors are isolated from the structural frame, the floor isolation technique can locally and globally benefit a building. When properly designed, this approach can reduce isolated floors’ acceleration responses while making them serve as inherent earthquake vibration suppressors for the entire building. In this system isolator bearings installed at several floor levels can provide additional reliability and redundancy with respect to the conventional TMD and BI systems, such that if a bearing fails, alternative ones are available to dissipate earthquake energy. This advantage would be present at the expense of installing the isolation system at multiple floor levels, which might impair constructability and increase the building’s initial cost. To the best of the authors’ knowledge, only limited research works have been conducted on this version of the partial isolation system. Some of these relevant works are briefly outlined herein.

Sakr (2015) applied a partial isolation system in five-, 25-, and 50-story shear-type buildings under harmonic loads and three historical records. Sakr used identical isolated floors at upper
stories and conducted a parametric study to optimize different configurations. Xiang and Nishitani (2015) optimized a floor isolation system in a 25-story building utilizing a Non-dominated Sorting Genetic Algorithm based on the minimization of the magnitude of the frequency response of the system. They applied identical characteristics for the isolated floors at different stories and assumed that one-third of each story’s mass was isolated. The two mentioned works illustrated that a floor isolation technique, using the TMD-related principle, could significantly reduce inter-story drift responses of the superstructure. In another study, Pourmohammad et al. (2006) applied a mass isolation system in a 10-story shear building assuming that 95% of the mass of each story was isolated with identical bearings. Through exposing the building to a single historical record, they illustrated that the proposed system could considerably reduce acceleration responses of the isolated floors as well as the superstructure inter-story drift responses.

The present chapter studies a PMI system in which masses at different stories are isolated from the superstructure. The PMI system is examined in three reference structural models with six, 12, and 20 stories that can represent typical low-, mid- and high-rise buildings, respectively. To account for multiple performance objectives, various structural responses are defined and evaluated. Parametric studies are utilized to minimize inter-story drift responses of the superstructure while constraints are specified to limit the isolated components (ICs) responses. The seismic performance of the optimized PMI configurations is compared with that of conventional passive TMD and BI systems, which are used as baseline configurations in this study. The ground excitation is modeled as a filtered white noise process to represent earthquake motions with different frequency contents (i.e., narrow- and broad-band excitations). The advantages and limitations of the PMI configurations with extreme IMRs of 5% and 90% are discussed, which provide motivation for further studies to evaluate the PMI configurations with intermediate IMRs.

7.4 Model formulation

The building superstructure is modeled as a linear-elastic shear-type two-dimensional frame (in this manuscript the term “superstructure” refers to the non-isolated portion of the PMI system including the structural frame). In this model, story masses are lumped at floor levels and a single translational (lateral) degree of freedom is assigned at each floor level. The masses of ICs are ideally lumped at floor levels as additional degrees of freedom attached to the superstructure
through a spring-dashpot system. The same assumption is adopted for modeling the mass damper in the TMD system. For the BI system isolation, layers are modeled with linear Kelvin-Voigt elements (i.e., linear stiffness and equivalent viscous damping). The mathematical models of the considered passive control systems for an n-story shear building are illustrated in Figures 7-2(a) to (c).

Figure 7-2  The schematic models of the three passive control strategies for an n-story linear-elastic shear building

To examine the efficiency of the proposed PMI technique in a wide structural frequency range, structural models with six, 12, and 20 stories are selected, which can represent low-, mid- and high-rise buildings, respectively. The story mass and the story stiffness coefficient are constant along the height of each building (i.e., \( m_1^e = m_2^e = \cdots m_n^e \) and \( k_1^s = k_2^s = \cdots k_n^s \)). The most salient dynamic characteristics of the structural models are presented in Table 7-1. Stiffness proportional damping, assuming 2% of the critical damping at the fundamental mode, is applied to the uncontrolled structures as well as to the superstructure (non-isolated part) in the controlled buildings. The 2%-damping assumption implies that the structural frame is mildly damped. For the two passive baseline configurations (i.e., conventional TMD and BI systems illustrated in Figures 7-2a and b), two-dimensional models with the same superstructure characteristics as those mentioned above for the PMI system are developed. The damper’s mass in the baseline TMD system is equal to the overall isolated mass in the corresponding PMI with an IMR of 5%.
Table 7-1  Dynamic characteristics of the uncontrolled test-bed buildings.

<table>
<thead>
<tr>
<th>Building</th>
<th>Stiffness Coeff. ((10^7 \text{ N/m}))</th>
<th>Story Mass ((10^3 \text{ Kg}))</th>
<th>Fundamental FrEquation (rad/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>six-story</td>
<td>3.77</td>
<td>20.00</td>
<td>10.50</td>
</tr>
<tr>
<td>12-story</td>
<td>3.48</td>
<td>20.00</td>
<td>5.23</td>
</tr>
<tr>
<td>20-story</td>
<td>3.36</td>
<td>20.00</td>
<td>3.14</td>
</tr>
</tbody>
</table>

The equations of motion for an \(n\)-story linear elastic shear-type building equipped with a PMI system, schematically illustrated in Figure 7-2©, can be expressed as

\[
\mathbf{M} \ddot{\mathbf{x}}(t) + \mathbf{C} \dot{\mathbf{x}}(t) + \mathbf{K} \mathbf{x}(t) = -\mathbf{M} \mathbf{r} \ddot{x}_g(t) \tag{7.1}
\]

where

\[
\mathbf{M} = \begin{bmatrix} \mathbf{M}^s & \mathbf{0} \\ \mathbf{0} & \mathbf{M}^{IC} \end{bmatrix}_{2n \times 2n} ; \quad \mathbf{C} = \begin{bmatrix} \mathbf{C}^s + \mathbf{C}^{IC} & -\mathbf{C}^{IC} \\ -\mathbf{C}^{IC} & \mathbf{C}^{IC} \end{bmatrix}_{2n \times 2n} ; \quad \mathbf{K} = \begin{bmatrix} \mathbf{K}^s + \mathbf{K}^{IC} & -\mathbf{K}^{IC} \\ -\mathbf{K}^{IC} & \mathbf{K}^{IC} \end{bmatrix}_{2n \times 2n}
\]

\[
\mathbf{M}^s = \text{diag}(m_1^s, m_2^s, \ldots, m_n^s) ; \quad \mathbf{M}^{IC} = \text{diag}(m_1^{IC}, m_2^{IC}, \ldots, m_n^{IC}) ; \quad m_i^s + m_i^{IC} = m_i^e
\]

\[
\mathbf{C}^s = 2\xi_1^s/\omega_1^s \mathbf{K}^s ; \quad \mathbf{C}^{IC} = \text{diag}(c_1^{IC}, c_2^{IC}, \ldots, c_n^{IC}) ; \quad c_i^{IC} = 2\xi_i^{IC} \sqrt{k_i^{IC}m_i^{IC}}
\]

\[
\mathbf{K}^s = \begin{bmatrix} k_1^s + k_2^s & -k_2^s & \cdots & 0 \\ -k_2^s & \ddots & \ddots & \vdots \\ \vdots & \ddots & k_{n-1}^s + k_n^s & -k_n^s \\ 0 & \cdots & -k_n^s & k_n^s \end{bmatrix} ; \quad \mathbf{K}^{IC} = \text{diag}(k_1^{IC}, k_2^{IC}, \ldots, k_n^{IC})
\]

\(\mathbf{M}, \mathbf{C}\) and \(\mathbf{K}\) are the global mass, damping, and stiffness coefficient matrices of the PMI system, respectively. The superscripts \(s\) and \(IC\) stand for the superstructure and isolated components, respectively. \(\mathbf{x} = [x_1^s x_2^s \ldots x_n^s x_1^{IC} x_2^{IC} \ldots x_n^{IC}]^T\) is the displacement vector of the system relative to the ground displacement, \(x_g(t)\). \(\mathbf{r}\) is the influence vector; since displacements are measured relative to the ground, \(\mathbf{r}\) is a column vector of ones. \(m_i^s, m_i^{IC}\) and \(m_i^e\) are the masses of the superstructure, of the IC, and of the entire story at the \(i\)-th story, respectively. \(\xi_1^s\) and \(\xi_i^{IC}\) are the damping ratio of the superstructure at the fundamental mode and the damping ratio of the IC at the \(i\)-th story, respectively. \(\omega_1^s\) is the superstructure fundamental frequency. \(k_i^s\) and \(k_i^{IC}\) are the stiffness coefficients of the superstructure and of the IC at the \(i\)-th story, respectively. Equation (7.1) can be represented in the state space form given by Equation (7.2)
\[
\begin{bmatrix}
\ddot{x}_1 \\
\ddot{x}_2 \\
\ddot{x}_3 \\
\ddot{x}_4
\end{bmatrix}
= \begin{bmatrix}
0_{2n \times 2n} & I_{2n \times 2n} \\
-M^{-1}K & -M^{-1}C
\end{bmatrix}
\begin{bmatrix}
x_1 \\
x_2 \\
x_3 \\
x_4
\end{bmatrix}
+ \begin{bmatrix}
0_{2n \times 1} & 0_{2n \times 2n}
\end{bmatrix}
\begin{bmatrix}
\ddot{x}_g \\
0
\end{bmatrix}
\] \hspace{1cm} (7.2)

or \(z = A z + B u\) where \(z\) is the state vector \([x \ \dot{x}]^T\); \(A\) and \(B\) are the coefficient matrices, and \(u\) is the system input vector. Solving Equation (7.2) results in displacement and acceleration responses that are relative to the ground. Since relative acceleration responses are not relevant seismic demands for design and evaluation, the ground acceleration responses are added to these relative responses to obtain absolute accelerations that are used in the evaluation of the control systems.

Equations of motion for a structure equipped with TMD and BI systems take forms similar to Equation (7.1) where the global mass, damping and stiffness coefficient matrices for the BI system are:

\[
M = \text{diag}(m^\text{BI}, m_1^\text{s}, \ldots, m_n^\text{s}), \quad K = \begin{bmatrix}
k^\text{BI} + k_1^\text{s} & -k_1^\text{s} \\
-k_1^\text{s} & K^\text{s}
\end{bmatrix}_{(n+1)(n+1)};
\]

\[
C = \begin{bmatrix}
c^\text{BI} + c_1^\text{s} & -c_1^\text{s} \\
-c_1^\text{s} & C^\text{s}
\end{bmatrix}_{(n+1)(n+1)}; \quad c^\text{BI} = 2\xi^\text{BI}\sqrt{k^\text{BI}(nm_1^\text{s} + m^\text{BI})}.
\]

and for the TMD system are:

\[
M = \text{diag}(m_1^\text{s}, \ldots, m_n^\text{s}, m^\text{TMD}), \quad K = \begin{bmatrix}
K^\text{s} + k^\text{TMD} & -k^\text{TMD} \\
-k^\text{TMD} & k^\text{TMD}
\end{bmatrix}_{(n+1)(n+1)};
\]

\[
C = \begin{bmatrix}
C^\text{s} + c^\text{TMD} & -c^\text{TMD} \\
-c^\text{TMD} & c^\text{TMD}
\end{bmatrix}_{(n+1)(n+1)}; \quad c^\text{TMD} = 2\xi^\text{TMD}\sqrt{k^\text{TMD}m^\text{TMD}}.
\]

7.5 Ground excitation

The earthquake-induced ground acceleration is modeled as a filtered Gaussian white noise process corresponding to the Kanai-Tajimi (K-T) spectrum (Soong and Grigoriu 1993). This model has been widely used in the literature for studying control systems (e.g., Ramallo et al. 2002; Schmelzer et al. 2010; De Angelis et al. 2012; Anajafi and Medina 2017). In this model, the ground acceleration of the earth surface layer is approximated by the absolute acceleration response of a linear single-degree-of-freedom (SDOF) oscillator subjected to a Gaussian white noise process. The white noise represents the earthquake acceleration at the bedrock, and the linear SDOF oscillator characterizes the filtering effects caused by the soil layers. As demonstrated in Figure 7-3, the oscillator can be defined by two parameters, \(\omega_g\) and \(\xi_g\), which are interpreted as the characteristics frequency and damping ratio of the ground layers, respectively.
Figure 7.3  Modeling of the ground excitation as a K-T filtered white noise process.

The process of deriving the K-T excitation can be expressed as

\[ \ddot{x}_g(t) + 2 \xi_g \omega_g \dot{x}_g(t) + \omega_g^2 x_g(t) = -\ddot{x}_b(t) \] (7.3)

where \( x_g(t) \) is the oscillator’s (i.e., the ground surface’s) displacement relative to the bedrock, and \( \ddot{x}_b(t) = a_b \) is the bedrock acceleration, which is assumed to be a Gaussian zero mean white noise signal, \( \ddot{w}(t) \). The Laplace transform of Equation (7.3) is

\[ X_g(s)\left[s^2 + 2 \xi_g \omega_g s + \omega_g^2\right] = -A_b(s) \] (7.4a)

where which can be expressed as

\[ \frac{X_g(s)}{A_b(s)} = \frac{-1}{s^2 + 2 \xi_g \omega_g s + \omega_g^2} \] (7.4b)

The transfer function with respect to the absolute acceleration of the oscillator, \( \ddot{x}_g(t) = b_g \), is

\[ F_{KT}(s) = \frac{B(s)}{A_b(s)} = 1 + \frac{B_g(s)}{A_b(s)} = 1 + \frac{B_g(s)}{A_b(s)} s^2 \] (7.5a)

Substituting Equation (7.4b) into Equation (5a) results in

\[ F_{KT}(s) = 1 + \frac{-s^2}{s^2 + 2 \xi_g \omega_g s + \omega_g^2} = \frac{2 \xi_g \omega_g s + \omega_g^2}{s^2 + 2 \xi_g \omega_g s + \omega_g^2} \] (7.5b)

The normalized power spectral density function \( (S_\omega / S_0) \) of an example K-T excitation versus the frequency is displayed in Figure 7-4 where \( S_0 \) is the power spectral density of the white noise. As shown, the K-T filter amplifies frequency components of the input white noise around \( \omega_g \), attenuates its high-frequency components, but does not influence the amplitude of low-frequency components. The lack of attenuation at low frequencies incorporates an inconsistency with respect to real ground motions. To modify the K-T spectral shape at low frequencies, a low-cut second
order filter is typically added to the K-T filter (Clough and Penzien 1993). The transfer function representation of the modifier filter in the Laplace domain is

\[ F_{CP}(s) = \frac{s^2}{s^2 + 2\xi_c\omega_c s + \omega_c^2} \]  

(7.6)

where \( \omega_c \) and \( \xi_c \) are the parameters of the additional filter, introduced to produce the desired filtering of low frequencies. Finally, the modified K-T filter is obtained as

\[ F(s) = F_{KT}(s) F_{CP}(s) \]  

(7.7)

which can be incorporated into the state space formulation presented in Equation (7.2). In this approach, the state variables can be obtained by solving a Lyapunov equation (for detailed descriptions of the Lyapunov equation, readers are referred to Lutes and Sarkani (1997)).

The filter parameters expressed in Equations (7.5) and (7.6) control the frequency content of the stochastic excitations. As shown in Table 7-2, the filter parameters are calibrated to represent different soil conditions, and consequently excitations with different characteristics (Der Kiureghian et al. 1991). The central frequency of each excitation, \( \omega_g \), is in the vicinity of the fundamental frequency of each one of the three reference buildings to simulate a near-resonance condition. The block diagram of the structural K-T model is illustrated in Figure 7-5.
Table 7-2  Clough and Penzien K-T model parameters.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$\omega_g$(rad/s)</th>
<th>$\xi_g$</th>
<th>$\omega_c$(rad/s)</th>
<th>$\xi_c$</th>
<th>Resultant Excitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Firm</td>
<td>15.0</td>
<td>0.6</td>
<td>1.5</td>
<td>0.6</td>
<td>High Frequency (Broad-band)</td>
</tr>
<tr>
<td>Soft I</td>
<td>5.0</td>
<td>0.2</td>
<td>0.5</td>
<td>0.6</td>
<td>Low Frequency (Narrow-band)</td>
</tr>
<tr>
<td>Soft II</td>
<td>3.0</td>
<td>0.2</td>
<td>0.3</td>
<td>0.6</td>
<td>Low Frequency (Narrow-band)</td>
</tr>
</tbody>
</table>

### 7.6 Performance objectives and design limitations

Conventionally, the primary aim in structural seismic design has been to reduce inter-story drift responses because excessive inter-story drifts cause seismic damage to structural (e.g., beam and columns) and nonstructural (e.g., partition walls) components. During the last few decades, additional emphasis has been placed on reducing floor acceleration responses to prevent injuries, loss of functionality and mitigate losses associated with damages to nonstructural components (e.g., suspended ceilings), equipment (e.g., HVAC systems), and contents (e.g., computer servers). In this study, the root-mean-square inter-story drift response of the superstructure is selected as the primary objective (OF) to be minimized. As illustrated in the results section (Figures 10-12), adopting this strategy has the added benefit of also improving the global acceleration responses of the considered passive control systems. The optimization procedure adopted in this study is described in the following paragraphs.

For a given passive control system (e.g., a TMD, PMI or BI system), excited by an assumed K-T process, an optimal solution of the system parameters (e.g., tuning frequency and damping ratio in a TMD system) can be derived by minimizing the average normalized root-mean-square (ANRMS) of inter-story drift responses:

$$J_{\text{drift}}^s = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(x_i^s - x_{i-1}^s)_c}{\text{RMS}(x_i^u - x_{i-1}^u)_u}$$

where $x_i^s$ is the superstructure displacement at the $i$-th floor level; $x_i^s$’s are relative displacements with respect to the ground (i.e., $x_0^s = 0$). The subscripts u and c stand for the uncontrolled and controlled systems, respectively. After designing the control system through the minimization of $J_{\text{drift}}^s$, the overall acceleration responses of the optimized system can be evaluated using a weighted OF, $J_{\text{accel.}}^w$.
\[ J_{\text{accel}}^w = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(\ddot{y}_i^s) \cdot c \left( 1 - \text{IMR}_i \right) + \text{RMS}(\ddot{y}_i^\text{Moving}) \cdot c \cdot \text{IMR}_i}{\text{RMS}(\ddot{y}_i^u)}, \quad \text{For PMI and BI} \quad (7.9a) \]

\[ J_{\text{accel}}^w = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(\ddot{y}_i^s) + \text{RMS}(\ddot{y}_i^\text{Moving}) \cdot c \cdot \rho_i}{\text{RMS}(\ddot{y}_i^u) \cdot (1 + \rho_i)}, \quad \text{For TMD} \quad (7.9b) \]

where \( \ddot{y}_i^s \) and \( \ddot{y}_i^\text{Moving} \) are the absolute acceleration responses (i.e., the total acceleration including the ground acceleration) of the superstructure and of the moving components (i.e., damper’s mass in TMD, ICs in iPMI, and the entire floor in BI system) at the \( i \)-th story, respectively. \( \text{IMR}_i = m_i^\text{IC} / m_i^e \) is the isolated component’s mass ratio at the \( i \)-th story. The term \( w \) in \( J_{\text{accel}}^w \) implies a weighted OF; according to Equation (7.9a), \( J_{\text{accel}}^w \) combines the non-isolated part and ICs acceleration responses in the PMI system, and renders a single OF which allows comparison of the overall acceleration response of the controlled system with that of the uncontrolled system. Equation (7.9a) can be directly implemented for a BI system equating all IMRs to 1.0 while, for consistency purposes, this equation should be modified as Equation (7.9b) for a TMD system.

In the conventional BI and TMD systems, optimizing the control system parameters through the minimization of the ANRMS inter-story drift responses as the primary OF can lead to significantly large seismic responses of the isolator bearings and damper’s mass, respectively. A similar behavior can cause significant problems in a PMI system given that the ICs may contain sensitive equipment/contents or living areas. Hence, in order to assess displacement and acceleration responses of the moving components in the optimally designed systems, two additional (secondary) OFs, denoted as \( J_{\text{displ}}^\text{Moving} \) and \( J_{\text{accel}}^\text{Moving} \), are defined as Equations (7.10) and (7.11), respectively. These equations provide a quantitative evaluation of the magnitude of the seismic responses of the moving components in the controlled systems (i.e., ICs in the PMI system) with respect to those of their corresponding parts in the uncontrolled systems.

\[ J_{\text{displ}}^\text{Moving} = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(x_i^\text{Moving}) \cdot c}{\text{RMS}(x_i^s) \cdot u} \]  

(7.10)
\[ J_{\text{accel}}^{\text{Moving}} = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(\ddot{y}_i^{\text{Moving}})}{\text{RMS}(\ddot{y}_i^{\text{u}})} \]  

(7.11)

Preliminary simulation results illustrate that, consistent with fundamental structural dynamics principles, if no constrains are applied on the characteristics of the BI system and the PMI system with a high IMR, the optimization process can lead to isolator bearings with unreasonably long periods and high damping ratios. To obtain reasonable and practical design characteristics, two constraints are incorporated into the optimization process. First, the damping ratio of the moving components (ICs in the PMI and the entire isolated building in the BI) is bounded to [0.02-0.30] of the critical, which is deemed to be reasonable for civil engineering structures. Second, using an approximate method, which is mostly based on engineering judgment, the fundamental period of the moving components is limited to a selected design level (see Appendix I). Adopting this latter constraint, the equivalent elastic period of the moving components, \( T_{\text{Equiv.}}^{\text{Moving}} = \left( T_{\text{Moving}}^{\text{Moving}}/\beta_L \right) \) in which \( \beta_L = (\xi^{\text{Moving}}/0.05)^{0.30} \), is limited to 2.50 s. This constraint implies that, for example, for the two extreme damping values of 0.02 and 0.30 the maximum elastic period of moving components, \( T_{\text{Max}}^{\text{Moving}} \), is limited to 1.90 and 4.28 s, respectively.

In a generic form of a PMI system with arbitrary (or dissimilar) ICs at each story, a total of \( 3n \) parameters, where \( n \) is the number of stories, should be optimized (i.e., stiffness, damping, and mass ratio of ICs at each story). However, a PMI system may not be ideal for the design and construction of a building when ICs parameters, especially IMRs, vary from story to story. Hence, in this study ICs are assigned identical characteristics at different stories (i.e., \( m_i^{\text{IC}} = m^{\text{IC}}, k_i^{\text{IC}} = k^{\text{IC}} \) and \( \xi_i^{\text{IC}} = \xi^{\text{IC}}, \) for \( i = 1, 2, 3, \cdots, n \)). For an identical PMI (iPMI) system, once the IMR is specified, only two parameters (i.e., stiffness coefficient and damping ratio of ICs) should be optimized. In this study, considering the limited number of the unknown parameters to be optimized, a parametric study approach is utilized. As a part of a parallel study, the authors evaluate a generic form of the PMI system with dissimilar ICs at different stories, which have the potential to provide a better seismic performance (Anajafi and Medina 2017).

7-15
7.7 Seismic performance of BI and TMD systems considering different K-T excitation parameters

Before optimizing the passive control strategies implemented in this study, it is informative to evaluate the seismic responses of TMD and BI systems with different characteristics (e.g., different tuning period and damping ratio of the damper’s mass in a TMD) exposed to various ground excitation types. The results of this evaluation will also serve as a benchmark for the assessment of the seismic performance of the proposed PMI system. As an example, Figures 6a-6c illustrate the variation of $J_{\text{drift}}^s$ with respect to the normalized period, $T_{\text{TMD}}/T_{\text{UNC}}$, and the damping ratio of the TMD system in the 12-story frame exposed to different ground excitations. Because in this study the evaluation of the seismic responses of the control systems is conducted in relative terms, i.e., with respect to the seismic responses of the uncontrolled structure, OFs with a value smaller or larger than 1.0 imply a performance improvement or worsening, respectively.

![Figure 7-6](image)

*Figure 7-6 Variation of $J_{\text{drift}}^s$ versus the TMD’s parameters for the 12-story building ($\omega_s = 5.23 \, \text{rad/s}$) assuming different soil profiles*

An evaluation of Figures 7-6(a) to (c) reveals that (i) for all three considered excitation types and all TMD configurations except for low-damping ones in Figure 7-6(c), the optimal performance of the TMD, i.e., $(J_{\text{drift}}^s)_{\text{min}}$, is achieved when it is tuned to the uncontrolled building fundamental frequency (i.e., frequency ratio near unity); (ii) The value of optimal $J_{\text{drift}}^s$ in Figure 7-6(b), where the central frequency of the K-T excitation is near the fundamental frequency of the building, is smaller than those of Figures 7-6(a) and 7-6(c). In other words, in the resonance situation an optimal TMD is more effective. Such a behavior was also reported by Villaverde and Koyama (1993) and Bernal (1996). For example, Villaverde and Koyama (1993) showed that in
the case of excitations with narrow band and long duration, such as those recorded on soft soils in Mexico City during the September 1985 earthquake, a properly designed TMD may be able to provide high reductions in structural responses in the resonance situation. Figure 6c shows that when the soil profile is softer than the structure (i.e., the K-T model’s central frequency is less than the superstructure’s fundamental frequency), an optimal TMD system is less effective. This behavior was also illustrated by Wang and Lin (2005). As consistently observed in Figures 6a-6c, when the tuning period ratio is less than 0.5, in most cases, a TMD could amplify the NRMS inter-story drift responses compared to the uncontrolled situation (i.e., \( J_{\text{drift}}^s > 1.0 \)). This amplification is more pronounced when the TMD’s damping ratio is low (e.g., 4% and 6%) and the soil profile is softer than the building (i.e. Figure 7-6c).

Figures 7-7(a) to (c) illustrate the performance of the BI system in terms of \( J_{\text{drift}}^s \) and \( J_{\text{displ.}}^\text{Moving} \) in the 12-story building assuming different soil conditions. According to Figures 7-7(a) to (c), in all cases these two metrics are in conflict with one another. As seen, the minimum \( J_{\text{drift}}^s \) occurs at the maximum possible isolation system’s period and damping ratio, while at this point the value of \( J_{\text{displ.}}^\text{Moving} \) is significantly high (i.e., between 1.58 and 20.0 for different cases in the considered BI period range).

![Figure 7-7](image)

Figure 7-7 Variation of \( J_{\text{drift}}^s \) and \( J_{\text{displ.}}^\text{Moving} \) versus isolator parameters for the BI system in the 12-story building, (\( \omega_s = 5.23 \text{ rad/s} \)) assuming different soil profiles

Similar to the observed trend in the TMD case, the best BI’s performance corresponds to the near-resonance situation (i.e., Figure 7-7b in which the frequency of the fixed-base superstructure is near the central frequency of the K-T excitation), which is consistent with the results of Islam et al. (2012). As seen in Figures 7-7(a) to (c), increasing the isolation system’s period consistently
reduces NRMS inter-story drift responses except for when the soil profile is softer than the fixed-base superstructure (i.e., Figure 7-7c) and the isolation system’s damping ratio is low; as depicted in Figure 7-7(c), for example, when \( \xi^{BI} = 0.04 \) and 0.10, increasing \( T^{BI}/T^{UNC} \) up to 2.6 and 2.1, respectively, can even amplify inter-story drift responses (i.e., \( f_{drift}^S > 1.0 \)). In this situation, for example when \( \xi^{BI} = 0.04 \), to achieve a 10% improvement in \( f_{drift}^S \), the isolation system’s normalized period, \( T^{BI}/T^{UNC} \), should be larger than 2.9. This implies accepting significantly large displacement responses compared to the uncontrolled case (i.e., \( J_{displ.}^{Moving} = 14.6 \) in this example).

Hence, applying the BI system for the 12-story building located on this soil profile is not an appropriate solution.

According to the results depicted in Figures 7-6(a) to (c) and 7.7(a) to (c), both TMD and BI systems are consistently effective in mitigating inter-story drift responses under a firm soil condition (i.e., a broad-band excitation), however, under the narrow-band excitation they are effective only at the near-resonance condition. Another important conclusion obtained from Figures 7-6 and 7.7 is that neither BI nor TMD is robust against changes in the primary structure’s period and the input excitation characteristics. For example, as seen in Figure 7-6(b), a TMD with a damping ratio of 11% at the tuning ratio of 1.18 is very effective (i.e., it can decrease \( f_{drift}^S \) by 56%), however, its effectiveness rapidly decreases with a reduction in the tuning ratio (e.g., for the tuning ratio of 0.90 reduction in \( f_{drift}^S \) is only 24%). Such performance sensitivity is not desirable given that estimating fundamental period of a building is associated with significant uncertainties. The robustness features of the conventional TMDs and BI systems are well addressed in the literature (e.g., see Chung et al. 1999; Wang and Lin 2005; Marano and Greco 2008; Greco and Marano 2016).

### 7.8 Optimizing the passive control systems considering different K-T model parameters

For all passive control systems, the primary OF used in this study (i.e., \( f_{drift}^S \) defined by Equation 7.8) is calculated over a wide range of design parameters (e.g., tuning ratio and damping ratio in the TMD system). The minimum value of \( f_{drift}^S \) that is associated with design parameter values within the admissible ranges (e.g., \( T_{Equiv.}^{IC} \leq 2.50 \) and \( \xi^{IC} \leq 0.30 \) for the iPMI system) is the
optimal solution. As an example, Figure 7-8 demonstrates the variation of $J_{\text{drift}}^S$ versus the control systems’ parameters for the 12-story building exposed to the firm soil based K-T excitation. These results are representative of those obtained for the 6- and 20-story buildings too, but such additional results are omitted herein for brevity.

![Variation of $J_{\text{drift}}^S$ versus passive control systems’ parameters for the 12-story building under the firm soil based K-T excitation (red dots denote the optimal solutions satisfying the defined constraints)](image)

Figure 7-8  Variation of $J_{\text{drift}}^S$ versus passive control systems’ parameters for the 12-story building under the firm soil based K-T excitation (red dots denote the optimal solutions satisfying the defined constraints)

An evaluation of Figures 7-8(a) and (b) reveals that the iPMI system with a low IMR of 5% behaves similarly to its equivalent TMD, whereas Figures 7-8(c) and (d) illustrate that for a large IMR of 90%, the iPMI system is akin to an ideal BI system. Unlike the TMD-like systems (i.e., the TMD and iPMI with a 5% IMR), $J_{\text{drift}}^S$ graphs for the BI-like systems (i.e., the ideal BI and iPMI with a 90% IMR) are no longer cone-shaped with a discernible global minimum. In other words, the optimal solutions for the BI-like systems are dictated by the specified design characteristic upper bounds (i.e., $T_{\text{Equiv.}}^\text{sys.} \leq 2.50$ and $\xi_{\text{sys.}}^\text{sys.} \leq 0.30$). The optimal stiffness coefficient normalized to the superstructure stiffness coefficient, $k_{\text{sys.}}/k_s$, and the optimal damping ratio of different passive control systems for the 12-story building exposed to different K-T excitation models are presented in Figure 7-9(a) and 7-9(b), respectively. As shown in Figure

7-19
7-9(a), the stiffness term in the optimally designed TMD and BI systems is significantly larger than that of individual ICs in the equivalent iPMI configurations. For example, \( k_{\text{sys}} / k_s \) for the TMD system under the firm soil based excitation is 0.01 while this parameter for the iPMI with a 5\% IMR is \( 7 \times 10^{-4} \). The reason for this observation is that in an iPMI system inherent energy absorbers/reflectors are distributed at all stories rather than at a single level. Figure 7-9(b) illustrates that the optimal damping term of the BI-like systems are the same while this parameter for the TMD-like systems is different (especially under Soft Soil II). Furthermore, while for the BI-like systems, soil type has a negligible effect on the optimal damping ratio or the stiffness term, in the TMD-like systems optimal parameters, especially damping ratio, depend on the soil type.

![Normalized stiffness](image1.png)

![Damping ratio](image2.png)

**Figure 7-9** Optimal parameters of the 12-story building under different K-T excitation models (i.e., different soil conditions): (a) normalized stiffness; (b) damping ratio

Optimization for all passive control strategies for the three test-bed buildings assuming different K-T excitation models is performed, and the OFs defined in Equations (7.8) to (7.11) are presented in Figures 7-10 to 7-12 (the application of the BI system in a 20-story building, although challenging, is presented as an ideal baseline to assess the iPMI system’s performance at a high IMR).
Figure 7-10  Performance objectives of optimized passive control systems in the six-story building, $\omega_s = 10.5$ rad/s, assuming different soil conditions and minimizing $J_{\text{drift}}^s$

Figure 7-11  Performance objectives of optimized passive control systems in the 12-story building, $\omega_s = 5.23$ rad/s, assuming different soil conditions and minimizing $J_{\text{drift}}^s$

Figure 7-12  Performance objectives of optimized passive control systems in the 20-story building, $\omega_s = 3.14$ rad/s, assuming different soil conditions and minimizing $J_{\text{drift}}^s$

As seen in Figures 7-10 to 7-12, in terms of the global behavior (i.e., $J_{\text{drift}}^s$ and $J_{\text{accel}}^w$), for all cases the iPMI system at a low or high IMR, on average, performs similarly to an equivalent TMD
or ideal BI system, respectively. The most important observations from Figures 7-10 to 7-12 are summarized below.

1) For most situations, in terms of improving $J_{\text{drift}}^s$, the iPMI system at a 90% IMR outperforms the BI system; other performance criteria of these two systems are very close.

2) Under the firm-soil based K-T excitation (i.e., Figures 9-10a to c, the TMD’s performance in terms of $J_{\text{drift}}^s$ for all three test-bed buildings is approximately the same as the iPMI system at a 5% IMR.

3) When the soil profile is softer than the uncontrolled structure (i.e., Figures 7-10b, 7-10c, and 7-11c), the iPMI at a 5% IMR is more effective than the TMD in terms of $J_{\text{drift}}^s$. However, in terms of other seismic responses, especially $J_{\text{Moving}}$ in Figure 7-11c, the TMD significantly surpasses the iPMI system. In fact, since the entire damper’s mass in a TMD system is placed at the roof floor, as compared to the iPMI system with the same overall mass uniformly distributed at all stories, it experiences less motions to minimize $J_{\text{drift}}^s$.

4) When the soil profile is softer than the uncontrolled superstructure (i.e., Figures 7-10b, 7-10c, and 7-11c), the effectiveness of both BI- and TMD-like systems significantly decreases (as compared to the reverse case when the soil profile is stiffer); in this case a relatively small improvement in $J_{\text{drift}}^s$ can be achieved at the expense of significant increases in other OFs.

5) Under firm soil conditions (i.e., a broad-band excitation), all passive control systems in all three test-bed buildings are effective in reducing $J_{\text{drift}}^s$. However, for the narrow-band motions, considering the achieved benefits (i.e., overall responses reductions in a building) and the associated adverse effects (e.g., significant movements of the ICs), it can be concluded that the studied passive strategies are effective only in the resonance condition.

6) The best performance of all passive control systems is achieved when the K-T excitation’s central frequency is near the fundamental frequency of the building (i.e., the resonance case). This observation is consistent with the results obtained by Wang and Lin (2005) for the TMD and Islam et al. (2012) for the BI.

7) As illustrated in Figures 7-10 to 7-12, optimizing the iPMI system following the adopted procedure in this study can cause the average RMS responses of the ICs to become larger than those of the corresponding parts in the uncontrolled buildings (i.e., $J_{\text{Moving}} > 1.0$). For example, according to Figure 7-10(a), the optimized iPMI system with a 5% IMR is associated with
$J_{\text{accel}}^{\text{Moving}} > 1.0$. This amplification occurs because ICs operate as sacrificial TMDs for the entire building. This behavior should be carefully accounted for and evaluated in the design process. For instance, since acceleration responses of the uncontrolled six-story building, because of its short fundamental period, are already high, $J_{\text{accel}}^{\text{Moving}} > 1.0$, which mean ICs’ acceleration responses larger than the corresponding parts in the uncontrolled building, may cause significant problems, especially if ICs contain acceleration-sensitive components or living areas. A similar discussion applies to the displacement responses of ICs (i.e., $J_{\text{displ.}}^{\text{Moving}}$) in the 20-story building that is long period and exposed to large displacement responses. In these cases, the optimum solution might not be the preferred one, and an alternative solution would be required in the design process in order to avoid high demands on the ICs (see Section 7.9).

### 7.9 A partial solution for controlling the relatively large seismic responses of the damper’s mass and ICs in optimally-designed TMD-like systems

In Section 7.8 it was shown that the proposed optimization procedure can result in significant drawbacks associated with relatively large displacement and acceleration responses of the moving components (i.e., the damper’s mass and ICs) in the TMD-like systems. The amplified acceleration responses can damage isolated units’ contents; large induced IC displacements require large seismic gaps and special bearings, which generate significant architectural and construction challenges. Tributsch and Adam (2012) illustrated that a TMD’s damping ratio larger than the optimal one can significantly reduce the damper’s mass responses, whereas the response reduction of the main structure remains almost unaffected. This approach is investigated for the TMD-like systems studied in this chapter.

As an example, variation of $J_{\text{drift}}^S$ and $J_{\text{displ.}}^{\text{Moving}}$ versus the control system parameters for a TMD system in the 12-story building exposed to the Soft Soil I excitation (the resonant K-T excitation for this building) is illustrated in Figures 7-13(a) and (b), respectively. As shown in Figure 7-13(a), $J_{\text{drift}}^S$ is not very sensitive to the change in the TMD’s damping ratio. However, according to Figure 7-13(b), a relatively small increase in TMD’s damping ratio can significantly decrease $J_{\text{displ.}}^{\text{Moving}}$. An increase in the tuning period ratio has the same positive effect on $J_{\text{displ.}}^{\text{Moving}}$ without deteriorating $J_{\text{drift}}^S$. Similar trends are observed in Figures 7-14(a) and (b) for the iPMI system with a 5% IMR.
As an example, various OFs for a modified TMD and iPMI system are presented in Table 7-3. In the modified configurations (i.e., the intentionally detuned cases), the damping ratios of the ICs and of the damper’s mass are increased from their optimal values of 10 and 13%, respectively, to 25%. For both systems the tuning period ratio, \( T_{\text{sys}} / T^{\text{UNC}} \), is also increased by a factor of 1.25 with respect to the optimal parameter. As seen in Table 7-3, for example, for the iPMI system, the applied modifications decrease \( J_{\text{displ.}}^{\text{Moving}} \) from 2.22 to 1.19, and \( J_{\text{accel.}}^{\text{Moving}} \) from 1.73 to 0.81, while the increase in \( J_{\text{drift}}^{s} \) (i.e., the performance degradation) is only from 0.45 to 0.57. This evaluation reveals that a simultaneous detuning of the TMD’s damping and period ratio with respect to the optimal characteristics is an effective solution to limit the moving component seismic responses.

![Graphs](image.png)

**Figure 7-13**  Variation of: (a) \( J_{\text{drift}}^{s} \); and (b) \( J_{\text{displ.}}^{\text{Moving}} \); versus the TMD system’s parameters in the 12-story building exposed to the resonant K-T excitation (\( \omega_s = 5.23 \text{ rad/s}; \omega_g = 5.0 \) and \( \xi_g = 0.2 \))
7.10 Evaluation of the near-optimal passive control strategies subjected to a set of ground motion records

To further verify the iPMI technique’s effectiveness, as an example, passive control systems optimized in Section 7.8 assuming the firm soil based K-T model are examined by exposing them to the 44 ground motion records of the far-field set provided in FEMA P695 (2009). For the TMD and the iPMI system with a 5% IMR, the modified characteristics, as those discussed in Section 7.9, are used. The FEMA P695 record set includes 22 pairs selected from the PEER NGA database with the following characteristics: moment magnitude between M6.5 and M7.6; either strike-slip or reverse fault mechanism; site class C (soft rock/very dense soil) or D (stiff soil) according to the NEHRP classification; site-source distance between 11.1 and 26.4 km; peak ground acceleration (PGA) between 0.21 and 0.82 g; peak ground velocity (PGV) between 19 and 115 cm/s. The 0.05-damped pseudo-spectral acceleration responses for the selected records are
illustrated in Figure 7-15. As seen, some of these records exhibit large spectral ordinates at long periods (i.e., the potential forward directivity effects) suggesting that the fault distance criterion might not be sufficient to avoid the presence of near-field, forward-directivity effects.

\[
\bar{\text{RMS}}(u_i) = \frac{1}{44} \sum_{j=1}^{44} [\text{RMS}(u_i)]_j
\]

Figure 7-15 0.05-damped ground spectra for the 44 selected ground motions records

The “average of the root-mean-square” (\(\bar{\text{RMS}}\)) responses under the 44 earthquake records are utilized to calculate OFs introduced in Equations (7.8) to (7.11):

\[
\bar{\text{RMS}}(u_i) = \frac{1}{44} \sum_{j=1}^{44} [\text{RMS}(u_i)]_j
\]

where \(u_i\) is the structural response of interest (e.g., inter-story drift) at the \(i\)-th floor level. Figures 7-17(a) and (b) depict \(\bar{\text{RMS}}\) acceleration and displacement responses of the TMD-like systems for the 12-story building, respectively. Figures 7-17(a) and (b) illustrate similar results for the BI-like systems.

Figure 7-16 \(\bar{\text{RMS}}\) acceleration (\(\text{m/s}^2\)) and displacement (\(\text{m}\)) responses of the 12-story building equipped with near-optimal TMD-like strategies subjected to the 44 ground motion records
As seen in Figure 7-16(a) and (b), the RMS acceleration and displacement responses of the superstructure, in both TMD-like systems at all stories, are consistently reduced with respect to the uncontrolled system responses. The acceleration responses of the ICs at all stories is less than the corresponding floor accelerations in the uncontrolled system while their displacement responses, especially at the bottom stories, exceed the floor displacement responses of the uncontrolled system.

Figure 7-17 RMS acceleration (m/s²) and displacement (m) responses of the 12-story building equipped with near-optimal BI-like strategies subjected to the 44 ground motion records

A similar evaluation of Figures 7-17(a) and (b) for the BI-like systems illustrate that the iPMI with an IMR of 90% can divide the building into two parts: flexible ICs with significantly reduced acceleration responses and a relatively stiff superstructure with significantly reduced displacement responses with respect to those experienced by the uncontrolled structure. The magnitude of the displacement responses of the ICs are consistent with those generated by the rigid body movement in the BI system.

Table 7-4 presents different OFs for the three test-bed buildings subjected to the earthquake records. As shown, all considered passive control strategies could significantly decrease inter-story drift responses in the three selected test-bed buildings. Similar to the results obtained in the previous sections, the iPMI system with a low or high IMR performs like an equivalent TMD or BI system, respectively. The TMD system consistently exhibits a better performance than its equivalent iPMI configuration (i.e., an IMR of 5%) in all example buildings. An iPMI system with a high IMR of 90% in most cases outperforms the ideal BI system.
Table 7-4 Various OFs (based on RMS responses) for the near-optimal control strategies in the three test-bed buildings subjected to the 44 ground motion records.

<table>
<thead>
<tr>
<th>Sys. Type</th>
<th>six-story</th>
<th>12-story</th>
<th>20-story</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$J_{\text{drift}}$</td>
<td>$J_{w\text{accel}}$</td>
<td>$J_{\text{Moving}}$</td>
</tr>
<tr>
<td>TMD 5%</td>
<td>0.61</td>
<td>0.66</td>
<td>0.98</td>
</tr>
<tr>
<td>iPMI 5%</td>
<td>0.68</td>
<td>0.71</td>
<td>1.41</td>
</tr>
<tr>
<td>iPMI 90%</td>
<td>0.06</td>
<td>0.12</td>
<td>3.29</td>
</tr>
<tr>
<td>BI</td>
<td>0.06</td>
<td>0.09</td>
<td>3.29</td>
</tr>
</tbody>
</table>

In the evaluation of the control systems in the previous sections, RMS responses are used. The RMS measurement depends on duration and provides an order of magnitude of the intensity (amplitude) of the response. RMS amplitudes can be interpreted as effective amplitudes that can be scaled by a peak factor to obtain design responses. From this point of view, RMS responses can be used as surrogates to conduct the evaluation of design responses or as responses useful to evaluate serviceability criteria. However, because (i) design responses are usually based on the estimated peak (maximum value) of the response parameters of interest, and (ii) the variability in peak responses is different from the one observed in RMS responses, this section explicitly investigates the efficiency of the studied passive control systems in reducing the maximum seismic responses. The average of the maximum absolute acceleration and displacement responses computed based on Equation (7.13), for the TMD-like systems exposed to the 44 records are illustrated in Figures 7-18(a) and (b), respectively.

$$\overline{\text{MAX}(u_i)} = \frac{1}{44} \sum_{j=1}^{44} \text{abs}(\text{Max}(u_i))_j$$  \hspace{1cm} (7.13)

![Figure 7-18](image)  

Figure 7-18 $\overline{\text{MAX}}$ acceleration (m/s²) and displacement (m) responses of the 12-story building equipped with near-optimal TMD-like strategies subjected to the 44 ground motion records
To compare the efficiency of the control systems in reducing maximum responses with their efficiency in reducing RMS responses, the OFs introduced through Equations (7.8) to (7.11) are re-computed using the $\overline{MAX}$ responses and are illustrated in Table 7-5. An evaluation of the value of different OFs values shown in Tables 7-4 and 7-5 reveals that all passive control systems, especially the TMD-like systems, are less effective in reducing the peak responses than in reducing RMS responses. For example, consider the TMD system in the six-story structure. While for this case the value of $J_{\text{drift}}^{s}$ and $J_{\text{accel}}^{w}$ calculated based on RMS responses (shown in Table 7-4) are 0.61 and 0.66, respectively, these metrics calculated based on peak responses (shown in Table 7-5) are 0.76 and 0.81, respectively.

Table 7-5 Various OFs (based on maximum responses) for the near-optimal control strategies in the three test-bed buildings subjected to the 44 ground motion records

<table>
<thead>
<tr>
<th>Sys. Type</th>
<th>six-story</th>
<th>12-story</th>
<th>20-story</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$J_{\text{drift}}^{s}$</td>
<td>$J_{\text{accel}}^{w}$</td>
<td>$J_{\text{Moving}}^{\text{displ}}$</td>
</tr>
<tr>
<td>TMD 5%</td>
<td>0.76</td>
<td>0.81</td>
<td>1.25</td>
</tr>
<tr>
<td>iPMI 5%</td>
<td>0.82</td>
<td>0.86</td>
<td>1.74</td>
</tr>
<tr>
<td>iPMI 90%</td>
<td>0.08</td>
<td>0.11</td>
<td>2.87</td>
</tr>
<tr>
<td>BI</td>
<td>0.07</td>
<td>0.10</td>
<td>2.87</td>
</tr>
</tbody>
</table>

Some previous studies (e.g., Tributsch and Adam 2012) have shown that under certain conditions TMD could even amplify the maximum structural responses. Figure 7-20(a) illustrates the inter-story drift responses of the superstructure at different floor levels in the TMD system normalized to those of the uncontrolled systems for individual ground motion motion records. The same
results for the iPMI with a 5% IMR are shown in Figure 7-20(b). These figures illustrate a significant record-to-record variability in the responses of the optimized TMD system. As seen, while for some cases the normalized responses are as low as 0.55 (i.e., 45% response reduction), for some other records the control system has amplified inter-story drift responses (i.e., normalized values larger than 1.0). This drawback is more highlighted in the TMD system than in the PMI system. The same analysis is conducted for the BI-like systems but the results are not plotted herein for brevity. The simulation results show that for the iPMI with an IMR of 90% and the ideal BI systems the normalized maximum inter-story drift responses under different records range between [0.05-0.47] and between [0.06-0.48], respectively. These observations reveal that the BI-like systems consistently reduce maximum inter-story drift responses under all records because the period lengthening in these systems is sufficient to avoid the response spectral regions that exhibit relatively large spectral accelerations.

![Figure 7-20](image.png)

*Figure 7-20  Normalized maximum inter-story drift responses of the superstructure for the 12-story building under individual ground motion records: (a) near-optimal TMD; (b) near-optimal iPMI with a 5% IMR

7.11 Conclusions

In this chapter, the seismic performance of a partial mass isolation (PMI) system, in which different portions of masses at all stories are isolated from the superstructure, is studied. It is shown that this system could effectively integrate the benefits of conventional tuned mass damper (TMD) and base isolation (BI) techniques while resolving some of their deficiencies particularly in high-rise buildings (e.g., practical and architectural challenges associated with the heavy additional mass in a TMD, and problems due to the inherent flexibility, heavy loads and overturning moments imposed on isolator bearings in a base-isolated tall building). The PMI system is examined in linear
elastic shear-building models with six, 12, and 20 stories representing low-, mid- and high-rise buildings, respectively. Optimization is carried out on the PMI system’s parameters to minimize average normalized root-mean-square of inter-story drift responses of the structural frame under earthquake excitations with different frequency contents, while constraints are specified to control the isolated components (ICs) seismic responses. Two well-adopted passive control strategies (i.e., TMD and BI systems) are also studied as baseline configurations for assessing the effectiveness of the proposed PMI strategy.

Simulation results indicate that an iPMI system (i.e., a PMI with identical ICs at all stories) with a low isolated mass ratio (IMR), e.g., 5%, could perform as effectively as an equivalent TMD system, with the advantage of implementing inherent mass dampers without the weight restrictions of common TMDs. At a high IMR (e.g., 90%), the system could perform similarly to an ideal BI. It is illustrated that all three considered passive control strategies are effective under broad-band excitations in the test-bed buildings evaluated in this study. However, under narrow-band excitations, the control systems are effective only if the uncontrolled superstructure is stiffer than the soil profile. In all test-bed buildings, the most efficient passive control system is associated with the near-resonance case. A BI-like system for a structure located on a soft soil site (i.e., under a narrow-band excitation) is highly effective for the resonance situation. However, the performance of this system is very sensitive to the dynamic characteristics of the fixed-base superstructure as well as the soft soil. This observation implies that any misestimation of these influential parameters could cause significantly large structural responses, especially isolator drifts. Hence, the application of a BI system for a soft soil profile needs particular care, and is not recommended.

As it is observed, all three studied passive control strategies are associated with unique advantages and disadvantages. A TMD system with a 5% damper’s mass ratio seems to be effective for all test-bed buildings. However, when the number of stories increases, the mass required for a single TMD to achieve such a performance improvement increases significantly. For example, a 5% damper mass ratio at the roof floor of the 20-story building weighs as much as the roof itself, significantly affecting its structural and architectural design, while each IC in the proposed PMI system weighs as little as 5% of the weight of each story. ICs in an iPMI strategy with a low IMR, because of their role as sacrificial mass dampers, sustain relatively large seismic
responses (a similar behavior is observed for the damper’s mass in the TMD system). Hence, floors that contain components sensitive to large seismic responses, and components that are themselves sensitive to large seismic response are not appropriate candidates to be used as ICs in the PMI system. A BI technique can effectively reduce inter-story drift and absolute floor acceleration responses but the entire isolated superstructure may sustain excessive displacements relative to the base ground. In an iPMI configuration with a high IMR of 90%, while retaining the structural response improvements of the BI system, the structural frame remains almost stationary. However, in this iPMI configuration the problem of large isolators’ drift affects all stories.

Future research in this area should consider applying intermediate IMR values that can lead to a more effective and robust system. Configurations with dissimilar ICs along the building height, which have the potential to provide a better seismic performance, could be also studied. Applying ICs at all stories can significantly increase the construction cost of a building. Hence, optimizing the number and locations of ICs along the height should be investigated.

7.12 Appendix I. Design constraints for the passive control strategies

Parametric studies illustrate that in BI-like systems, with increasing either the damping ratio or fundamental period of the moving components (i.e., ICs in the PMI), the ANRMS inter-story drift values generally tend to decrease (see Figure 7-8c). In other words, $J_{\text{drift}}$ presents no discernible global optimum, and the optimization process may result in extremely large ICs’ damping and fundamental periods. For practical purposes, the equivalent viscous damping ratio of common isolator systems is rarely greater than 30% (e.g., see the characteristics of lead rubber bearings in the FIP INDUSTRIALE catalogs at http://www.fipindustriale.it/public/S03_LRB-eng.pdf). Hence, in this study, the maximum permissible damping ratio of the considered control systems is limited to 30%. A 2% lower bound is also specified on the damping ratio. On the other hand, significantly large ICs periods are associated with excessive drifts, which can be unacceptable because ICs may contain sensitive equipment or living areas. Furthermore, accommodating large ICs’ drifts requires wide seismic gaps and especial isolator bearings, which can result in significant architectural challenges.

An iPMI system with a BI-like behavior (i.e., $IMR = 90\%$) basically divides a building into two parts: a relatively stiff structural frame and flexible ICs. Hence, assuming a completely rigid
structural frame, isolated units can be oversimplified as SDOF systems, and the admissible period upper bound can be calculated by bounding the allowable ICs’ drift response to a given design period value. Because in this study K-T excitations are used as the earthquake excitations, the output ICs’ drift responses are normalized RMS quantities and cannot be directly linked to a design period value. In this appendix, attempts are made to estimate this period criterion based on an ASCE 7-16 (2016) design spectrum. It is acknowledged that the adopted strategy is associated with an inherent inconsistency (because of using a design spectrum that is different than the K-T excitations), and also the interaction between ICs and structural frame in tall buildings may invalidate the concept of an equivalent SDOF system for ICs. However, it can lead to a reasonable upper bound for the period of ICs. The adopted strategy is briefly described next:

The allowable design drift under the Design Basis Earthquake (DBE) is assumed to be 0.35 m (meaning that the allowable design drift under the Risk-targeted Maximum Considered Earthquake (MCE\textsubscript{R}) is approximately 0.50 m). The spectral drift of an IC represented by a SDOF system can be computed using the simple equation of \( S_d = S_a T^2 / 4\pi^2 \beta_L \) in which \( \beta_L = (\xi_{IC} / 0.05)^{0.30} \), and \( T \) is the target isolation period. An ASCE 7-16 (2016) design spectrum for a site of high seismicity (a region in San Francisco, soil type C: \( S_a = 0.563 g/T \)) is assumed. If the isolator’s drift demand under DBE is to be 0.35 m, the IC’s fundamental period and damping ratio should satisfy the inequality of \( T_{IC} \beta_L \leq 2.5 \). This approximate approach limits \( T_{IC} \) to 1.90 s and 4.28 s for the two extreme damping values of 2\% and 30\%, respectively. The same period limitation is applied to the BI system to control excessive drifts at the isolation interface.

7.13 Acknowledgments

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7.14 References


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Chapter 8

Partial Mass Isolation System for Seismic Vibration Control of Buildings


8.1 Abstract

In the previous chapter a partial mass isolation (PMI) system was studied that through isolating different portions of story masses can provide a building with multiple inherent vibration suppressors. It was shown that the PMI strategy with isolated mass ratios (IMRs) of 5% or 90% could perform as effectively as an equivalent tuned mass damper or an ideal base isolation system, respectively. In the present chapter, the PMI system is examined in structural models with different fundamental periods. PMI configurations in a wide IMR range of (5%:2.5%:90%) are optimized illustrating that applying an IMR of 25%-50% can provide an efficient system, simultaneously satisfying the constraints related to different performance objectives (i.e., mitigating the overall building seismic responses and controlling isolated components (ICs) responses while integrating these components into the building architecture). Simulation results reveal that using identical ICs at different stories, which has the advantage of facilitating the design and construction of the system, can lead to a near-optimal solution. It is also demonstrated that in terms of the spatial distribution of ICs, an adequate seismic performance improvement can be achieved by allocating ICs only at a subset of upper stories (e.g., top half stories), which can further simplify the PMI systems’ construction.

Keywords: Partial mass isolation technique; Vibration suppressor; Optimization; Modified Kanai-Tajimi filter; Parametric studies; Genetic algorithm.
8.2 Introduction

A partial seismic isolation technique, in which a local structural component, nonstructural system or floor slab is isolated from a primary building, has been utilized for various purposes. In a well-adopted application of this technique, denoted as the content protection system, acceleration sensitive nonstructural component and systems (e.g., delicate artworks, sensitive equipment, computer servers, etc.) are isolated from the main floor using a secondary raised floor system (Iwan 1978; Hamidi and El Naggar 2007; Liu and Warn 2012; Lu et al. 2013; Jia et al. 2014). This technique aims to reduce acceleration demands on the target components enhancing their seismic performance. A partial isolation technique can be also used to mitigate structural responses. For example, Pourmohammad et al. (2006) studied a partial isolation system in a 10-story shear building in which 95% of the main floor slabs at all stories were isolated from the primary structural frame. They performed a parametric study, assuming identical characteristics for the isolated floors, and selected a few isolation bearings’ stiffness and damping coefficients to optimize the system under a single historical earthquake. Pourmohammad et al. concluded that such a floor isolation technique is effective in reducing the acceleration responses of the isolated floors as well as the main structural frame inter-story drift responses.

A partial isolation technique utilizing tuned mass damper (TMD)-related principles has been considered to mitigate both local and global seismic demands experienced by a building. This technique through isolating different structural systems, components, floors or stories can provide a building with heavy inherent TMDs without adding any unnecessary extra weight. Since a TMD’s effectiveness is highly dependent on its weight (De Angelis et al. 2012), such a strategy can lead to a superior control system. A large TMD has the additional advantage of providing a robust control system against possible changes in the dynamic characteristics of the primary structure (Hoang et al. 2008). A “mega-sub-controlled” building approach, proposed by Feng and Chai (1997), is one of the partial isolation configurations that can fulfill this twofold performance. This configuration consists of a mega structure as the main structural frame and several isolated multistory substructures. This technique can convert a flexural tall building, where a typical base isolation (BI) is not suitable, into several shear sub-buildings that can benefit from the seismic isolation technology. Meanwhile, isolated substructures can function as multiple TMDs for the entire building without imposing any burdensome extra weight.

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Another partial isolation approach based on TMD-related principles is the upper-story isolation technique. This configuration separates one (i.e., roof story) or several upper stories from the rest of a building to provide a heavy mass damper for the non-isolated part of the structure below the isolation interface (Ziyaeifar and Noguchi 1998; Villaverde and Mosqueda 1999; Villaverde 2002; Tian et al. 2008; Matta and De Stefano 2009; Tsuneki et al. 2009; Chey et al. 2010, 2013). This approach can also lead to a reduction in inter-story drift and acceleration responses of the isolated multi-story part above the isolation interface. Alternatively, a multi-floor isolation system is another version of the partial isolation system in which main floor slabs are isolated from the primary structure to provide a building with multiple inherent TMDs. In 2014, Xiang and Nishitani designed a multi-floor isolation system for a 6-story shear building model using a gradient-based optimization method and selecting the steady-space inter-story drift responses as the performance indices to be minimized. They showed that the absolute acceleration responses of the isolated floors were reduced with respect to the corresponding floors in the uncontrolled structure. For the optimization process, Xiang and Nishitani assumed isolated floors with identical fundamental periods and damping ratios and a fixed isolated mass ratio (the ratio of an isolated floor mass to the overall story mass) of one-third at each floor level. In another study, Xiang and Nishitani (2015) optimized two configurations of a multiple floor isolation system in a 25-story flexural-shear type building utilizing a non-dominated sorting Genetic Algorithm. In the first configuration, all stories were equipped with isolated floors, while in the second case isolated floors were designated only at the top 10 stories. In 2015, Sakr examined this version of partial isolation technique with identical isolated floors in five-, 25-, and 50-story shear buildings. Sakr conducted a parametric study to optimize a few configurations for each example building under harmonic loads as well as three historical records. The conducted simulations illustrated that while the proposed strategy could benefit the five and 25-story buildings, it was not effective in the 50-story structure. Sakr attributed the deficiency of the floor isolation technique in the 50-story building to the selected parameters for isolated floors, and proposed further investigation for high-rise buildings.

The authors recently studied a partial mass isolation (PMI) system in which different portions of the story masses were isolated from the main structure to provide a building with inherent dynamic vibration suppressors (Anajafi and Medina 2018). Two PMI configurations with isolated mass ratios (IMRs) of 5% and 90% were optimized in three structural models under Kanai-Tajimi
filtered white noise excitations with different frequency contents representing various soil profiles. The authors demonstrated that a PMI system with identical isolated components (ICs) at all stories and a 5% or 90% IMR could exhibit dynamic responses that are similar to those exhibited by an equivalent TMD or an ideal BI system, respectively. Meanwhile, the proposed PMI system could mitigate some deficiencies associated with the application of TMD and BI systems, especially in high-rise buildings. This system converts existing parts of a building to inherent dampers, and hence, resolves the problems associated with the large secondary mass generally required in a common TMD system. While heavy overturning moments and large axial loads inserted on isolator bearings in base-isolated mid- and high-rise buildings can cause significant problems, a PMI system with ICs at several stories does not suffer these deficiencies. Furthermore, the proposed PMI technique incorporates multiple inherent dampers, which provides a more redundant system, and if a damper fails, many alternative locations exist to provide the required energy dissipation. However, this study showed that PMI configurations with IMRs of 5% and 90%, although effective in reducing the primary building seismic responses, could suffer significant drawbacks. For example, ICs in the PMI system at a 5% IMR, could sustain relatively large drift (stroke) and acceleration responses because they operate as sacrificial TMDs.

A partial solution for the undesirable effects on ICs is to increase the ICs damping ratios with respect to the optimal characteristics. A TMD’s damping coefficient larger than the optimal one can reduce the damper responses, whereas the response reduction of the main structure remains almost unaffected (Tributsch and Adam 2012). However, the TMD’s stroke in high-rise buildings with a long fundamental period may be still significant. On the other hand, providing a large mass ratio of 90% could be impractical due to a variety of structural and architectural constraints. Therefore, this study evaluates intermediate IMR values that can result in (i) a more effective system with respect to the PMI system with a low IMR of 5%, and (ii) a system that exhibits a better integration with the building architecture than a configuration with a high IMR of 90%. In the previously studied PMI system, ICs were installed at all stories. This can significantly increase construction costs. In the present study, partial PMI configurations with ICs installed at a limited number of stories are investigated as well.

In the present work, the PMI system is examined in linear elastic shear-building models with six, 12, 20, and 40 stories representing a wide range of structural periods. Different seismic
performance objectives, and also structural and architectural constraints are introduced and evaluated. A wide IMR range with many discretized values (i.e., 5\%: 2.5\%: 90\%) is studied, and the possibility of the existence of an optimal IMR is evaluated. Configurations involving ICs with different properties along the height, which have the potential to lead to a better seismic performance, are optimized. To simplify the design and construction of the PMI system, partial configurations with ICs only at a subset of stories (e.g., top half stories) are also studied. The optimization approach is based on parametric studies and the implementation of a Genetic Algorithm. The parameters of the PMI system are optimized with the objective of minimizing the average normalized root mean square (ANRMS) of the inter-story drift responses of the structural frame under a filtered Gaussian white noise random excitation. In this process, appropriate constraints are applied to control the seismic demands imposed on the ICs.

### 8.3 Model formulation

Numerical simulations are conducted using two-dimensional linear-elastic shear-type models (i.e., story masses are lumped at floor levels, and one lateral degree of freedom is specified to each mass). It is assumed that ICs are symmetrically distributed in the buildings’ plans and torsional effects are neglected. To examine the efficiency of the proposed PMI system in a wide range of structural frequencies, structural models with six, 12, 20, and 40 stories are selected. The story mass and stiffness coefficient are constant along the height of each building (i.e., $m_1^s = m_2^s = \ldots m_n^s$ and $k_1^s = k_2^s = \ldots k_n^s$). It is recognized that using shear-type models and assuming uniform characteristics along the height (particularly story stiffness) is not realistic, especially for high-rise structures. However, the purpose of this study is to investigate the effectiveness of the proposed PMI system using simplified models. Given that a large number of optimizations are performed and many different configurations are studied, using such simplified models that can accelerate the analyses seems to be justifiable.

The most salient dynamic characteristics of the test-bed buildings are presented in Table 8-1. The stiffness proportional damping, assuming 2\% of the critical damping at the fundamental mode, is applied to approximate the superstructure (i.e., the entire building in the uncontrolled structure as well as the non-isolated part in the controlled systems) viscous damping. The 2\%-damping assumption implies that the superstructure is mildly damped. Isolation layers are modeled with
linear Kelvin-Voigt elements (i.e., linear stiffness and equivalent viscous damping). Two passive control strategies (i.e., conventional TMD and BI systems) are also presented as appropriate baselines for assessing the seismic performance of the proposed passive PMI strategy.

![Schematic models of the uncontrolled and passive PMI systems in an n-story linear-elastic shear building](image)

Figure 8-1  Schematic models of the uncontrolled and passive PMI systems in an n-story linear-elastic shear building

<table>
<thead>
<tr>
<th>Building</th>
<th>Stiffness Coeff. (10^7 N/m)</th>
<th>Story Mass (10^3 Kg)</th>
<th>Fundamental Period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-story</td>
<td>3.77</td>
<td>20.00</td>
<td>0.60</td>
</tr>
<tr>
<td>12-story</td>
<td>3.48</td>
<td>20.00</td>
<td>1.20</td>
</tr>
<tr>
<td>20-story</td>
<td>3.36</td>
<td>20.00</td>
<td>2.00</td>
</tr>
<tr>
<td>40-story</td>
<td>3.28</td>
<td>20.00</td>
<td>4.00</td>
</tr>
</tbody>
</table>

Table 8-1  Dynamic characteristics of the uncontrolled buildings

The equations of motion for an n-story linear-elastic shear-type building equipped with a PMI system (shown in Figure 8-1) can be expressed as

\[
\mathbf{M} \ddot{\mathbf{x}}(t) + \mathbf{C} \dot{\mathbf{x}}(t) + \mathbf{K} \mathbf{x}(t) = -\mathbf{M} \ddot{\mathbf{z}}_g(t) + \mathbf{f}
\]  

(8.1)

where
\[
K' = \begin{bmatrix}
  k_1^s + k_2^s & -k_2^s & 0 & \cdots & 0 \\
  -k_2^s & k_2^s + k_3^s & -k_3^s & \cdots & 0 \\
  \vdots & \cdots & \ddots & \cdots & \vdots \\
  0 & \cdots & \cdots & k_{n-1}^s + k_n^s & -k_n^s \\
  0 & \cdots & \cdots & -k_n^s & k_n^s \\
\end{bmatrix}_{n \times n}
\]

\[
M = \begin{bmatrix}
  M_{\text{non}} & 0 \\
  0 & M_{\text{IC}}
\end{bmatrix}_{2n \times 2n},
K = \begin{bmatrix}
  K^s + K_{\text{IC}} & K_{\text{IC}} \\
  K_{\text{IC}} & K_{\text{IC}}
\end{bmatrix}_{2n \times 2n},
C = \begin{bmatrix}
  C^s + C_{\text{IC}} & C_{\text{IC}} \\
  C_{\text{IC}} & C_{\text{IC}}
\end{bmatrix}_{2n \times 2n}
\]

\[
M_{\text{non}} = \text{diag}(m_1^{\text{non}}, m_2^{\text{non}}, \ldots, m_n^{\text{non}}),
M_{\text{IC}} = \text{diag}(m_1^{\text{IC}}, m_2^{\text{IC}}, \ldots, m_n^{\text{IC}}) \quad \text{and} \quad m_i^{\text{non}} + m_i^{\text{IC}} = m_i^s,
\]

\[
K_{\text{IC}} = \text{diag}(k_1^{\text{IC}}, k_2^{\text{IC}}, \ldots, k_n^{\text{IC}}),
C_{\text{IC}} = \text{diag}(c_1^{\text{IC}}, c_2^{\text{IC}}, \ldots, c_n^{\text{IC}}) \quad \text{and} \quad c_i^{\text{IC}} = 2 \sqrt{k_i^{\text{IC}} m_i^s}
\]

\[
M, C, \text{ and } K \text{ are the global mass, damping, and stiffness matrices of the PMI system, respectively.}
\]

\[C^s \text{ is the damping matrix of the superstructure (i.e., the non-isolated part).}\]

\[x = [x_1^s, x_2^s, \ldots, x_n^s, x_1^{\text{IC}}, x_2^{\text{IC}}, \ldots, x_n^{\text{IC}}]^T \text{ is the displacement vector relative to the ground, and } \ddot{x}_g(t) \text{ is the ground acceleration.}\]

\[x_i^s \text{ and } x_i^{\text{IC}} \text{ are the displacements of the superstructure and of the ICs relative to the ground at the } i\text{-th story, respectively.}\]

\[f \text{ is the external force vector of the system (e.g., wind), and } \mathbf{1} \text{ is a column vector of ones. } m_i^{\text{non}} \text{ and } m_i^{\text{IC}} \text{ are the masses of the non-isolated part and of the isolated component (IC) at the } i\text{-th story, respectively.}\]

\[k_i^s \text{ and } k_i^{\text{IC}} \text{ are the stiffness coefficients of the structural frame and of the ICs at the } i\text{-th story, respectively.}\]

Equation (8.1) can be represented in the following state space form:

\[
\begin{bmatrix}
  \dot{x} \\
  \ddot{x}
\end{bmatrix}_{4n \times 1} = \begin{bmatrix}
  0_{2n \times 2n} & \mathbf{1}_{2n \times 2n} \\
  -M^{-1}K & -M^{-1}C
\end{bmatrix}_{4n \times 4n} \begin{bmatrix}
  x \\
  \dot{x}
\end{bmatrix}_{4n \times 1} + \begin{bmatrix}
  0_{2n \times 1} & 0_{2n \times 2n} \\
  \mathbf{1}_{2n \times 1} & M^{-1}
\end{bmatrix}_{2n \times (2n+1)} \begin{bmatrix}
  \ddot{x}_g \\
  \mathbf{f}
\end{bmatrix}_{(2n+1) \times 1}
\]

or \[\dot{z} = Az + Bu,\] where \(z\) is the state vector \([x \ \dot{x}]^T\); \(A\) and \(B\) are the coefficient matrices, and \(u\) is the system input vector. Since \(f\) is assumed to be zero for earthquake loads, \(u\) depends only on the ground acceleration.
8.4 Ground excitation

The earthquake-induced ground acceleration is modeled as a modified filtered Gaussian white noise random process corresponding to the Kanai-Tajimi (K-T) spectrum (Soong and Grigoriu 1993). This model has been widely used in the literature for studying control systems (e.g., Ramallo et al. 2002; Schmelzer et al. 2010; De Angelis et al. 2012). The transfer function representation of the modified K-T filter in the Laplace domain is

\[ F_{K_T}^{\text{Modified}}(s) = \frac{2g}{s^2 + 2g} \left( \frac{s^2}{s^2 + 2c} + \frac{1}{s} \right) \]

(8.3)

In Equation (8.3), the first term is the transfer function of the well-known K-T filter, and the second term is a low-cut filter added to modify the spectral shape of the filter at low frequencies (Clough and Penzien 1993). The filter parameters expressed in Equation (8.3), which control the frequency content of the stochastic excitation, are selected as \( g = 15 \text{(rad/s)} \), \( g = 0.60 \) and \( c = 1.5 \text{(rad/s)} \), \( c = 0.60 \) to reflect a firm-soil condition (Der Kiureghian et al. 1991). The modified filter can be incorporated into the state space formulation presented in Equation (8.2). In this approach, the state variables can be obtained by solving a Lyapunov equation (for detailed descriptions of the Lyapunov equation, readers are referred to Lutes and Sarkani (1997)).

8.5 Performance objectives and design limitations

Generally, the primary aim in structural seismic design is to limit inter-story drift and floor acceleration responses below predefined targets under specified design loads. The reason is that excessive inter-story drifts are correlated with seismic damage to nonstructural (e.g., partition walls) and structural components, and excessive floor accelerations cause damage to nonstructural components (e.g., suspended ceilings), equipment (e.g., HVAC systems), and building contents (e.g., computer servers). In this study, the primary objective is to minimize the root mean square inter-story drift responses. As shown in the results section of this paper (Figure 8-3b), fulfilling this primary objective has the added benefit of improving the global acceleration responses of the PMI system. The optimization procedure adopted in this study is described in the following paragraphs.
For a given passive control system, exposed to an assumed K-T process, an optimal solution of the system parameters (e.g., stiffness and damping ratio of ICs) can be derived by minimizing the average normalized root mean square (ANRMS) of inter-story drift responses as the primary objective function (OF):

\[
J_{\text{drift}} = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(x_{is}^s - x_{is-1}^s)}{\text{RMS}(x_{is}^s - x_{is-1}^s)}
\]

(8.4)

where \( x_{is}^s \) is the superstructure (i.e., non-isolated part in the PMI system) displacement at the \( i \)-th floor level; \( x_{is}'s \) are relative displacements with respect to the ground (i.e., \( x_0^s = 0 \)). The subscripts \( u \) and \( c \) stand for the uncontrolled and controlled configurations, respectively. The overall acceleration responses of the optimized systems can be evaluated using a weighted OF, \( J_{\text{accel}}^w \):

\[
J_{\text{accel}}^w = \begin{cases} 
\frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(\ddot{x}_i^s) \cdot (1 - \text{IMR}_i) + \text{RMS}(\ddot{x}_i^{\text{Moving}}) \cdot \text{IMR}_i}{\text{RMS}(\ddot{x}_i^s)} & \text{For PMI & BI} \\
\frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(\ddot{x}_i^s) + \text{RMS}(\ddot{x}_i^{\text{Moving}}) \cdot \mu_i}{\text{RMS}(\ddot{x}_i^s)(1 + \mu_i)} & \text{For TMD} 
\end{cases}
\]

(5a)

where \( \ddot{x}_i^s \) and \( \ddot{x}_i^{\text{Moving}} \) are absolute acceleration responses of the superstructure and of the moving components (i.e., damper’s mass in TMD, ICs in PMI, and the entire floor in BI system) at the \( i \)-th story, respectively. \( \text{IMR}_i = m_{IC}^s / m_i^s \) is the isolated component’s mass ratio at the \( i \)-th story. The vector \( \{0, m_{TMD}^s / \sum_{i=1}^{n} m_i^s \} \) defines the mass ratio of the TMD system. The term “w” in \( J_{\text{accel}}^w \) implies a weighted OF. According to Equation (8.5a), \( J_{\text{accel}}^w \) combines the non-isolated part and ICs acceleration responses in the PMI system, and renders a single OF which is analogous to ones used for the respective TMD or BI systems. Equation (8.5a) can be directly implemented for a BI system equating all \( \text{IMR}_s \) to 1.0 while, for consistency purposes, this equation should be modified as Equation (8.5b) for a TMD system.

In conventional BI and TMD systems, optimizing the control system parameters minimizing the inter-story drift responses as the primary OF, can lead to significantly large seismic responses of the isolator bearings and of the damper’s mass, respectively. A similar behavior can cause significant problems in a PMI system given that ICs may contain sensitive equipment/contents. Hence, in order to assess displacement and acceleration responses of the moving components in
the optimized systems, two additional OFs, denoted as $J_{\text{disp.}}^\text{Moving}$ and $J_{\text{accel.}}^\text{Moving}$, are defined as Equations (8.6) and (8.7). These equations provide a quantitative evaluation of the magnitude of the seismic responses of the moving components in the controlled systems (i.e., ICs in the PMI system) with respect to those of their corresponding part in the uncontrolled systems.

$$J_{\text{disp.}}^\text{Moving} = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(x_i^{\text{Moving}})}{\text{RMS}(x_i^s)}$$

$$J_{\text{accel.}}^\text{Moving} = \frac{1}{n} \sum_{i=1}^{n} \frac{\text{RMS}(\ddot{x}_i^{\text{Moving}})}{\text{RMS}(\ddot{x}_i^s)}$$

Preliminary simulation results illustrate that, consistent with structural dynamics principles, if constrains are not applied on the characteristics of the PMI system with a high IMR (e.g., 90%), the optimization process leads to isolators with unreasonably long periods and high damping ratios. To obtain reasonable and practical design characteristics, two constraints are applied herein. First, the ICs damping ratio is bounded to [0.02-0.30] of the critical damping, which is deemed to be reasonable for civil engineering structures. Secondly, to control the fundamental period of the ICs, based on engineering judgment, the equivalent elastic period of the ICs (i.e., $T_{\text{equiv.}}^\text{IC} = T_{\text{IC}}^b L$) in which $L = (T_{\text{IC}}^b / 0.05)^{0.30}$, is limited to 2.50 s. This constraint is derived from an approximate method described in (Anajafi and Medina 2018). Adopting this approach implies that, for example, for the two extreme damping values of 0.02 and 0.30, the maximum elastic period of ICs, $T_{\text{Max}}^\text{IC}$, is limited to 1.90 and 4.28 s, respectively. The same constraints are specified for BI systems.

8.6 PMI system’s configurations and optimization methods

In a generic form of a PMI system with arbitrary ICs at each story, a total of $3n$ parameters should be optimized (i.e., stiffness, damping ratio, and mass ratio of ICs at each story), where $n$ is the number of stories. However, a PMI system with parameters that vary with height, especially IMRs, may not be ideal for the design and construction of a building. Hence, in this study, for all PMI configurations, ICs are assigned the same mass ratio at each story. Overall two configurations of the PMI system are investigated. In the first configuration, identical ICs are assumed at each story. This configuration is denoted as an identical PMI (iPMI) system. In this case, since the IMR is fixed at each story, only two parameters (i.e., stiffness and damping ratio of ICs) should be
optimized. This configuration is meant to facilitate the design and construction of the building. For this kind of PMI system, the optimization process is conducted first for different IMRs assuming that all stories are equipped with identical ICs. Then in a subsequent section of this chapter, partial PMI systems with identical ICs only installed at a subset of stories (e.g., upper half stories) are optimized as well. In the second configuration, ICs’ stiffness and damping ratios may change at different stories. Therefore, for this case, once an IMR value is assumed, a total of $2n$ parameters (two per story) should be optimized. This configuration can lead to a better seismic performance at the expense of increased design and construction difficulties.

The optimization approach implemented in this paper for the TMD, BI, and PMI systems is based on either parametric studies or a Genetic Algorithm (GA) method, when appropriate. Enumerative techniques, such as a conventional parametric study, calculate OFs for all possible combinations of unknown parameters (e.g., stiffness and damping coefficients of ICs) in the search area. Hence, for optimization problems with many variables, such approaches are computationally expensive. For such problems, stochastic and multi-dimensional methods (e.g., a GA) that intelligently search the solution space can be computationally efficient. When optimizing the first PMI configuration (i.e., PMI with identical ICs at each story), this study aims to optimize the PMI system for 35 different IMRs between 0.05 and 0.90 at increments of 0.025. In this case, considering the limited number of unknown parameters at each IMR (i.e., one stiffness term and one damping coefficient) and the desired outputs, it is more convenient to perform a parametric study. However, for the second configuration (i.e., ICs’ with possibly dissimilar stiffness and damping coefficients at different stories), due to the large number of unknown variables (i.e., $2n$ for each IMR), a GA is employed.

### 8.7 Identical partial mass isolation system (iPMI)

For an iPMI system with a given IMR, the primary OF (i.e., $J_{drift}$ defined by Equation 8.4) is calculated over a wide range of system parameters. The optimal solution is the one that provides the minimum value of $J_{drift}$ obtained for configurations with parameter values within predefined ranges (e.g., $T_{Equiv.}^{IC} = 2.50$ s and $IC = 0.30$ for the iPMI system).
As an example, Figures 8-2(a) to (d) demonstrate the variation of $J_{\text{drift}}$ versus the control systems’ parameters (i.e., the IC’s period normalized to the uncontrolled building period, $T_{IC}/T_{UNC}$, and IC’s damping ratio) for the six-story building assuming three different IMRs. As seen, the PMI configuration with a low IMR is more sensitive to the change in ICs’ parameters than the configurations with larger IMRs meaning that the larger IMRs can provide a more robust system.

Figure 8-2 Variation of $J_{\text{drift}}$ versus iPMI system’s parameters for the six-story building assuming three different IMRs

The same procedure as the one used to obtain the results shown in Figure 8-2, is conducted for other IMRs of interest. Figure 8-3 illustrates the variation of $J_{\text{drift}}$ with the IMR for the four example buildings. In this figure optimization results for the TMD and BI systems, which are used herein as baseline configurations, are depicted as well. It is worthwhile to note that the mass of the baseline TMD is equal to the overall isolated mass of the iPMI system at the lowest IMR considered (i.e., 5%).
Figure 8-3 Variation of different OFs with respect to the IMR for the optimized systems

Figure 8-3(a) illustrates how $J_{\text{drift}}$ changes with respect to the IMR for different example buildings. Each point in this figure corresponds to the minimum ANRMS of inter-story drift responses at a specific IMR. As demonstrated, for all example buildings, as IMR increases, $J_{\text{drift}}$ decreases asymptotically implying that the efficiency of isolating story masses decreases as IMR increases. As seen, the steepest rate of reduction occurs at low IMRs. This asymptotic behavior is more pronounced in the 40-story building, which is more affected by the constraints imposed on the damping ratio and period of the ICs.

Figure 8-3(b) depicts the ANRMS of weighted acceleration responses, $J_{\text{accel}}$, of the optimized configurations. As depicted, with increasing the value of IMR, unlike the observed trend in $J_{\text{drift}}$, the value of $J_{\text{accel}}$ continuously decreases. Figure 8-3(b) also confirms that minimizing $J_{\text{drift}}$ in the studied iPMI configurations has the additional benefit of significantly decreasing $J_{\text{accel}}$. According to Figures 8-3(a) and (b), the efficiency of the studied passive control strategies in taller structures
(e.g., the 40-story example) generally decreases with respect to shorter structures (e.g., the six-story example). In terms of the weighted acceleration OF, this behavior is observed because the high-rise uncontrolled buildings already possess a long period, and hence, they are less affected by the selected K-T excitation (i.e. a K-T filter with a firm-soil characteristics). In terms of the inter-story drift OF, the significant flexibility of the superstructure renders the control systems less effective in high-rise buildings.

Figures 8-3(a) and (b) illustrate that an iPMI system ranks between a common equivalent TMD and an ideal BI system in terms of its efficiency to mitigate inter-story drift and floor acceleration seismic demands. On average the iPMI system at an IMR of 5% behaves similarly to its equivalent TMD while the TMD tends to be consistently more efficient than the iPMI. For instance, in terms of the weighted acceleration response reduction, the TMD outperforms the iPMI by 11.7%, 8.7%, 8.5%, and 8.2% (i.e., $J^{TMD}/J^{iPMI}$) in the six-, 12-, 20-, and 40-story buildings, respectively. In fact, since the entire damper’s mass in a TMD system is placed at the roof level, it is usually more effective than an iPMI system with the same overall damper mass uniformly distributed throughout the building height. However, when the number of stories increases, the mass required for a single TMD to achieve such an improvement increases significantly. For example, a 5% damper mass ratio at the roof level of the 20-story building weighs as much as the roof itself, significantly affecting the structural and architectural design of that floor while each IC weighs as little as 5% of each story. As a disadvantage of the iPMI, this system affects the design of all stories, especially due to the relatively large displacement and acceleration responses of ICs, whereas a single TMD affects only one floor. A similar comparison can be made for the BI-like systems (i.e., the ideal BI and the iPMI at a large IMR, e.g., 90%).

Figures 8-3(c) and (d) depict the ANRMS of displacement and acceleration responses of the moving components for different control strategies. As shown, the adopted optimization procedure causes significant drawbacks in the seismic responses of moving components (i.e., ICs in the PMI system) in the TMD-like systems. In other words, at the low-IMR region (e.g., IMR <10%), ARMS displacement and acceleration responses of ICs are significantly larger than those of the corresponding components in the uncontrolled buildings (i.e., $J > 1.0$). The amplified acceleration responses can damage isolated units’ contents. On the other hand, the large induced IC displacements require large seismic gaps and special bearings, which are associated with
significant architectural and construction challenges. Consistent with the fundamentals of structural dynamics, a partial solution to decrease the ICs responses in iPMI configurations with low IMRs is to increase the ICs damping ratio with respect to the optimal value. As shown in Figure 8-4(a), $J^s_{drift}$ is not very sensitive to the change in the ICs damping ratio. However, according to Figure 4b, a small change in ICs damping ratio can significantly decrease the magnitude of $J^{Moving}_{displ}$. Hence, increasing the ICs damping ratio can reduce the ICs’ seismic responses without negatively influencing the global seismic performance of the system.

Figure 8-4 Variation of OFs versus ICs’ parameters in the six-story building: (a) $J^s_{drift}$ (b) $J^{Moving}_{displ}$.

(note that $T^{IC}/T^{UNC}=1.05$ and $\xi^{IC}=0.11$ corresponds to the optimal solution)

For example, the optimal damping ratio for the iPMI configuration with a 0.05 IMR in the six- and 20-story buildings is 0.11 and 0.12, respectively. If the ICs damping ratio in these configurations is increased to 0.25, the OF monitoring ICs displacement responses decreases from 1.98 to 1.37 for the six-story and from 2.16 to 1.49 for the 20-story structure (see Figure 8-5). However, the primary OFs remain almost unaffected. A similar reduction is observed in the ICs displacement responses of the modified iPMI configurations. It should be noted that even the modified ICs responses can be still large and cause problems in the design and operation of the iPMI system. For example, the modified $J^{Moving}_{displ}$ of 1.49 in the 20-story structure means that average RMS displacement response of ICs is 1.49 as large as that of the corresponding parts in the uncontrolled system while, because of the long fundamental period, displacement responses of the uncontrolled system are already large.
Comparing various OFs of the optimally designed and modified iPMI configurations (IMR = 0.05) in the six- and 20-story buildings

8.8 Efficient IMR range for the iPMI system

Determining the optimal IMR for the iPMI system requires a comprehensive cost-benefit analysis that accounts for different weighting factors for the considered OFs; architectural and structural constraints; the cost of isolating different portions of story masses; losses after earthquake events; etc. In this section, a simplified procedure is implemented to find an efficient IMR range. The decision criteria to estimate such an IMR range are considered to be: (i) reduction of the value of the primary OF; (ii) efficiency with respect to the IMR value (i.e., structural benefits versus the cost of adding such a new technology to a building); (iii) minimization of undesirable deformation and acceleration demands on ICs; (iv) compliance with general architectural and structural constraints. The implementation of the aforementioned decision criteria results in designs that exhibit an intermediate IMR range, as described in the section below.

(i) Reduction of the value of the primary OF; the implementation of this criterion results in the elimination of low values of IMR. In other words, the improvement in $J_{\text{drift}}$ and $J_{\text{accel}}$ is not significant in the low IMR region (e.g., less than 10%) as compared to high and intermediate values of IMRs.

(ii) Efficiency; the implementation of this criterion results in the rejection of high IMR values. According to Figure 8-3(a), the minimum value of $J_{\text{drift}}$ is obtained at the largest possible IMR. However, the largest IMR is not necessarily the optimal solution for the iPMI system. In fact, the terms “minimum” and “efficient” should be carefully distinguished. For example, consider the 40-story building. In this case, isolating the first 25% of the story mass can reduce ARMS inter-story
drift by 65% (i.e., \( J_{\text{drift}} = 0.65 \)) while isolating the additional 65% (i.e., using an IMR of 90%) further reduces this metric by only 13%. Hence, applying an intermediate IMR seems to provide a more efficient iPMI system, even though the minimum \( J_{\text{drift}} \) is achieved at a 90% IMR.

(iii) Minimization of demands on ICs; the implementation of this criterion results in the elimination of both low and high IMRs. As seen in Figures 8-3(c) and (d), \( J_{\text{displ.}}^{\text{Moving}} \) and \( J_{\text{accel.}}^{\text{Moving}} \) at a low IMR are significantly large in all example buildings. At a high IMR, isolated components in the six- and 20-story buildings experience larger displacement responses than the corresponding components in the uncontrolled systems.

(iv) Architectural and structural constraints; the implementation of this criterion can reject both low and high IMRs because of the configuration and arrangement of different spaces in a building. Isolating a small portion of a story mass (e.g., 5%) may be even more difficult than isolating a bigger portion (e.g., 25%). For example, in the schematic design shown in Figure 8-6, isolating two-thirds of the story mass may allow for a more flexible integration with the building’s architectural layout than isolating a much smaller portion of the floor mass. Furthermore, applying the PMI system as a new technique itself imposes a significant initial cost on a building, regardless of whether the IMR is low or high. In other words, the cost of the first 5% IMR may be several times more expensive than that of the next 5%. Hence, isolating a larger mass, which further improves structural seismic performance, can justify the implementation of such a new system. On the other hand, achieving a very high IMR (e.g., 90%) may be structurally and architecturally prohibitive. In such a configuration, a large portion of each story mass should be isolated and allowed to experience a relatively large horizontal movement. Furthermore, isolating some heavy components and partitions (e.g., exterior walls, staircases, locations specified for mechanical and electrical equipment) may be very difficult. Hence, using an intermediate IMR seems to be more practical.
As discussed, prescribing a constant optimal IMR value for all building types is a challenging task. However, an intermediate range of IMRs (e.g., 25% and 50%) seems to be an appropriate selection. In this range, an adequate balance between reducing seismic demands on the ICs and the entire building can be obtained. Intermediate values of IMRs result in designs that comply with appropriate tradeoffs between various performance objectives, especially when taking into account practical and architectural limitations. As demonstrated in Figure 8-7, although an intermediate IMR is not a superior configuration in terms of any OF, it is not the worst scenario with respect to any OF either. In this range, considerable reductions in inter-story drift and acceleration responses of the overall structure are achieved without unreasonable increases in the ICs’ seismic responses.
8.9 Optimization of the PMI system using a Genetic Algorithm

In the previous sections, the PMI configurations were assigned identical stiffness and damping coefficients as well as an equal IMR across all stories to simplify the system design and construction. In this section, a generic configuration of the PMI system (gPMI), in which ICs’ stiffness and damping terms can vary from story to story, is optimized. Such a generic configuration results in $2n$ unknown parameters (i.e., 2 per story) for an $n$-story building. Considering the large number of unknown parameters, the optimization process is conducted using MATLAB’s Genetic Algorithm (GA) toolbox. The ultimate aim of this section is to evaluate the seismic performance of the relatively simple iPMI configuration with respect to that of the more general gPMI configuration. The gPMI system with two intermediate IMRs of 25% and 50% is presented in detail while a similar overall trend was observed for other IMRs. The GA procedure is briefly discussed next.

Evolutionary optimization algorithms, such as GAs, can handle optimization problems with many variables without the extensive computational demands associated with an enumerative technique such as a conventional parametric study. Since these approaches utilize information from many search points simultaneously, there is less chance to be trapped in any local optimal point. The GA, firstly introduced by Holland (1975) and then implemented in structural engineering by Goldberg (1989), solves optimization problems using a technique inspired by the principles of biological evolution. The process starts with a set of candidate optimal solutions, called an initial population. Each individual in the population is named a chromosome, which represents a possible solution to the problem. Over successive iterations (generations) the population evolves toward an optimal solution. At each iteration, through a fitness-based process, a fraction of the present population is stochastically selected into the mutation pool to form a new generation. The fitness is generally the value of an objective function that is to be optimized; the fitness-based process refers to the principle of survival of the fittest, meaning that the fitter individuals are typically allotted a higher chance of passing their genes into the subsequent generations. Preserving the less-fit solutions is also crucial to guarantee the genetic diversity of later generations. In order to produce a new solution (chromosome), a pair of parent solutions is selected from the mutation pool. Then individual genomes are recombined and randomly mutated to generate a child solution, which
generally inherits many of its parent’s features. Since at each step superior organisms are selected for breeding, typically the average fitness of consecutive populations gradually improves as the algorithm proceeds. The algorithm terminates if either of the following conditions is fulfilled: a user-specified number of generations has been created, a specified time has elapsed, or there is no improvement in the objective function for a sequence of consecutive iterations. In the subsequent paragraphs the GA-optimized configurations are discussed.

In the GA optimization process, for a better convergence, the near-optimal IPMI system’s parameters obtained in the previous sections are selected as initial populations, and the simulations are repeated multiple times to ensure the stability of the solutions. First, gPMI system’s parameters (i.e., \( k_{gPMI} \) and \( \zeta_{gPMI} \)) are compared to those of the IPMI system (i.e., \( k_{IPMI} \) and \( \zeta_{IPMI} \)) in Figure 8-8 for the six- and 20-story buildings for an IMR of 25%. Though the 12- and 40-story buildings were also optimized, such graphs are not presented for these example buildings due to space limitations.

![Figure 8-8](image)

**Figure 8-8** Evaluation of optimal ICs’ parameters in the gPMI and IPMI configurations with a 25% IMR: (a) and (b) for the six-story; (c) and (d) for the 20-story building

Figures 8-8(a) and (c) illustrate that the stiffness terms of the gPMI configurations at the lowermost and uppermost stories are less than those of the IPMI configurations while at mid-stories an irregular trend is observed. As shown in Figures 8-8(b) and (d), the optimal ICs’ damping
ratios at almost all stories in the gPMI configurations are significantly less than that of the corresponding iPMI configurations (i.e., $g_{PMI}/i_{PMI} < 1.0$). Similar results were observed for the IMR of 50%.

Table 8-2 presents a comparison between the gPMI and iPMI systems for the example buildings at two different IMRs. In this table changes in the values of the OFs are illustrated (i.e., the values of $J_{gPMI}/J_{iPMI} < 1.0$). As seen, a gPMI configuration slightly outperforms its corresponding iPMI in terms of $J_{drift}$ (at best by 7% for the six-story building with a 25% IMR). In fact, in a gPMI configuration, unlike in an iPMI, ICs are permitted to have different stiffness and damping terms at different stories. This flexibility results in the mitigation of both higher-mode and fundamental-mode effects, and further reductions of inter-story drift demands. However, this improvement generally comes at the expense of drastically increased ICs’ acceleration demands (at worst 81% for the 20-story building with a 25% IMR), as more earthquake energy is transferred to these inherent vibration suppressors. Therefore, a gPMI configuration cannot be necessarily considered as a superior strategy compared to the iPMI configuration. Additionally, the iPMI system has the advantage of applying identical ICs at different stories. Therefore, this approach is likely to be a more practical design scheme than the gPMI configuration. In the next section, the iPMI system performance is further investigated.

<table>
<thead>
<tr>
<th>IMR</th>
<th>six-story</th>
<th>12-story</th>
<th>20-story</th>
<th>40-story</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$J_{drift}$</td>
<td>$J_{Moving accel.}$</td>
<td>$J_{drift}$</td>
<td>$J_{Moving accel.}$</td>
</tr>
<tr>
<td>25%</td>
<td>-6%</td>
<td>+70%</td>
<td>-6%</td>
<td>+98%</td>
</tr>
<tr>
<td>50%</td>
<td>-9%</td>
<td>+23%</td>
<td>-9%</td>
<td>+46%</td>
</tr>
</tbody>
</table>

Note that a negative percentage means an improved performance.

8.10 Effect of number of ICs in an iPMI system

The previous sections illustrate that the proposed iPMI technique can effectively mitigate structural seismic responses of the presented example buildings. However, this system needs component masses to be isolated at all stories, which may complicate the construction process, provide additional challenges in terms of feasible architectural layouts, and consequently increase an overall building’s cost, especially for high-rise buildings.
In a conventional TMD system, the roof level is usually the optimal location for placing the damper mass. However, the implementation of the proposed PMI system necessitates an evaluation of the number of ICs and their location along the height of the building. For example, if 5% of the total mass of a 20-story building is to be isolated, and an iPMI strategy is adopted, 5% of each story mass should be isolated. However, isolating a few components at upper stories with a larger IMR seems to be a more effective strategy (e.g., isolating 50% of masses of the top two stories which results in an equal overall isolated mass if 5% of every story mass is isolated). In other words, the iPMI configuration does not appear to be a seismically optimal solution in terms of the spatial allocation of the ICs, and hence, the IMRs.

In this section, a simple and practical approach is pursued to investigate the possibility of reducing the number of ICs, while retaining the major efficiency of the PMI control strategy. Four different IMRs of 5%, 25%, 50%, and 90% are selected. For each IMR, \( n \) configurations (\( n \) is the number of stories) are optimized using a parametric study. In the first configuration, one IC is assigned only for the roof, and in the second configuration, the top two stories are equipped with identical ICs. The pattern of adding one more identical IC one story at a time continues until the \( n \)-th configuration with \( n \) ICs (i.e., the previously studied iPMI configuration) is formed. As seen, the overall IMRs of different configurations are not the same; for example, the overall isolated mass in a configuration with ICs at the top three stories is three times as large as that of a system with only one IC at the roof. Parametric studies for different configurations are conducted, and the optimization results for the example buildings are presented in Figure 8-9 for the primary OF, i.e., \( J_{\text{drift}}^* \).
As Figure 8-9 demonstrates, for all configurations at a constant IMR, there is a near-saturation point for the primary OF. In other words, with increasing the number of ICs, the minimum value of the OF for a given configuration changes asymptotically. While the first few upper ICs are considerably effective in reducing $J_{\text{drift}}^s$, installing the next IC units at bottom stories is less effective. Consider an example performance level of $J_{\text{drift}}^s = 0.40$ in the 20-story building. In this case, installing two upper ICs with a 90% IMR is equivalent to installing four upper ICs with a 50% IMR, and also to installing eight upper ICs with a 25% IMR, while the overall isolated mass in the three configurations is approximately equal.

To further investigate the effect of number of ICs on the PMI system’s seismic performances, Figure 8-10 shows various OFs for several configurations of the PMI strategies in the 20-story building. According to Figure 8-10, in PMI configurations with 50% and 90% IMRs adding the 10-lower ICs can further improve the primary OF by 0.063 and 0.059 (i.e., $(J_{\text{drift}}^s)_{20} - (J_{\text{drift}}^s)_{10}$),
respectively but at the expense of increasing $J_{\text{Moving}}^{\text{displ.}}$ by 0.56 and 0.46, respectively. This behavior suggests that assigning ICs only at the upper-half stories can be considered as a reasonable solution to decrease construction costs without significantly compromising efficiency. It is also illustrated that a PMI system equipped with two identical top ICs having a 90% IMR performs very similarly to a PMI system with four identical top ICs having a 50% IMR.

![Graph showing various objective functions (OFs) for different configurations of the iPMI system in the 20-story building](image)

**Figure 8-10** Various objective functions (OFs) for different configurations of the iPMI system in the 20-story building

**8.11 Concluding remarks and future works**

In this study, a partial mass isolation (PMI) system, in which different portions of stories are isolated from the main structure, is studied. The PMI technique is examined in linear-elastic shear-building models with six, 12, 20, and 40 stories representing low to high-rise buildings with different fundamental periods. Optimization is carried out on the PMI system’s parameters to minimize average normalized root mean square (ANRMS) inter-story drift responses as the primary objective function (OF), while constraints are applied to control the isolated components’ (ICs) seismic responses. Passive tuned-mass-damper (TMD) and base-isolation (BI) systems are presented as appropriate baselines for assessing the effectiveness and applicability of the proposed PMI approach. Overall two configurations of the PMI system are studied. In the first configuration, denoted as the $i$PMI system, ICs are assigned identical properties at all stories. For this configuration, parametric studies are utilized to optimize $i$PMI systems with different isolated mass ratios (IMRs). In the second configuration, the stiffness and damping terms of ICs could vary from story to story to possibly provide a better seismic performance. For this configuration, considering the large number of unknown parameters to be optimized, a Genetic Algorithm (GA) is employed.
Simulation results indicate that three different ranges for the IMR could result in an iPMI system with behavior consistent with a TMD, BI, or a TMD-BI hybrid system. It is demonstrated that (i) ICs at a low IMR can provide a building with multiple inherent mass dampers without the weight restrictions of common TMDs; (ii) ICs at a high IMR can divide a building into a relatively stiff super-frame and flexible ICs, such a scenario conceptually behaves similarly to an ideal BI, except that the structural frame remains almost stationary as opposed to the entire structure experiencing rigid-body displacements relative to the base; (iii) an iPMI system at intermediate IMRs ranks between a common TMD and an ideal BI system. In balancing ICs seismic responses and the global inter-story drift and lateral-floor acceleration demands on the structure, and taking to account structural and architectural constraints, the authors recommend an intermediate IMR (25%-50%) as an appropriate range for the implementation of the iPMI technique. Such a system, as a partial isolation technique, would retain significant advantages of traditional TMD and BI techniques (i.e., inter-story drift and acceleration response reduction of the entire building) while preventing significant displacement and acceleration demands on the ICs. The GA-optimized PMI (gPMI) system, achieved by tuning ICs to different modes of vibration, could mitigate structural responses controlled by higher modes as well as those controlled by the structure’s fundamental mode. In addition, the gPMI system slightly outperforms the iPMI configuration in terms of inter-story drift response reduction. However, this improvement is achieved at the expense of large ICs’ displacement and acceleration demands.

The iPMI approach, although effective, considerably influences the design, construction, and serviceability of a building at multiple floors while a conventional passive control system (e.g., a TMD or a BI system) affects only one specific floor. To address this deficiency, seismic performance of the PMI technique with identical ICs at different subsets of stories (i.e., identical ICs at only roof, top two stories, etc.) is investigated. It is shown that ICs at a few top stories contribute the most to the PMI system efficiency. Hence, the number of ICs can be significantly reduced without a significant reduction in the efficiency of the PMI technique.

Optimizing the placement of ICs along a building height, when either the overall isolated mass or the number of ICs is limited to a specific value, needs a comprehensive study, which is in progress by the authors. The effect of isolating components on the vertical acceleration responses should be also studied. The PMI system’s efficiency in structural models with nonlinear seismic
behavior should be examined as well. The effect of torsional responses caused by the out-of-phase movements of ICs should be investigated using three-dimensional models.

In this study, it is shown that an optimized PMI system with identical ICs can perform as effectively as the GA-optimally designed PMI system (with possible dissimilar ICs). To make a more complete evaluation, the robustness of these two configurations with respect to the change in the system parameters as well as the ground excitation characteristics should be investigated. The aesthetical aspects and possible environmental benefits of the proposed PMI system should be considered. For example, if isolated units (floors) are capable of accommodating large movements in the horizontal directions, the proposed technique can optimally use the sunlight during the daytime and save the building energy consumption. This latter characteristic can justify the initial cost of this relatively new control system, given that the structural control improvement component is achieved only during rare ground motions in a building lifetime.

8.12 Appendix: verification of design characteristics of the near-optimized iPMI configurations

The optimal design characteristics of the iPMI system (i.e., $\gamma_{IC}$ and $\gamma_{Equiv.}^{IC}$) versus the IMR, obtained from the parametric studies, are depicted in Figure A-1. According to Figure A-1(a), for all example buildings, the optimal damping term of ICs increases with increasing the IMR. The damping ratio of the optimal ICs in the six-, 12-, 20-, and 40-story buildings reaches the upper admissible bound (i.e., $\gamma_{IC} = 0.30$) at 38%, 33%, 30%, and 20% IMRs, respectively. At IMRs less than 10% (i.e., the TMD-like region), the optimal damping ratios in the six- and 20-story buildings are approximately the same and are consistent with the solutions presented by Warburton (1982) for a single TMD attached to a single-degree-of-freedom (SDOF) primary system. However, in this region a significant discrepancy is observed for the 40-story building. The reason of this discrepancy is described next.
Figure A-1. Variation of ICs’ parameters versus IMR for the near-optimized iPMI configurations.

According to Figure A-1(b), the constraint $T_{\text{Equiv.}}^{\text{IC}} \leq 2.50$ s affects the six-, 12-, and 20-story buildings at IMRs larger than 83%, 78%, and 63% respectively, while for the 40-story building this constraint is activated at the beginning of the IMR interval, and thus affects the optimization results in the entire IMR range. The reasons for the observed discrepancies for the 40-story building are described in the following paragraph.

At low IMRs (e.g., consider $IMR = 0.10$), the PMI system exhibits dynamic responses that are similar to the responses of a TMD system. In this case, the ICs of the PMI system serve as inherent mass dampers. If in the optimization process constraints are not applied to the system parameters, such TMDs tend to tune to the fundamental period of the primary structure. The optimal TMD damping ratio, for the IMR range of interest, is usually less than 15% (see Warburton (1982) Table 5: Optimum parameters for absorbers attached to 1DOF damped systems excited by a stochastic acceleration process at the base). For the six-, 12- and 20-story buildings at low IMRs the above-mentioned optimal period and damping ratio satisfy the constraint $T_{\text{Equiv.}}^{\text{IC}} \leq 2.50$ s. Hence, the optimization is not affected by the applied constraints, and the optimal parameters comply with the solutions provided by Warburton (1982). However, for the 40-story building, for example at a low IMR of 5%, the first solution that can fulfill this criterion is associated with a damping ratio of 23%, which is significantly different from the expected value for an equivalent TMD. For this case, the ICs tend to tune to a fundamental period of about 4.0 s that is prevented by the constraint $T_{\text{Equiv.}}^{\text{IC}} \leq 2.50$ s (since $T_{\text{Equiv.}}^{\text{IC}} = T^{\text{IC}}/ (\sqrt{\text{IC}}/0.05)^{0.30}$, for $\text{IC} = 0.23$, if the elastic period $T^{\text{IC}}$ is 4.0 s,
$T_{\text{Equiv.}}^{\text{IC}}$ equals 2.54 s that exceeds the 2.50 limit). Hence, for the 40-story building, the constraint $T_{\text{Equiv.}}^{\text{IC}} \leq 2.50$ s is activated at the beginning of the interval, significantly affecting the optimization results.

8.13 Acknowledgment

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8.14 References


Chapter 9

Robust Seismic Design of a Multi-floor Isolation System
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Chapter 9

Robust Seismic Design of a Multi-floor Isolation System


9.1 Abstract

A Multi-floor isolation (MFI) technique can provide a building with inherent dynamic vibration suppressors through decoupling different portions of floors masses from the superstructure. This paper evaluates the seismic effectiveness and robustness of the MFI technique using a test-bed 20-story building. First, a parametric study approach is utilized to optimize MFI configurations with different isolated mass ratios (IMRs) and different number of isolated floor subsystems (IFSs). The response quantity to be optimized is the sum of root-mean-square inter-story drift responses of the superstructure under a stochastic excitation. The sensitivity of the seismic performance of the optimally-designed MFI configurations with respect to uncertainties in the properties of the superstructure, IFSs and the ground excitation is evaluated. Simulation results illustrate that with the presence of these uncertainties the effectiveness of the optimally-designed MFI configurations with low and high IMRs (e.g., 5% and 90%) is significantly impaired while configurations with intermediate IMRs (e.g., 50%) exhibit a relatively stable performance. Evaluation of the optimal placement of IFSs along the building height, when the number of IFSs is limited, reveals that placing the IFSs at top-stories can lead to a near-optimal system in terms of the spatial distribution of IFSs. A Genetic Algorithm is used to design a robust system for a selected configuration with top-10 IFSs and an IMR of 50%. The robust design is performed via minimizing the maximum deviation with respect to the design root-mean-square inter-story drift response when the design parameters vary within a given uncertainty range.
Keywords: Multi-floor isolation; Modified Kanai-Tajimi filter; Epistemic uncertainty; Aleatory variability; Robustness; Frequency Response; Genetic Algorithm.

9.2 Introduction

The structural seismic design process is associated with inherent uncertainties in the estimation of demand and capacity. These uncertainties are present due to differences between the assumed parameters in the design procedure, for the ground excitation and a structure, and the actual parameters, or due to the fluctuation of a structure’s parameters during its lifetime (i.e., aging). A system that exhibits a performance that is stable, i.e., not significantly affected by the mentioned uncertainties, is known as a robust system in the literature.

It is well-recognized that conventional seismic protection systems (e.g., moment resisting frames and shear walls) are not robust to deviations in the structure characteristics as well as the earthquake excitation parameters. For example, if the actual fundamental frequency of a conventional moment resisting frame system is larger than the one assumed in the design, the building may experience seismic force demands larger than those predicted in the design procedure. Many passive control systems such as conventional tuned-mass-damper (TMD) and base-isolation (BI) systems have been developed to enhance the structural seismic performance. These systems are capable of improving the structural responses at a specific design point (i.e., the optimal solution), but they are not generally effective over a wide range of design parameters. This implies that, for example, even a relatively small deviation in building fundamental frequency can degrade the seismic performance of these passive control systems.

Several research studies have incorporated uncertainties in the optimization process of TMDs (e.g., Papadimitriou et al. 1997; Marano and Greco 2008; Matta and De Stefano 2009; Dehghan-Niri et al. 2010; Marano et al. 2010; Chakraborty and Roy 2011; Mohtat and Dehghan-Niri 2011; Lucchini et al. 2013; Adam et al. 2014; Venanzi 2015; Yang et al. 2015; Rathi and Chakraborty 2017) and BI systems (e.g., Taflanidis et al. 2008; Takewaki 2008; Roy and Chakraborty 2015; Castaldo et al. 2016; Greco and Marano 2016). The results of these studies illustrate that to avoid unexpected performance deteriorations observed in conventional TMD and BI systems, the uncertainties must be accounted for. In other words, deterministic design of these passive control systems (i.e., ignoring the inherent uncertainties) can overestimate their seismic protection.
efficiency. To improve the robustness of the passive control systems, intelligent seismic protection systems such as active and semi-active control strategies have been proposed that are adaptive to input excitations with various frequency contents. However, these systems are associated with inherent deficiencies. For example, active control systems need external energy that cannot be ensured during long-return period ground motions. Furthermore, these systems are complicated and require sensors and controller equipment that may prohibit their application in typical buildings.

In this chapter, a multi-floor isolation (MFI) technique is studied that can provide an effective and robust passive seismic protection system. In an MFI approach, main floors are isolated from the superstructure at several stories to provide a building with multiple inherent vibration suppressors without imposing any additional mass to a building (Xiang and Nishitani 2015; Sakr 2015.http://dx.doi.org/10.1016/j.hbrcj.2015.04.004; Anajafi and Medina 2018a; Anajafi and Medina 2018b). As part of this study, several MFI configurations are deterministically optimized for a test-bed 20-story structure. Then, the sensitivity of the seismic performance of the optimally designed configurations with respect to deviations in the design parameters is investigated. Finally, a robust design procedure for a selected MFI configuration is presented.

9.3 Background

9.3.1 Epistemic uncertainties and aleatory variabilities

A major source of uncertainty arises from estimating and modeling the properties of a structure. These relevant properties include structural frequencies (or stiffness and mass), strength of materials, frequency shifts caused by the system inelasticity during a severe earthquake, hysteretic behavior of seismic fuses, viscous damping ratio of the superstructure, soil-structure-interaction effects, the contribution of nonstructural components (e.g., partition walls) to the structural frequencies and strength among others. The method that is used in modeling, analyzing and designing a structure also incorporates uncertainties into the problem. For example, the results of the response history analysis of a system using various engineering software packages may be different due to the diversity in the assumptions adopted in modeling the structural elements and solving the equations of motion. These types of uncertainties that arise from the imperfect models
of the real world, because of the insufficient or imperfect knowledge of reality, are denoted as “epistemic uncertainties”.

Another source of uncertainty in the seismic design of a structure deals with estimating the seismic demand. The routine seismic design of structures is generally performed based on simplified methods (e.g., the equivalent static lateral force method) that in many cases might not accurately represent the magnitude and distribution of actual ground-motion induced demands on structural and nonstructural components. Even when the design process incorporates advanced nonlinear response history analysis, there is no guarantee that the seismic-induced demands estimated in the analysis will be consistent with those experienced by the structure in a future earthquake. This type of uncertainty that is associated with the randomness of the underlying phenomenon is known as “aleatory variability”.

Epistemic uncertainties and aleatory variabilities can result in a new structural dynamics problem, i.e., a structure and a seismic excitation with characteristics that are different from those assumed in the design process. In this context, a system is robust if its performance is not very sensitive to these deviations. In the next subsection, the robustness features of two common passive control systems are briefly discussed to gain more insight into the problem.

9.3.2 Robustness of common passive control systems

Passive control strategies are widely used to enhance the seismic performance of structural and nonstructural systems. Tuned mass damper (TMD) and base isolation (BI) systems are two of the most utilized passive seismic protection techniques. These systems through adjusting the structural fundamental frequency and damping can substantially reduce the transmitted earthquake energy to building structures increasing the likelihood that a building remains operational after an earthquake event.

A TMD consists of an auxiliary mass-spring-dashpot system that is usually designed to oscillate at the same frequency as a primary system but in an opposite phase to attenuate undesired dynamic vibrations. Although a passive TMD system can be effective for a specific design assumption, it is not robust against deviations in the primary system characteristics (e.g., fundamental frequency and damping ratio) as well as changes in the frequency content of the input excitation with respect to the one assumed in the system’s design. A main disadvantage of a single
TMD is its sensitivity to the tuning frequency (Papadimitriou et al. 1997). Since a TMD is tuned to a specific frequency, it can only effectively suppress structural responses in a narrow frequency band and is less effective for excitations outside this frequency range. Furthermore, if structural elements undergo inelastic actions during an earthquake, the primary system’s frequency may shift, degrading the TMD’s expected performance (Constantinou et al. 1998; Sgobba and Marano 2010). Detuning may also occur due to inevitable errors and uncertainties in estimating the fundamental frequency of a building even if the system vibrates in the elastic range. An optimized TMD could perform very effectively in reducing a primary structure’s dynamic vibrations, but its effectiveness depends on the relation between ground motion characteristics and structural parameters (Bernal 1996; Soto-Brito and Ruiz 1999; Murudi and Mane 2004; Wang and Lin 2005; Marano and Greco 2008; Anajafi and Medina 2018a). For example, Wang and Lin (2005) and Anajafi and Medina (2018a) showed that when the predominant frequency of a building is located within the bandwidth of the external ground excitation spectrum, a TMD could consistently reduce the building seismic responses; however, when the external excitation frequency is less than the building fundamental frequency (i.e., the soil profile is soft relative to the building), the TMD system’s effectiveness significantly decreases. In another study, Tributsch and Adam (2012) illustrated that under near-filed earthquakes a TMD cannot reduce the structural displacement responses as efficiently as under far-field excitations. The TMD’s performance also depends on the primary system damping ratio such that this system is more effective in a low-damped primary building (Tributsch and Adam 2012; Lucchini et al. 2013). Some previous studies (e.g., Tributsch and Adam 2012; Anajafi and Medina 2018a) have shown that under certain conditions, a TMD could even slightly amplify the maximum structural responses. The abovementioned observations suggest that if uncertainties that affect the parameters of the system are not considered in the design, the TMD’s performance could be significantly overestimated.

BI systems can decouple dynamic responses of a building from the horizontal components of ground excitations by interposing low-horizontal stiffness bearings at the isolation interface (Skinner et al. 1993; Naeim and Kelly 1999). Like the TMDs, the efficiency of BI systems highly depends on the characteristics of the fixed-base superstructure, the control system (isolators in this case) as well as the input excitation (Chung et al. 1999; Kulkarni and Jangid 2002; Liao et al. 2004; Matsagar and Jangid 2004; Mishra and Chakraborty 2013; Castaldo et al. 2015, 2016; Castaldo
and Ripani 2016; Anajafi and Medina 2018a). For example, Anajafi and Medina (2018a) showed that under narrow-band excitations, the BI technique is effective only when the fixed-base superstructure is stiffer than the soil profile. In another study, Matsagar and Jangid (2004) and Kulkarni and Jangid (2002) illustrated that for nonlinear BI systems, the response of the structure is significantly influenced by the hysteretic response of the isolator bearings. They also concluded the superstructure flexibility could increase the floor acceleration responses in a BI system. Castaldo et al. (2016) through conducting a seismic reliability analysis on a friction pendulum isolation system indicated that the uncertainty characterizing the friction coefficient and the vertical components of the seismic excitations could negatively influence the system performance. Liao et al. (2004) showed that seismic isolation is more efficient to reduce the base shear when the structure is exposed to far-filed ground motions than to near-fault ground motions.

These observations illustrate that, although the passive TMD and BI systems are effective at a specific design point, any misestimate in the relevant design parameters can significantly deteriorate their seismic performance. Many research works have attempted to resolve the aforementioned concerns associated with the conventional TMD and BI techniques. For example, to enhance the robustness of TMDs, some researchers have suggested using multiple tuned-mass-dampers (MTMDs) that are tuned to multiple frequencies in a building (Yamaguchi and Harnpornchchai 1993; Abe and Fujino 1994; Igusa and Xu 1994; Kareem and Kline 1995). Although MTMDs can mitigate the detuning problem of a single TMD, they still require the addition of external masses to be attached to a building. Intelligent seismic protection systems such as active (e.g., see Chang and Soong 1980) and semi-active mass dampers (e.g., see Hrovat et al. 1983; Karnopp 1990) have been also proposed that are adaptive to input excitations with various frequency contents. However, these systems are associated with significant inherent deficiencies. In spite of the cost efficiency and reliability of the semi-active devices, their effectiveness is restricted within the limit of the maximum capacity of the passive devices on which they are based. Active control systems require significant external energy that cannot be ensured during long-return period natural hazards. They are complicated, and require sensors and controller equipment. Furthermore, active devices add or remove energy from the system, which may result in an unwanted or even unstable condition (Saaed et al. 2015).
This chapter aims to show that an MFI system is capable of providing a robust seismic performance even when exposed to large uncertainties in the design parameters, a feature that is lacking in many passive control systems such as conventional TMD and BI systems. In previous works the authors studied MFI configurations with different isolated mass ratios (IMRs) in six-, 12-, 20- and 40-story buildings (Anajafi and Medina 2018a; Anajafi and Medina 2018b). A deterministic design optimization was conducted through minimizing inter-story drift responses of the structural frame under stochastic excitations. It was shown that an MFI system with a low IMR (e.g., 5%) provides sufficient inherent mass to negate the need to add an external mass damper to the system; at a high IMR (e.g., 90%), an MFI system exhibits a global dynamic response reduction similar to the one obtained using a traditional base isolation system. Taking into account different performance objectives that deal with the mitigation of structural inter-story drift, displacement and acceleration responses of isolated floor subsystems (IFSs), as well as structural and architectural constraints, Anajafi and Medina (2018b) showed that MFI configurations with intermediate IMRs (e.g., 25%-50%) can provide an effective and efficient system.

The study presented in this chapter evaluates the robustness of different optimized MFI configurations using a test-bed 20-story structure. For the robustness analysis, in a strict sense, reliability-based methods in which each parameter is represented by means of a random variable should be applied (for reliability analysis of seismically isolated systems see e.g., Alhan and Gavin 2005; Chen et al. 2007; Castaldo et al. 2015, 2016). In this study, a sensitivity analysis is conducted to implicitly consider the inherent uncertainties associated with various design parameters while the probabilistic nature of the problem is not explicitly accounted for. Partial MFI configurations with different number of IFSs are optimized, and the optimal placement of IFSs along the building height is performed. In the optimization process, multiple performance objectives are introduced, and either a Genetic Algorithm (GA) or parametric studies are utilized when appropriate. Finally, a robust design procedure for a selected MFI configuration is presented.

### 9.4 Multi-floor isolation (MFI) system

#### 9.4.1 Modeling the system

The building superstructure is modeled as a linear-elastic shear-type two-dimensional frame (in this manuscript the term “superstructure” refers to the non-isolated part of the MFI system
including the structural frame). In this model story masses are lumped at floor levels and are assigned a single lateral degree of freedom. The masses of IFSs are ideally lumped at floor levels as additional degrees of freedom attached to the superstructure through a spring-dashpot system. The mathematical models of the uncontrolled and the MFI systems for an $N$-story shear building are illustrated in Figures 9-1(a) and (b), respectively.

![Figure 9-1 Schematic models of: (a) the uncontrolled system; (b) the passive MFI system; in an $N$-story linear-elastic shear building](image)

Numerical simulations are conducted based on a 20-story building with an identical story mass and an identical lateral story stiffness along the height. The Stiffness Proportional approach is used to approximate the viscous damping of the uncontrolled structure as well as the superstructure in the controlled system. The overall story mass (i.e., the mass of non-isolated part plus IFSs at each story) for all MFI configurations studied in this paper is $20 \times 10^6$ kg. Unless mentioned otherwise, a 2% damping ratio is assigned to the fundamental mode of the superstructure, and the lateral story stiffness of the superstructure is assumed to be $3.36 \times 10^7$ N/m. The abovementioned story mass and lateral stiffness characteristics render a fundamental frequency of $3.14$ rad/s (i.e., a period of $2.0$ s) for the uncontrolled system.

In this chapter, different configurations of the MFI system are studied. In terms of the spatial distributions of IFSs, two configurations are considered: (i) configurations with IFSs at every story; (ii) configurations with IFSs installed at a limited number of stories (e.g., top-10 stories). In terms of IFSs dynamic characteristics, two design schemes are defined: (i) configurations with
identical IFSs at each story; (ii) configurations with IFSs that may have dissimilar stiffness coefficients and/or dissimilar damping ratios at different stories while their IMR is still identical (it is realized that the design, construction and serviceability of an MFI system with IFSs’ masses that vary from story to story is associated with significant challenges, therefore, in all configurations studied in this study IFSs are assigned identical IMRs at every story).

For the optimization process, considering the number of unknown parameters either a Genetic Algorithm (GA) or a parametric study is employed. For MFI configurations with identical IFSs, once an IMR is assigned, only two parameters (i.e., IFSs stiffness and damping ratio) should be optimized; for this type of MFI system, parametric studies are used. For MFI configurations with dissimilar IFSs installed at \( n \) stories a total of \( 2n \) parameters should be optimized; in this case, considering the large number of unknown parameters, a GA is applied.

9.4.2 Governing equations and ground excitations

The equations of motions for an \( N \)-story linear-elastic shear-type building equipped with IFSs at every story can be expressed as

\[
\mathbf{M} \ddot{\mathbf{x}}(t) + \mathbf{C} \dot{\mathbf{x}}(t) + \mathbf{K} \mathbf{x}(t) = -\mathbf{M} \ddot{\mathbf{x}}_g(t) + \mathbf{f} \tag{9.1}
\]

where

\[
\mathbf{M} = \begin{bmatrix} \mathbf{M}^s & \mathbf{0} \\ \mathbf{0} & \mathbf{M}^{IFS} \end{bmatrix}_{2n \times 2n}; \quad \mathbf{C} = \begin{bmatrix} \mathbf{C}^s + \mathbf{C}^{IFS} & -\mathbf{C}^{IFS} \\ -\mathbf{C}^{IFS} & \mathbf{C}^{IFS} \end{bmatrix}_{2n \times 2n}; \quad \mathbf{K} = \begin{bmatrix} \mathbf{K}^s + \mathbf{K}^{IFS} & -\mathbf{K}^{IFS} \\ -\mathbf{K}^{IFS} & \mathbf{K}^{IFS} \end{bmatrix}_{2n \times 2n};
\]

\[
\mathbf{M}^s = \text{diag}(m_1^s, m_2^s, \cdots, m_n^s); \quad \mathbf{M}^{IFS} = \text{diag}(m_1^{IFS}, m_2^{IFS}, \cdots, m_n^{IFS}); \quad m_i^s + m_i^{IFS} = m_i^e;
\]

\[
\mathbf{C}^s = 2\xi_1^s/\omega_1^s \mathbf{K}^s; \quad \mathbf{C}^{IFS} = \text{diag}(c_1^{IFS}, c_2^{IFS}, \cdots, c_n^{IFS}); \quad c_i^{IFS} = 2\xi_i^{IFS} \sqrt{k_i^{IFS} m_i^{IFS}};
\]

\[
\mathbf{K}^s = \begin{bmatrix} k_1^s & k_2^s & -k_2^s & \cdots & 0 \\ -k_2^s & \ddots & \ddots & \vdots & \vdots \\ \vdots & \ddots & \ddots & k_n^s & -k_n^s \\ 0 & \cdots & -k_n^s & k_n^s \end{bmatrix}; \quad \mathbf{K}^{IFS} = \text{diag}(k_1^{IFS}, k_2^{IFS}, \cdots, k_n^{IFS}).
\]

\( \mathbf{M}, \mathbf{C} \) and \( \mathbf{K} \) are the global mass, damping, and stiffness coefficient matrices of the MFI system, respectively. Superscripts \( s \) and \( IFS \) stand for the superstructure and isolated floor subsystems, respectively. \( \mathbf{x} = [x_1^s x_2^s \cdots x_n^s x_1^{IFS} x_2^{IFS} \cdots x_n^{IFS}]^T \) is the displacement vector of the system relative
to the ground displacement, \( x_g(t) \). \( \mathbf{r} \) is a column vector of ones, and \( \ddot{x}_g(t) \) is the ground acceleration. \( \mathbf{f} \) is the external force vector of the system (e.g., wind). \( m^s_i \), \( m^{IFS}_i \) and \( m^e_i \) are the masses of the superstructure, of the IFS, and of the entire story at the \( i \)-th story, respectively. \( \xi^s_i \) and \( \xi^{IFS}_i \) are the damping ratio of the superstructure at the fundamental mode and the damping ratio of the IFS located at the \( i \)-th story, respectively. \( \omega^s_i \) is the superstructure fundamental frequency. \( k^s_i \) and \( k^{IFS}_i \) are the stiffness coefficients of the superstructure and of the IFS at the \( i \)-th story, respectively. Equation (9.1) can be represented in the state space form given by Equation (9.2):

\[
\dot{\mathbf{z}} = \mathbf{A}\mathbf{z} + \mathbf{B}\mathbf{u}
\]

where the state is \( \dot{\mathbf{z}}(t) = [\dot{x}^T \ \ddot{x}^T]^T \) and the coefficient matrices are defined as

\[
\mathbf{A} = \begin{bmatrix} \mathbf{0}_{2n \times 2n} & \mathbf{I}_{2n \times 2n} \\ -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C} \end{bmatrix}_{4n \times 4n} ; \quad \mathbf{B} = \begin{bmatrix} \mathbf{0}_{2n \times 1} & \mathbf{0}_{2n \times 2n} \\ -\mathbf{r}_{2n \times 1} & \mathbf{M}^{-1} \end{bmatrix}_{4n \times (2n+1)} ; \quad \mathbf{u} = [\ddot{x}_g \ \mathbf{f}^T]_{(2n+1) \times 1}^T.
\]

For earthquake loads when a passive control system is used, \( \mathbf{f} \) is assumed to be zero and \( \mathbf{u} \) depends only on the ground acceleration.

The earthquake-induced ground acceleration is modeled as a filtered Gaussian white noise process corresponding to the Kanai-Tajimi (K-T) spectrum (Soong and Grigoriu 1993). In this model, the ground acceleration of the earth surface layer is approximated by the absolute acceleration response of a linear single-degree-of-freedom (SDOF) oscillator exposed to a Gaussian white noise process. The white noise represents the earthquake acceleration at the bedrock, and the linear SDOF oscillator characterizes the filtering effects caused by the soil layers. The oscillator can be defined by two parameters, \( \xi_g \) and \( \omega_g \), which are interpreted as the characteristics frequency and damping ratio of the ground layers, respectively. The transfer function representation of the K-T model with respect to the absolute acceleration of the oscillator in the Laplace domain is

\[
\mathcal{F}_{KT}(s) = \frac{2\xi_g \omega_g s + \omega_g^2}{s^2 + 2\xi_g \omega_g s + \omega_g^2}
\]

(9.3)

where \( s \) is a complex number frequency parameter. The K-T filter amplifies frequency components of the input white noise around \( \omega_g \), attenuates its high-frequency components, but
does not influence the amplitude of its low-frequency components (for more details see Anajafi and Medina 2018a). This latter issue is important given that real earthquake ground motions generally have low energy contents at low frequencies. To overcome this deficiency, a low-cut second order filter, as shown in Equation (9.4), is typically added to the K-T filter (Clough and Penzien 1993).

\[
F_{CP}(s) = \frac{s^2}{s^2 + 2\xi_c \omega_c s + \omega_c^2}
\]

where \(\xi_c\) and \(\omega_c\) are the parameters of the additional filter that are incorporated to produce the desired filtering of low frequencies. The transfer function of the modified K-T filter is

\[
F(s) = F_{KT}(s) F_{CP}(s)
\]

Unless otherwise mentioned, parameters representing the ground excitation expressed by Equations (9.3) and (9.4) are selected as \(\omega_g = \pi \text{ rad/s}\) and \(\xi_g = 0.30\), \(\omega_c = 0.10 \omega_g\) and \(\xi_c = 0.60\), which can reflect a narrowband excitation (or a relatively soft soil profile) (Der Kiureghian and Neuenhofer 1991) corresponding to the resonance situation for the uncontrolled test-bed 20-story building. A K-T model with a predominant frequency in the vicinity of the fundamental frequency of the structure has been used in the literature in the robust design or the reliability assessment of passive control systems (e.g., see Marano et al. 2010; Chakraborty and Roy 2011; Greco and Marano 2016). The transfer function of the modified K-T model (i.e., Equation 9.5) can be incorporated into the state space equation presented in Equation (9.2); then, the state variables can be obtained by solving a Lyapunov equation (for detailed descriptions of the Lyapunov equation see Lutes and Sarkani 1997). Solving Equation (9.2) results in displacement and acceleration responses that are relative to the ground. Since relative acceleration responses are not relevant seismic demands for design and evaluation, the ground acceleration responses are added to these relative responses to obtain absolute accelerations that are used in the evaluation of the MFI control system.
9.5 Evaluation of the robustness of optimally-designed MFI configurations with identical IFSs

Assume an MFI system is deterministically designed based on the structural characteristics and ground motion parameters described in Section 9.4.2. Designing the MFI system results in IFSs damping ratios and stiffness coefficients associated with specific seismic structural responses. It is evident that estimating and modeling the superstructure dynamic characteristics, IFSs parameters, and the K-T excitation parameters, are associated with inherent uncertainties. Therefore, the actual responses of the system can be different than the design predictions. This section intends to evaluate and quantify the robustness of the seismic performance of the MFI system in the presence of the mentioned uncertainties. First, MFI configurations with five different IMRs of 5%, 10%, 25%, 50% and 90%, which represent systems with low, intermediate and high control masses, are designed based on a deterministic optimization process. Then, in order to assess the robustness of the MFI system, the sensitivity of the seismic responses of the optimally-designed configurations with respect to variations in the structural characteristics and ground excitation parameters is investigated.

9.5.1 Design of MFI systems based on a deterministic optimization process

Given that inter-story drift responses are correlated with structural and nonstructural damages, the “normalized sum of root-mean-square (RMS) inter-story drift responses” of the superstructure is selected as the primary objective function (OF) to be minimized

\[ J_{\text{drift}} = \frac{\sum_{i=1}^{n} \text{RMS}(x_i^s - x_{i-1}^s)_c}{\sum_{i=1}^{n} \text{RMS}(x_i^s - x_{i-1}^s)_u} \]

where \( x_i^s \) is the superstructure displacement relative to the ground at the \( i \)-th floor level (i.e., \( x_0^s = 0 \)). Subscripts u and c stand for the uncontrolled and controlled structures, respectively. After optimizing the system, through minimization of \( J_{\text{drift}}^s \), three additional performance indices are introduced to evaluate acceleration responses of the entire building, IFSs drift and IFSs acceleration responses, as shown in Equations (9-7), (9-8) and (9-9), respectively.

\[ J_{\text{accel}}^w = \frac{\sum_{i=1}^{n} [\text{RMS}(\dot{y}_i^s)_c(1 - IMR_i) + \text{RMS}(\dot{y}_i^{IFS})_c IMR_i]}{\sum_{i=1}^{n} \text{RMS}(\dot{y}_i^s)_u} \]

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\[ J_{\text{IFS}}^{\text{drift}} = \frac{\text{sum}[\text{RMS}(x^*_l - x^*_c)]}{\text{sum}[\text{RMS}(x^*_l)_{\text{u}}]} \]  
\[ J_{\text{IFS}}^{\text{accel.}} = \frac{\text{sum}[\text{RMS}(\ddot{y}_l^\text{IFS})_{\text{c}}]}{\text{sum}[\text{RMS}(\ddot{y}_l^\text{IFS})_{\text{u}}]} \]  

where \( \ddot{y}_l^s \) and \( \ddot{y}_l^\text{IFS} \) are the absolute acceleration (i.e., the total acceleration including the ground acceleration) responses of the superstructure and of the IFS at the \( i \)-th story, respectively. \( IMR \) is the ratio of the isolated floor’s mass to the entire story mass at the \( i \)-th story. According to Equation (9.7), \( J_{\text{IFS}}^{\text{accel.}} \) is a weighted OF that combines the absolute acceleration responses of both the non-isolated part and the IFSs in the MFI system, and renders a single OF which allows comparison of the overall acceleration response of the controlled system with that of the uncontrolled system. In Equations (9.8) and (9.9) the subscript \( l \) stands for the location of IFSs within the building height and can vary from 1 to 20 (e.g., \( x^*_l \) is the IFS displacement relative to the ground at the \( l \)-th story).

To prevent the selection of impractical design parameters in the optimization process, the IFSs’ damping ratio, \( \xi^\text{IFS} \), is bounded to [0.02-0.30]; and the equivalent elastic period of IFSs (in seconds), \( T_{\text{Equiv.}}^\text{IFS} = T^\text{IFS} / \beta_L \), is limited to [0.25-2.50 s] where \( \beta_L = (\xi^\text{IFS} / 0.05)^{0.30} \) is used to account for the IFSs’ damping ratio other than 5%.

First, an MFI system with identical IFSs installed at all stories, denoted as \( i \text{MFI} \), is considered. A parametric study is performed to optimize \( i \text{MFI} \) configurations with different \( IMRs \). The primary OF, \( J_{\text{IFS}}^{\text{drift}} \), is calculated over a wide range of IFS’s damping ratio and stiffness coefficient. Then the minimum value of \( J_{\text{IFS}}^{\text{drift}} \) that is associated with the IFS parameters that are within the previously mentioned admissible ranges is selected as the optimal solution. Figure 9-2 depicts the variation of \( J_{\text{IFS}}^{\text{drift}} \) versus the IFS’s damping ratio and stiffness coefficient for an example \( i \text{MFI} \) configuration with an \( IMR \) of 25%. The same procedure is performed for other \( IMRs \) of interest, and the optimization results are presented in Table 9-1.
Figure 9-2  Variation of primary OF vs. IFSs’ parameters for an iMFI configuration with a 25% IMR

Table 9-1  Design characteristics of the optimized iMFI configurations with different IMRs

<table>
<thead>
<tr>
<th>IMR</th>
<th>(10^4 (k_{IFS}/k_s))</th>
<th>(\xi_{IFS})</th>
<th>(J_{drift}^s)</th>
<th>(J_{accel}^w)</th>
<th>(J_{IFS_{drift}})</th>
<th>(J_{IFS_{accel}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>2.6</td>
<td>10.5%</td>
<td>0.48</td>
<td>0.56</td>
<td>1.84</td>
<td>1.65</td>
</tr>
<tr>
<td>10%</td>
<td>4.7</td>
<td>14.5%</td>
<td>0.39</td>
<td>0.49</td>
<td>1.22</td>
<td>1.00</td>
</tr>
<tr>
<td>25%</td>
<td>8.2</td>
<td>21.0%</td>
<td>0.26</td>
<td>0.38</td>
<td>0.78</td>
<td>0.47</td>
</tr>
<tr>
<td>50%</td>
<td>6.6</td>
<td>29.0%</td>
<td>0.16</td>
<td>0.25</td>
<td>0.74</td>
<td>0.20</td>
</tr>
<tr>
<td>75%</td>
<td>9.6</td>
<td>30.0%</td>
<td>0.13</td>
<td>0.19</td>
<td>0.69</td>
<td>0.18</td>
</tr>
<tr>
<td>90%</td>
<td>12.0</td>
<td>30.0%</td>
<td>0.13</td>
<td>0.17</td>
<td>0.67</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Since in this study the evaluation of the seismic responses of the MFI system is conducted in relative terms, i.e., with respect to the seismic performance of the uncontrolled structure, an OF with a value smaller than 1.0 implies a performance improvement. According to Table 9.1, with isolating only 5% of each story mass, 52% and 44% reductions (i.e., \(1 - J\)) are obtained in the inter-story drift and weighted acceleration responses of the controlled system with respect to the uncontrolled building responses, respectively. However, these improvements come at the expense of relatively large responses of IFSs (i.e., \(J_{IFS_{drift}} = 1.84\) and \(J_{IFS_{accel}} = 1.65\)). An evaluation of Table 1 illustrates that selecting intermediate IMRs (e.g., 50%), while providing an extra reduction in the primary OF, significantly reduces the three other performance indices. For example, increasing the IMR from 5% to 50% provides an extra reduction of 32% in \(J_{drift}^s\) and a simultaneous improvement of 110% in \(J_{IFS_{drift}}\). Increasing the IMR up to an extreme value of 90% provides a negligible improvement in seismic performance with respect to the intermediate IMRs (e.g., only 3%
additional reduction in $J_{d, \text{drift}}^s$ compared to the 50% IMR). These observations suggest that applying an intermediate IMR can result in an effective and efficient MFI system.

9.5.2 Sensitivity of the seismic performance of the optimally-designed iMFI configurations to epistemic uncertainties and aleatory variabilities

9.5.2.1 Frequency response of the optimized iMFI configurations

Before conducting the sensitivity analysis, the frequency response of the optimized iMFI configurations, which can give insight into understanding the effect of deviation in the ground excitation dominant frequency, is studied. Taking the Laplace Transform from Equation (9.1) results in

$$X(Ms^2 + Cs + K) = -Mr\ddot{X}_g$$

(9.10)

The transfer function of the superstructure relative displacement with respect to the ground acceleration can be obtained as

$$H(s) = \frac{X}{\ddot{X}_g} = \frac{-Mr}{Ms^2 + Cs + K}$$

(9.11)

Substituting $s = \omega j$, where $j = \sqrt{-1}$, the transfer function in the frequency domain is

$$H(\omega j) = \frac{X}{\ddot{X}_g} = \frac{-Mr}{-M\omega^2 + C\omega j + K}$$

(9.12)

The magnitude of the transfer function at the roof level versus the normalized frequency, $\omega / \omega_s$, for configurations with different IMRs is plotted in Figure 9-3(a). As seen, the operating range (i.e., the distance between the two spikes in the frequency response) of the iMFI system at low IMRs (e.g., 5% and 10%) is relatively small compared to those at intermediate IMRs (e.g., 25% and 50%). The implication is that, for example, a relatively small fluctuation of +/-10% in the normalized frequency renders the iMFI system with a 5% IMR virtually ineffective while at larger IMRs such as 50% the system is effective in a broad frequency bandwidth. These observations suggest that an iMFI configuration with an intermediate IMR is more robust than a configuration with a low IMR. Another test to fully assess the robustness of the proposed approach is to evaluate the deviation in the system responses over a given frequency bandwidth. In this context, a flatter frequency response implies a more robust behavior (i.e., the roof displacement...
response is less dependent on the normalized frequency). To investigate this behavior, in Figure 9.3(b) for each configuration the magnitude of the transfer functions, $|H(\omega_j)|$, is normalized to the magnitude of its transfer function at $\omega/\omega_s = 1.0$, $|H(\omega)|_o$ (i.e., the magnitude at the resonance condition that is assumed as the design case). As seen, for a relatively small deviation from $\omega/\omega_s = 1.0$, the transfer function of the iMFI system with intermediate IMRs exhibits a lower variation; for example, when $\omega/\omega_s$ changes between 0 and -22% with respect to $\omega/\omega_s = 1.0$, the iMFI with a 50% IMR is associated with the least performance sensitivity. For relatively large changes in $\omega/\omega_s$ with respect to $\omega/\omega_s = 1.0$, (e.g., beyond -44%) with increasing the IMR, the magnitude of the normalized transfer function consistently increases. This latter observation suggests that iMFI configurations with large IMRs, such as 90%, fail to provide the required level of robustness when exposed to relatively large deviations in the predominant frequency of the ground excitation.

![Graph](image_url)

(a) absolute values, $|H(\omega)|$

(b) normalized to $|H(\omega)|$ at $\omega/\omega_s = 1.0$

Figure 9-3 Magnitude of the transfer function of the superstructure roof displacement for the optimally-designed iMFI configurations with different IMRs

9.5.2.2 Evaluation of the effectiveness and sensitivity of the performance of the MFI system exposed to variations in the design parameters

In this section, the robustness of the performance of the optimally-designed iMFI configurations is evaluated based on two criteria that are described next. (i) Effectiveness criterion – the effectiveness of the iMFI system when variations in the design parameters are assumed is estimated by calculating $J_{\text{drift}}^d$ for many discretized points in a given uncertainty range. This criterion reveals whether or not the iMFI system is still effective, when, for example, the
superstructure stiffness deviates from the design value by a given ratio. If the maximum value of \( J_{\text{drift}}^s \) over the variation range of interest is less than unity, it means that the MFI system is effective when exposed to uncertainty. (ii) Sensitivity criterion – the sensitivity of the seismic response of an optimally-designed system is quantified by normalizing “the values of the seismic responses of the system taking into account deviations with respect to the optimal solution parameters” to the values obtained from the optimization process (see Equation 9-13).

\[
SM = \frac{\sum_{i=1}^{n} \text{RMS}(x_i^s - x_{i-1}^s)_{c} \text{with uncertainty}}{\sum_{i=1}^{n} \text{RMS}(x_i^s - x_{i-1}^s)_{\text{designed}}}
\]  

(9.13)

The sensitivity criterion quantifies the variation of the inter-story drift responses of the system due to the uncertainties with respect to the inter-story drift responses obtained using the optimal, deterministic design parameters. An iMFI configuration that over a wide range of the design parameters exhibits consistently small \( J_{\text{drift}}^s \) values, and \( SM \) values near unity, is considered a perfectly robust system.

The proposed approach to quantify the variation in structural responses with respect to changes in the design (structural and ground motion) parameters can be applicable to any other passive control systems and buildings with different heights. First, in this evaluation process, it is assumed that only a single design parameter can vary at a time (i.e., only the uncertainty in one parameter is taken into account).

Figures 9-4(a) and (b) depict the variation of \( J_{\text{drift}}^s \) and \( SM \) versus the normalized superstructure stiffness coefficient, \( k^s/k_o^s \), respectively, where \( k_o^s \) is the superstructure stiffness coefficient associated with the optimal (design) solution. As seen in Figures 9-4(a) and (b), a reduction in the superstructure stiffness can significantly impair the seismic performance of the MFI configurations in terms of both \( J_{\text{drift}}^s \) and \( SM \). In other words, for \( k^s < k_o^s \), the value of \( J_{\text{drift}}^s \) increases, and the value of \( SM \) also becomes larger than unity. The main reasons for these performance degradations are described next: (i) in Section 9.5.1 the optimization was conducted assuming a resonance condition. As illustrated by Anajafi and Medina (2018a), the best performance (i.e., the least \( J_{\text{drift}}^s \) value) of an MFI system is achieved when the superstructure fundamental frequency coincides with the predominant frequency of the K-T excitation (i.e., the resonance condition). With reducing the superstructure stiffness, the fundamental frequency of the structure is no longer tuned
to the K-T excitation predominant frequency. As a result, the MFI system effectiveness in terms of improving $J_{drift}^s$ decreases. (ii) In terms of $SM$, the observed degradation is because of the shape of displacement spectrum of the K-T excitation. For the considered K-T model, with decreasing the structural frequency, RMS displacement responses tend to increase, and as a result, the SM values become larger than unity.

As seen in Figure 9-4(a), a deviation with respect to the optimal value of the superstructure stiffness can significantly deteriorate the performance of the iMFI systems with low IMRs; for example, for an IMR of 5%, at a superstructure stiffness of $0.50k_0^s$ the performance index $J_{drift}^s$ equals 1.22. The implication is that applying this iMFI system for the building whose actual superstructure stiffness is 50% of the one assumed in the optimization process, can even increase the sum of RMS inter-story drift responses by 22% ($1 - J_{drift}^s$). As seen, for larger IMRs (e.g., 90%) when the superstructure stiffness is varied by +/-50%, the iMFI system loses part of its effectiveness but it is still effective over the wide range of superstructure stiffness considered. An evaluation of $SM$ values presented in Figure 9-4(b), illustrates that, for example, the maximum value of $SM$ for the configurations with 5%, 50% and 90% is 2.56, 2.12 and 2.01, respectively, revealing that the iMFI system with a 50% IMR exhibits a more stable performance compared to the other considered configurations.

The same analyses as the ones presented in Figures 9-4(a) and (b) are performed considering variation in the superstructure’s damping ratio, $\xi^s$. Simulation results illustrate that generally the iMFI system is more effective for a low-damped superstructure. However, an evaluation of the $SM$
criterion illustrates that when assuming a +/-50% variation in $\xi$, no significant changes occur in the sum of RMS inter-story drift response of the considered iMFI configurations. For the worst case (the iMFI configuration with a 5% IMR), the value of $SM$ is limited to 1.07, meaning only 7% response sensitivity for the defined uncertainty range. The figures illustrating the effects of the variation in $\xi$ are not presented herein for brevity. Figures 9-5(a) and (b) illustrate the variation of $J_{drift}^S$ and $SM$ with respect to the change in the IFS’s stiffness coefficient, respectively. In these figures, $k_{IFS}^O$ is the IFS stiffness coefficient associated with the optimal iMFI configurations. As shown in Figure 9-5(a), a variation in $k_{IFS}$ can significantly degrade the efficiency of the configurations with low IMRs. For example, in the iMFI configuration with a 5% IMR a +50% change in $k_{IFS}$ modifies $J_{drift}^S$ from 0.48 to 0.71. The reason for this performance deterioration can be described as following: IFSs with a low IMR serve as inherent TMDs, and with changes in their stiffness with respect to the optimal values, they are no longer tuned to the fundamental frequency of the building (i.e., detuning can impair a TMD’s effectiveness). Figure 9-5(b) illustrates that inter-story drift responses of the iMFI at low and high IMRs are sensitive to the deviations in $k_{IFS}$ while the configuration with an intermediate IMR of 50% exhibits a stable response (it is well-understood that a relatively flat $SM$ curve); as seen for the range of $k_{IFS}$ considered herein, the maximum value of $SM$ for the configuration with a 5% IMR and 90% IMR is 1.49 and 1.23, respectively whereas this quantity for the configuration with a 50% IMR is limited to 1.06.

![Figure 9-5](attachment:Figure_9-5.png)

*Figure 9-5  Variation in the seismic performance of the optimally-designed iMFI configurations versus the IFS stiffness coefficient*

Figures 9-6(a) and (b) depict the variation of $J_{drift}^S$ and $SM$ with respect to the change in the IFS’s damping ratio, respectively. According to Figure 9-6(a), the effectives of different iMFI
configurations is not highly degraded with deviations in IFS’s damping ratio (all $J_{\text{drift}}^S$ curves are relatively flat). An evaluation of the $SM$ values shown in Figure 9-6(b) reveals that the inter-story drift response of the $i$MFI with a high $IMR$ is more dependent on the IFS’s damping ratio than that of the $i$MFI with a low $IMR$. As seen, while for the $IMR$ of 90% the maximum value of $SM$ is 1.25, this quantity for the other configurations is bounded by 1.11. From Figures 9-6(a) and (b), it can be concluded that $i$MFI configurations with low and intermediate $IMRs$ are more robust to changes in the IFS’s damping ratio than configurations with high $IMRs$.

![Figure 9-6](image)

**Figure 9-6** Variation in the seismic performance of the optimally-designed $i$MFI configurations versus the IFS damping ratio

Figures 9-7 and 9-8 plot the sensitivity analysis results for different $i$MFI configurations exposed to aleatory variabilities (i.e., variation in the K-T excitation predominant frequency, $\omega^g$, and damping ratio, $\xi^g$). Figure 9-7(a) shows that with a deviation in the predominant frequency of the K-T excitation, especially a reduction, the effectiveness of all $i$MFI configurations decreases. As seen, with a -50% change in the normalized predominant frequency, $J_{\text{drift}}^S$ for the configuration with a 5% $IMR$ (the worst case) increases from 0.48 to 0.67, and for configuration with a 50% $IMR$ (the best case) increases from 0.16 to 0.50. The main reason for these performance degradations, especially in configurations with low $IMRs$, is the detuning between the IFSs frequencies as inherent TMDs and the ground excitation predominant frequency. This observation also suggests that the MFI approach is more effective for the resonance condition (i.e., the design assumption for the optimization process), and when the soil profile is softer than the superstructure, the system efficiency decreases. An evaluation of Figure 9-7(b) illustrates that the inter-story drift responses of the $i$MFI configurations with low $IMR$ values (e.g., 5% and 10%) are not negatively affected by variations in the predominant frequency of the ground excitation while at $IMR$ values of 50% and
90% a change of 50% in the ground excitation predominant frequency can increase the sum of the RMS inter-story drift responses by a factor up to 1.32.

Figure 9-8(a) illustrates that the negative effect of variations in the K-T filter damping ratio, $\xi^g$, on the performance of the configurations with larger IMRs is more pronounced. For example, a +50% deviation in $\xi^g$ increases $J_{\text{drift}}^S$ of the configuration with an IMR of 5% from 0.48 to 0.50 (i.e., only 4% worse) while the same deviation increases $J_{\text{drift}}^S$ of the configuration with a 90% IMR from 0.13 to 0.16 (i.e., 23% worse). Figure 9-8(b) reveals that changes in $\xi^g$ can significantly affect the sum of the RMS inter-story drift responses, especially for low IMRs. For example, in the iMFI configuration with a 5% IMR, a -50% deviation can increase the response by 51% while for the same deviation, the increase in the response of the configuration with a 90% IMR is limited to 7%.
An evaluation of Figures 9-4 to 9-8 reveals that the variation (reduction) in the superstructure lateral stiffness coefficient is more detrimental than changes in other design parameters (i.e., IFSs and ground excitation characteristics). As observed in Figure 9-4(a), for all configurations when the superstructure stiffness decreases, the MFI system’s effectiveness heavily deteriorates.

In the analyses conducted in Figures 9-4 to 9-8 it was assumed that only a single design parameter changes at a time. Simultaneous deviations with respect to the design parameters would have the potential to deteriorate the seismic performance of the system at a higher rate. For instance, Figures 9-9(a) to (c) depict sensitivity of three selected optimally-designed iMFI configurations (i.e., IMRs of 5%, 50% and 90%) with respect to the simultaneous variation in the K-T excitation parameters. As seen in Figure 9-9(a), for the iMFI system with an IMR of 5%, a simultaneous deviation of 9% in \( \omega_g \) and 50% in \( \xi_g \) can result in a SM of 1.55 (i.e., 55% increase in the sum of RMS inter-story drift response with respect to the one obtained in the optimization process). According to Figures 9-9(b) and 9-9(c), for IMRs of 50% and 90% the simultaneous variation in both parameters can worsen the system responses by a factor up to 1.93 and 1.82, respectively. These results can be compared with the ones observed in Figures 9-7(b) and 9-8(b), where only a single K-T parameter changes at a time. For example, for the iMFI configuration with a 90% IMR according to Figures 9-7(b) and 9-8(b), changes in the predominant frequency and damping ratio of the K-T can result in SM factors up to 1.31 and 1.07, respectively, which are significantly less than the SM factor of 1.92 observed in Figure 9-9(c).

![Figure 9-9 Sensitivity of the optimally-designed iMFI configurations with respect to the changes in the K-T excitation predominant frequency and damping ratio](image)

To evaluate the effect of the simultaneous deviations in the design parameters on the seismic performance of the iMFI system, vectors that account for epistemic uncertainties and aleatory...
variabilities, denoted as $v_{\text{epistemic}} = [k^s, \xi^s, k^{\text{IFS}}, \xi^{\text{IFS}}]$ and $v_{\text{aleatory}} = [\omega^g, \xi^g]$, are defined. It is assumed that each element of the epistemic uncertainty and aleatory variability vectors is associated with a relatively large variation of ±50%. This range is deemed to be reasonably conservative for the parameters evaluated and covers typical ranges used in previous studies related to the evaluation of the robustness of passive control systems, e.g., TMD systems, considering the uncertainty in design characteristics. For instance, Chakraborty and Roy (2011) assumed variations of up to 20% for the frequency and damping ratio of a TMD system. Igusa and Xu (1994) considered ±10% deviations in the predominant frequency of the main structure. Hoang et al. (2008) applied a 20% variation in the primary system frequency. Adam et al. (2014) assumed that the actual value of the TMD’s frequency and damping ratio and also the K-T filter parameters (i.e., predominate frequency and damping ratio) lie in a range of ±40% of their nominal. Marano et al. (2008) applied levels of uncertainty of up to 50% in lateral structural stiffness and damping ratio, TMD’s mass ratio, and the K-T filter parameters. Mohtat and Dehghan-Niri (2011) assumed uncertainty levels up to 10%, 20% and 75% for the mass of each floor, lateral stiffness of the first floor and the damping ratio of the first mode, respectively.

A GA is used to find the minimum and maximum $J^{s}_{\text{drift}}$ values that can occur in the defined range of parameters for configurations with different IMRs. This strategy can illustrate the best and the worst possible seismic performance of the MFI systems for the assumed uncertainty range. The same procedure is performed for the $SM$. For the vector $v_{\text{epistemic}}$ Figures 10a and 10b illustrate variations in $J^{s}_{\text{drift}}$ and $SM$ for different IMRs, respectively. In these figures, the design points (i.e., the responses associated with the deterministically optimized configurations shown in Table 9-1) are also presented. As shown in Figure 9-10(a), with the presence of epistemic uncertainties, an iMFI at an IMR of 50% with a maximum $J^{s}_{\text{drift}}$ of 0.34 outperforms other configurations. At low IMRs, especially 5%, the maximum $J^{s}_{\text{drift}}$ is near unity, implying that the system effectiveness is, for practical purposes, nonexistent. Figure 9-10(b) illustrates that the sum of RMS inter-story drift response of the configuration with an IMR of 50% is less affected by the presence of uncertainty in the design parameters than the response of other configurations. As seen for IMR = 50%, the $SM$ value is limited to 2.86 while for other configurations this quantity can be as large as 4.47.
Figures 9-10 illustrate the variation in the performance of the optimally-designed iMFI configurations with different IMRs due to the epistemic uncertainties.

Figures 9-11(a) and (b) illustrate the same analysis for the aleatory uncertainty vector.

As seen in Figure 9-11(a), for all configurations, especially for the IMR of 90%, variations in the ground motion parameters (i.e., aleatory variabilities) significantly deteriorate the iMFI system effectiveness such that the maximum $J_{\text{drift}}$ for all configurations exceeds 0.79. For low IMRs (e.g., 5% and 10%), this performance deterioration occurs because the IFSs frequencies are no longer tuned to the predominant frequency of the ground motion. For the high IMRs (e.g., 90%) this observation is consistent with the performance of base-isolated buildings. As shown by Anajafi and Medina (2018a) a base-isolation system is effective only if the fundamental frequency of the fixed-base superstructure is greater than that of the soil profile, whereas, in the sensitivity analysis conducted for the iMFI system, the ground motion predominant frequency can be smaller than the
uncontrolled superstructure frequency of $\pi$ rad/s by a factor up to 2.0. An evaluation of Figure 9-11(b) reveals that all iMFI configurations exhibit an approximately same level of sensitivity to the magnitude of the selected aleatory variabilities; in this case the configuration having an IMR of 50% with an SM value of 1.93 is the most sensitive configuration. In Section 9.6, as an initial step toward designing a robust MFI system, several MFI configurations are optimally designed in terms of the spatial distribution of the IFSs. Later in Section 9.7, for a selected configuration with top-10 IFSs, a robust design procedure is proposed.

9.6 Partial MFI configurations with different numbers of IFSs

9.6.1 Deterministic optimization of a partial MFI system with IFSs installed at a subset of upper stories using a parametric study

Installing IFSs at all stories may complicate the building’s construction, conflict with architectural design considerations, and consequently increase the overall building’s cost, especially for high-rise buildings. Such considerations may limit the number of IFSs or the total mass that can be decoupled in a building. From a practical point of view, fundamental issues need to be evaluated when considering the incorporation of an MFI system in the design process. For instance, if a designer is limited to isolating only 5% of the mass of a 20-story building, there will be a need to evaluate design alternatives and select the most effective one. In this case, an option would be to adopt an iMFI strategy, i.e., the design could be based on decoupling 5% of the mass of every story. Alternatively, a designer could consider using a few IFSs only at the upper stories with a larger IMR (e.g., isolating 50% of the mass of the top-two stories). Selection between these alternatives, in addition to structural seismic performance considerations, depends on several parameters such as architectural preferences, cost-benefit concerns, etc.; however, given that in a conventional TMD system the roof level is usually the optimal location for placing the damper’s mass, using a few IFSs with a larger IMR has the potential to be more effective in mitigating structural seismic responses than using IFSs at every story with a lower IMR.

To gain insight into this problem, provide quantitative information, and facilitate the selection of a practical and efficient design option, partial MFI configurations with different number of IFSs are studied. For this investigation, five different IMRs of 5%, 10%, 25%, 50% and 90% are selected. For each IMR, 20 configurations are optimized (through minimizing $J_{\text{drift}}$) using a
parametric study. In the first configuration, an IFS is placed at the roof level only, and the system is optimized; in the second configuration, the top two stories are equipped with identical IFSs, and a new optimization is performed. The pattern of generating new MFI configurations by adding identical IFSs (from top to bottom) one story at a time continues until the 20th configuration with 20 IFSs is formed (i.e., the full iMFI system previously optimized in Section 9.5.1). Different OFs, defined through Equations 9-6 to 9-9, for the optimized MFI configurations are plotted in Figures 9-12(a) to (d).

As shown in Figure 9-12(a), for MFI systems with the same overall control mass (isolated mass), a configuration equipped with fewer top IFSs having a larger IMR is more effective than a configuration with more IFSs having a lower IMR. For instance, an MFI configuration equipped with top-two IFSs having an IMR of 50% outperforms the configuration with 20 IFSs having an IMR of 5% by 17% (i.e., $J_{\text{drift}}^s$ of 0.40 versus 0.48) while their overall isolated masses are equal. Figures 9-12(a) and (b) demonstrate near-saturation performance levels for all MFI configurations regardless of the value of IMR. The configurations with just a few IFSs are considerably effective in reducing $J_{\text{drift}}^s$ and $J_{\text{accel}}^w$, whereas installing additional IFSs at lower stories is less effective. For instance, at 50% and 90% IMRs while a configuration with top-10 IFSs reduces the sum of RMS inter-story drift responses by 80% and 86% (i.e., $1 - J_{\text{drift}}^s$), respectively, adding IFSs at the bottom-10 stories can further improve $J_{\text{drift}}^s$ by only 4% and 1%, respectively. This trend suggests that placing IFSs only at the top half of the structure can be considered as a reasonable solution to decrease construction costs without compromising efficiency.

According to Figures 9-12(c) and (d), for the MFI configurations with low IMRs (e.g., 5%), IFSs experience relatively large seismic responses (e.g., OF values as large as 8.0 in some cases). At larger IMRs (e.g., 50%), in addition to significant reductions in inter-story drift and weighted acceleration responses of the primary building, such adverse effects are not observed. These results suggest that using an MFI configuration with an intermediate IMR (e.g., 50%) with only a few IFSs (e.g., at the top half of the building) is the best candidate configuration to provide effective seismic protection in the most practical way.
9.6.2 Optimal placement of IFSs in an MFI system

Sections 9.5 and 9.6.1 illustrate that in terms of robustness and effectiveness, the MFI configurations with intermediate IMRs (e.g., 25% and 50%) outperform configurations with extremely low and high IMRs. In the rest of this chapter, the intermediate IMRs are further studied.

In Section 9.6.1, based on a fundamental understanding of the dynamic behavior of TMD systems, IFSs were located at the top stories. In this section, the optimal placement of IFSs along the building height for selected MFI configurations with 10 IFSs and IMRs of 25% and 50% is performed. IFSs are allowed to have different characteristics (i.e., different stiffness coefficients and different damping ratios). Hence, for an MFI system with a given IMR, overall 30 unknown parameters (i.e., 10 locations, 10 stiffness coefficients and 10 damping ratios) should be optimized. A GA is used for the optimization process. First, optimization is conducted through minimizing $J_{\text{drift}}^s$. The optimization results indicate that when $J_{\text{drift}}^s$ is minimized, the 10 IFSs are automatically
located at the top-10 stories regardless of the value of IMR (25 or 50%). This observation is commensurate with the assumption made in Section 9.5.1 regarding the location of IFSs based on fundamental structural dynamics concepts. Figure 9-13 presents a comparison between the simple case of the MFI system with identical top-10 IFSs (designed in Section 9.6.1) and the more complex case of the GA-optimally designed, denoted as the gMFI system, for the case with an IMR of 25%; an evaluation of different OFs illustrates that the gMFI configuration outperforms the simple MFI configuration in terms of inter-story drift performance index by 7% but at the expense of increasing IFSs drift and acceleration performance indices by 30% and 42%, respectively. Hence, in terms of overall effectiveness, the gMFI is not deemed to be superior to the MFI with identical IFSs.

![Graphs showing performance comparison](image)

(a) identical IFSs are manually placed at top-10 stories  
(b) 10 dissimilar IFSs are optimally placed along the height

Figure 9-13 Design characteristics of two different MFI configurations with 10 IFSs and IMR = 25% optimized via minimizing the primary OF

As an alternative design philosophy for the optimal placement of the 10 IFSs, the sum of the RMS acceleration response of IFSs, \(\sum_{i} \text{RMS}([y_{i}^{\text{IFS}}])\), is selected as the OF to be minimized. For the optimization, two different scenarios are adopted. In the first scenario, the optimization is conducted conditioned on \(J_{\text{drift}} \leq 0.5\); the aim of this scheme is to minimize acceleration responses of IFSs while reducing the inter-story drift responses to a certain design target. In the second scenario, in addition to the constraint \(J_{\text{drift}} \leq 0.5\), the coefficient of variation (COV) of RMS acceleration responses of IFSs, \(\text{COV}([y_{i}^{\text{IFS}}])\), is limited to 0.10. The goal of this
design scheme is, in addition to providing the benefits of the first scheme, to reduce the variation in the acceleration responses of IFSs along the height, i.e., having a relatively uniform acceleration profile. Figures 9-14(a) and (b) present the optimization results for the two mentioned design schemes adopted for optimizing the IFSs locations in the MFI system with an IMR of 25%. As seen, for each scenario, the IFSs are located at different stories. In the first design scheme (Figure 9-14a) IFSs have approximately identical characteristics while in the second scheme (Figure 9-14b) characteristics of IFSs, especially the stiffness term, vary from story to story. In the first MFI scheme, \( \text{sum} \left[ \text{RMS}(\ddot{y}_j^{IFS})_c \right] \) and \( \text{COV} \left[ \text{RMS}(\ddot{y}_j^{IFS})_c \right] \) are 31.71 and 0.33, respectively while in the uncontrolled system these quantities for the corresponding floors are 103.81 and 0.54, respectively. In the second design scheme, \( \text{sum} \left[ \text{RMS}(\ddot{y}_j^{IFS})_c \right] \) and \( \text{COV} \left[ \text{RMS}(\ddot{y}_j^{IFS})_c \right] \) are 34.28 and 0.10, respectively while in the uncontrolled building these quantities for the corresponding floors are 110.22 and 0.34, respectively. These observations illustrate the flexibility of the MFI technique to improve different structural responses of interest while retaining the primary objective function, \( J_{\text{drift}}^s \), below predefined target values.

**Figure 9-14** Design characteristics of MFI configurations with 10 IFSs and IMR = 25% optimized through minimizing \( \text{sum} \left[ \text{RMS}(\ddot{y}_j^{IFS})_c \right] \) and adopting two different scenarios
9.7 Robust design of an MFI system

In Section 9.5 it was observed that iMFI configurations with intermediate IMRs could provide a more robust performance compared to those with low and high IMRs. Section 9.6 revealed that installing IFSs at the top-half stories provides a seismic response that is consistent with the one obtained when the system is equipped with IFSs at all stories. It was also illustrated that the MFI system with arbitrary IFSs optimized using a GA could surpass the iMFI configuration by only a few percentages. This latter observation implies that applying identical characteristics for IFSs, which is preferred when taking into account constructability, can lead to a near-optimal solution.

In this section, a robust design procedure is presented for the MFI system. An MFI configuration with top-10 IFSs and an IMR of 50% is designed to minimize the maximum of the SM considering ranges of epistemic uncertainty and aleatory variability while additional constraints are applied to limit the sum of RMS inter-story drift responses to a predefined value. For this configuration, the robust design is examined for systems with identical and dissimilar IFSs to investigate the possibility of using identical IFSs to provide a robust MFI control system.

9.7.1 Designing a robust MFI system less sensitive to epistemic uncertainties

First, an MFI system is designed that can exhibit a robust behavior against the epistemic uncertainties. In the context of the robustness, a system must be designed to perform effectively over a wide range of possible values of the relevant design parameters instead of designing it to perform optimally at a specific design point. To achieve this objective, in this chapter a GA is used to minimize the maximum value of the SM over the uncertainty range of interest (i.e. 50% deviation in the design parameters). In the GA process, IFSs can be assigned dissimilar characteristics (i.e., stiffness coefficients and damping ratios) at different stories. Preliminary simulation results illustrate that minimizing the maximum value of the SM can significantly deteriorate the MFI performance in terms of the inter-story drift response reduction compared to the optimally-designed MFI system. To overcome this deficiency, an additional constraint is incorporated into the robust design process as shown by Equation 9-14

\[
\frac{\sum_{i=1}^{n} \text{RMS}(x_i^s - x_{i-1})_{\text{with uncertainty}}}{\sum_{i=1}^{n} \text{RMS}(x_i^s - x_{i-1})_{\text{optimum}}} \leq 1.25 \tag{9.14}
\]
Applying the constraint given by Equation 9.14 guarantees that when minimizing the $SM$, the sum of RMS inter-story drift response of the superstructure does not exceed the sum of RMS inter-story drift response of the optimally-designed MFI system (i.e., the optimized configuration without taking into uncertainties) by a factor larger than the arbitrarily selected 25%. The robust design process is conducted for the selected MFI configuration assuming that only a single parameter from the vector of epistemic uncertainty can change at a time. $J_{\text{drift}}^S$ and $SM$ versus the uncertainty ratio for different robust configurations are plotted in Figures 9-15(a) and (b), respectively. According to Figures 9-15(a) and (b), if the system is robustly designed with respect to changes in the superstructure stiffness, $k^s$, the maximum values of $J_{\text{drift}}^S$ and $SM$ will be 0.32 and 1.68, respectively. For systems that are robustly designed with respect to variations in either of the three remaining design parameters (i.e., the superstructure’s damping ratio, $\xi^s$, the IFSs damping ratio, $\xi^{IFS}$, or the IFSs stiffness, $k^{IFS}$) the value of $J_{\text{drift}}^S$ ranges from 0.27 to 0.29. For these systems, the maximum value of $SM$ over the defined uncertainty range is limited to 1.05. These observations illustrate that it is possible to design a robust system with respect to deviations in $\xi^s$, $\xi^{IFS}$ and $k^{IFS}$. However, for deviations in $k^s$ there is no ideal solution, and the effect of uncertainties should be accounted for in the design process (e.g., the maximum value of the system response over the uncertainty range can be used as the design target).

![Figure 9-15](image)

*Figure 9-15 Performance of different robust gMFI systems with an IMRs of 50% and top-10 IFSs subjected to uncertainties in different design parameters*

### 9.7.2 Design of a robust MFI system less sensitive to the aleatory variabilities

In this section, the selected MFI configuration (i.e., the configuration with top-10 IFSs and an IMR of 50%) is first designed following four different scenarios. Then, the robustness and the
effectiveness of the MFI system designed based on each scenario against variations in the K-T model parameters are evaluated. The first two design schemes are the previously *optimized* MFI systems with identical (iMFI) and dissimilar IFSs (gMFI) in Sections 9.5.1 and 9.5.2. The second two schemes are designed based on *minimizing the maximum value of the* roof displacement transfer function, $H(s)$, and *minimizing the maximum value of SM*, over the vector of aleatory variabilities assuming a +/-50% deviation for each element of the vector; these two design schemes are denoted as *robust* systems.

Figure 9-16 illustrates $J_{\text{drift}}^3$ values for different MFT systems. As shown, the GA-optimally designed MFI system with dissimilar IFSs (gMFI) exhibits the minimum $J_{\text{drift}}^3$ value (i.e., $J_{\text{drift}}^3 = 0.19$), and therefore outperforms the other configurations in terms of the best performance. The robustly designed system based on the minimization of the SM with a $J_{\text{drift}}^3$ value of 0.23 (i.e., 21% increase with respect to the best system) is the worst-case scenario. The magnitude of the transfer function of the roof displacement for different design scenarios is depicted in Figure 9-17. As seen, the system designed based on the minimization of “the maximum value of the transfer function over the uncertainty range” exhibits a flat response that is weakly dependent on the variation in the predominant frequency of the ground excitation. Whereas, the frequency response of the other design schemes is strongly dependent on this deviation. For example, the iMFI system exhibits the least response ($|H(j\omega)| = 0.11$), at a normalized frequency near the resonance situation while with +36% variation in the normalized frequency, the response increases by a factor of 2.45 (i.e., increasing $|H(j\omega)|$ from 0.11 to 0.27). This observation illustrates that a compromise needs to be accepted between the effectiveness (i.e., the best possible performance) and the robustness (i.e., less deviations with respect to the response associated with the design point).

![Figure 9-16](image-url)

*Figure 9-16 The value of the primary OF for an MFI system with top-10 IFSs & an IMR of 50% designed based on different scenarios*
Figure 9-17 Roof displacement frequency response of an MFI system with top-10 IFSs and an IMR of 50% designed based on different scenarios

The SM values versus the K-T excitation parameters for different MFI systems are illustrated in Figures 9-18(a) to (d). According to Figures 9-18(a) and 9-18(b), the two optimally-designed systems exhibit seismic performances that are strongly dependent on the magnitude of the aleatory variabilities (the maximum value of the SM over the uncertainty ranges for these configurations is 1.80 and 1.84). For these configurations, the system response is sensitive to the value of $\omega^g$ as well as $\xi^g$. The seismic performance of the system that was robustly designed through minimizing SM, illustrated in Figure 9-18(d), is relatively stable with respect to the change in the aleatory variabilities; for this configuration, the maximum value of the SM is 1.31. As seen, this system is very robust with respect to changes in $\omega^g$. 
Figure 9-18 Performance sensitivity of the MFI configurations with top-10 IFSs and an IMR of 50% with respect to aleatory variabilities

The results of this section illustrate that in terms of the best performance, i.e., the minimum value of $J_{\text{drift}}^s$ at the design point, the seismic response improvement of the MFI system equipped with identical IFSs (i.e., the iMFI configuration) can be approximately the same as a GA-optimized MFI configuration with possible dissimilar IFSs, however, an iMFI configuration fails to provide a robust system when accounting for aleatory variabilities.

9.8 Conclusions

A multi-floor isolation (MFI) system, which was proposed by the authors in a previous study, is evaluated herein in terms of its effectiveness and robustness to mitigate earthquake-induced inter-story drift demands in buildings. The proposed MFI system is based on the isolation of different portions of floor masses at various locations throughout the height of the building such that the isolated masses serve as inherent vibration suppressors. By mitigating seismic-induced inter-story drift demands as the primary performance objective, the MFI system has proved to be useful to
mitigate seismic-induced floor acceleration demands, as well drift and acceleration demands on the isolated portions of a floor mass. Various MFI configurations are designed and evaluated by modifying the number of isolated floor subsystems (IFSs) and the isolated mass ratio (IMR). In this evaluation, effectiveness relates to the ability of the system to significantly mitigate inter-story drift demands, and hence, improve seismic performance with respect to the uncontrolled case. Robustness refers to the ability of the system to provide a stable performance in the presence of epistemic uncertainties and aleatory variabilities.

The seismic effectiveness and robustness of the optimally-designed MFI configurations is evaluated using a 20-story, shear-type planar frame structure. Sensitivity analyses are conducted by incorporating variations in structural design parameters as well as in ground motion characteristics. The first step in the sensitivity analyses is to design MFI configurations with identical IFSs located at all story levels – this configuration is denoted as iMFI. The iMFI configuration is optimized deterministically by minimizing an objective function, $J_{\text{drift}}^s$, that is the sum of the root-mean-square inter-story drift response of the superstructure in the controlled system normalized to that of the corresponding response of the uncontrolled system under a stochastic Kanai-Tajimi ground excitation (in this manuscript the term “superstructure” refers to the non-isolated part of the MFI system including the structural frame). The parameters varied during the sensitivity analyses are: the superstructure stiffness coefficient and damping ratio; the IFSs stiffness coefficient and damping ratio; the predominant frequency and damping ratio of the Kanai-Tajimi excitation. A relatively large variation of +/-50% is assumed for these parameters.

Results demonstrate that the effectiveness (i.e., the minimum value for the $J_{\text{drift}}^s$ at the design point) of the MFI system increases with an increase in the IMR and tends to saturate at IMR values approximately greater than 50%. As an initial step, an evaluation of the magnitude of the frequency response of the superstructure displacement at the roof level is performed. This evaluation suggests that iMFI configurations with low IMRs (e.g., 5% and 10%) although effective in reducing the superstructure inter-story drift response at the resonance condition, do not provide an adequate effectiveness when exposed to relatively small deviations (i.e., 10%) in the predominant frequency of the ground excitation. In terms of robustness, configurations with intermediate IMRs (e.g., 25% and 50%) provide significant seismic performance improvement with respect to the uncontrolled case for a wide range of the excitation predominant frequencies.
To further investigate the robustness of the optimally-designed iMFI configurations, two additional evaluations are performed for the assumed deviations in the various structural design and ground motion parameters: (i) $J_{\text{drift}}^s$ values are quantified to assess changes in effectiveness; (ii) a sensitivity measure (SM) is proposed and used to quantify the sensitivity of the inter-story drift seismic responses of an optimally designed system that incorporates parameter uncertainties with respect to the seismic responses obtained from the optimization process in the absence of uncertainties. Simulation results illustrate that configurations with low IMRs although effective at the design point (e.g., using a 5% IMR can reduce $J_{\text{drift}}^s$ by 52%), can exhibit a decrease in effectiveness in the presence of changes in the design parameters. Overall, iMFI configurations with intermediate IMRs (e.g., 50%) show a more stable/robust performance. For example, for a simultaneous deviation of 50% in the superstructure stiffness coefficient and damping ratio, as well as in the IFSs stiffness and damping ratio, the worst $J_{\text{drift}}^s$ value for the iMFI systems considered with IMRs of 5%, 50% and 90% is 0.99, 0.32, and 0.40, respectively, meaning that the configuration with an IMR of 50% exhibits the most robust performance. These results are consistent with those obtained from the evaluation of the roof displacement frequency response.

Additional studies are conducted to evaluate the optimum number and placement of IFSs along the height of the structure. Based on the optimization criteria adopted in this study, for a given number of IFSs, the IFSs can be simply located at the top stories. It is also shown that IFSs can be distributed over the height of the building in such a way that they experience uniform acceleration responses.

Given the aforementioned observations related to the optimum placement of IFSs along the height and values of IMR that result in more effective and robust designs, a Genetic Algorithm is used to design a robust system for an MFI configuration with top-10 IFSs and an IMR of 50% in which the IFSs can have different properties along the height. An assessment of the behavior of this system with respect to the response of an optimally-designed system with identical IFSs illustrates that in terms of effectiveness, a near-optimal solution can be achieved by using identical IFSs; however, such a system is unable to provide a robust design.

The study conducted in this paper illustrates that an MFI system with intermediate IMRs (e.g., 50%) and a few IFSs at upper stories (e.g., top-half), developed based on the concepts of tuned-mass-damper and base-isolation systems, can provide an effective and robust system to mitigate
seismic demands. The main advantage of this system over conventional passive control strategies relies on its ability to behave almost equally effectively over a wide range of inherent uncertainties without compromising efficiency significantly. Future research in this area should focus on the aesthetical aspects and possible environmental benefits of the proposed MFI system. For example, if IFSs are capable of accommodating large movements in the horizontal directions, the proposed technique can optimally use the sunlight during the daytime and save the building energy consumption. This latter characteristic can justify the initial cost of this relatively new control system, given that the structural control improvement component of the system is achieved only during rare ground motions in a building’s lifetime.

9.9 Acknowledgment

The study conducted in this paper was partially supported by a Summer Fellowship Award provided by the Department of Civil and Environmental Engineering, University of New Hampshire to the first author. This support is gratefully appreciated.

Nomenclatures

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
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<tbody>
<tr>
<td>BI</td>
<td>Base isolation</td>
</tr>
<tr>
<td>$c_i^{IFS}$</td>
<td>Damping coefficient of IFS at $i$-th story</td>
</tr>
<tr>
<td>$c_i$</td>
<td>Global damping coefficient of superstructure at $i$-th story</td>
</tr>
<tr>
<td>$C$</td>
<td>IFSs damping coefficient matrix of the MFI system</td>
</tr>
<tr>
<td>$C_i^{IFS}$</td>
<td>IFSs damping coefficient matrix</td>
</tr>
<tr>
<td>$C'$</td>
<td>Superstructure damping coefficient matrix</td>
</tr>
<tr>
<td>COV</td>
<td>Coefficient of variation</td>
</tr>
<tr>
<td>$g_{MFI}$</td>
<td>MFI with generic (dissimilar) IFSs at different stories</td>
</tr>
<tr>
<td>GA</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>$H(j\omega)$</td>
<td>Roof displacement frequency response</td>
</tr>
<tr>
<td>iMFI</td>
<td>MFI with identical IFSs at different stories</td>
</tr>
<tr>
<td>$IMR_i$</td>
<td>Isolated mass ratio (ratio of IFS mass to entire story mass) at $i$-th level.</td>
</tr>
<tr>
<td>IFS</td>
<td>Isolated floor subsystem</td>
</tr>
<tr>
<td>$I$</td>
<td>Identity matrix</td>
</tr>
<tr>
<td>$J_{drift}^s$</td>
<td>Structural frame inter-story drift performance index</td>
</tr>
<tr>
<td>$J_{acc.}^w$</td>
<td>Weighted absolute acceleration performance index</td>
</tr>
<tr>
<td>$J_{drift}^{IFS}$</td>
<td>IFSs drift performance index</td>
</tr>
<tr>
<td>$m_i^s$</td>
<td>Mass of the superstructure at $i$-th story</td>
</tr>
<tr>
<td>$M^{IFS}$</td>
<td>Superstructure mass matrix</td>
</tr>
<tr>
<td>$M'$</td>
<td>IFSs mass matrix</td>
</tr>
<tr>
<td>MFI</td>
<td>Multi-floor isolation</td>
</tr>
<tr>
<td>OF</td>
<td>Objective function</td>
</tr>
<tr>
<td>$r$</td>
<td>Influence vector</td>
</tr>
<tr>
<td>RMS</td>
<td>Root mean square</td>
</tr>
<tr>
<td>$s = j\omega$</td>
<td>A complex number frequency parameter</td>
</tr>
<tr>
<td>$SM$</td>
<td>Sensitivity measure</td>
</tr>
<tr>
<td>$T^{IFS}$</td>
<td>IFS fundamental period</td>
</tr>
<tr>
<td>$T_{Equiv.}^{IFS}$</td>
<td>Equivalent elastic fundamental period of IFS</td>
</tr>
<tr>
<td>$x_i^{IFS}(t)$</td>
<td>IFS displacement relative to the ground at $i$-th story</td>
</tr>
<tr>
<td>$x_i'(t)$</td>
<td>Superstructure displacement relative to the ground at $i$-th story</td>
</tr>
<tr>
<td>$x_g(t)$</td>
<td>Ground displacement</td>
</tr>
<tr>
<td>$\ddot{x}_g(t)$</td>
<td>Ground acceleration</td>
</tr>
<tr>
<td>$y_i^{IFS}(t)$</td>
<td>IFS absolute acceleration at $l$-th story</td>
</tr>
<tr>
<td>$y_i'(t)$</td>
<td>Superstructure absolute acceleration at $l$-th story</td>
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Chapter 10

Application of Control Systems for Protecting Nonstructural Components
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Chapter 10

Application of Control Systems for Protecting Nonstructural Components

10.1 Abstract

In most previous research works dealing with seismic control systems, referenced in Chapters 7-9 (e.g., Tsai and Lin 1993; Hoang et al. 2008; Marano and Greco 2008; Johnson et al. 2015; Di Matteo et al. 2017), the attention was restricted to fundamental-mode dominated responses such as inter-story drift and roof displacement responses. In Chapters 7-9 of this dissertation, the performance of different seismic control systems for mitigating fundamental-mode dominated responses was evaluated. The present chapter investigates the effectiveness of two different control systems for reducing the floor spectral acceleration responses in the high-frequency region, denoted as the higher-mode dominated response herein. For the study conducted in this chapter, a 12-story test-bed linear-elastic shear building is used. A Rayleigh damping approach based on initial stiffness is used to approximate the uncontrolled building viscous damping mechanism, where a 5% viscous damping ratio is specified to the first two modes of the building. The lateral stiffness and masses of the stories are identical along the building height. Based on the modal analysis results, the first three modal frequencies of the building are 5.2 and 15.6 and 25.8 rad/s (i.e., modal periods are 1.2, 0.4 and 0.24 s). A partial mass isolation (PMI) system and an equivalent tuned-mass-damper (TMD) system are designed to mitigate nonstructural components (NSCs) seismic responses in the high-frequency region of floor spectra. The robustness and effectiveness of the two passive control systems are evaluated.

Keywords: Higher-mode dominated responses; Nonstructural components; TMD; Partial mass isolation system; Uncertainty; Robustness.
10.2 The effectiveness and robustness of an optimized TMD system for reducing floor spectral acceleration responses

Based on the results of testing many NSCs by OSHPD (presented in NIST GCR 17-917-44 2017), and also some recent experiments (e.g., Watkins et al. 2009; Watkins 2011; Archila et al. 2012; Astroza et al. 2015), the fundamental frequency of most of the tested equipment falls above $4\pi$ rad/s (see Chapter 5 for a detailed discussion). Based on this observation, in this chapter, the range $(4-200)$ rad/s (i.e., 0.01-0.5 s) is considered as the operational (or practical) range of NSCs. Within this frequency range, the maximum value of the roof floor spectrum of the 12-story building occurs in the vicinity of the building second mode.

An elastic single-degree-of-freedom (SDOF) system, which represents a NSC, with a mass of 0.1\% of the building mass and a viscous damping ratio of 2\% is attached to the roof of the building in the numerical model using a spring-dashpot element. Therefore, the dynamic interaction between the building and NSC is explicitly taken into account. First, it is assumed that the NSC is tuned to the second mode of the building (i.e., $\omega_{\text{NSC}} = 15.6$ rad/s). This configuration is denoted as the uncontrolled primary-secondary system. Then, a single TMD, which is tuned to the second mode of the building (and also the period of the NSC), is added to this system to protect the NSC. The TMD’s mass is 5\% of the building mass, and its viscous damping ratio is 2\% (simulation results suggest that a low-damping TMD is much more effective in mitigating the roof spectral values in the vicinity of the second mode of the building). The magnitude of the transfer function of the absolute acceleration response for the uncontrolled (UNC) and the TMD-controlled (C) primary-secondary systems are calculated and illustrated in Figure 10-1.

As seen in Figure 10-1, the optimal TMD can significantly reduce the maximum value of the transfer function of the roof and also that of the NSC at excitation frequencies in the vicinity of the second mode of the building (i.e., at the tuning condition). This reduction for the NSC is from 68.2 to 4.0. However, the TMD’s operating range is relatively narrow (i.e., from 14.0 to 17.0 rad/s) meaning that such TMD is not effective in reducing the NSC demands for excitation frequencies beyond the tuning. As seen, the optimal TMD can even slightly amplify the NSC responses at some frequencies (i.e., in the vicinity of the two spikes observed in the transfer function of NSC). In other words, an optimal TMD is effective in reducing NSC response at a
specific design point (i.e., tuning frequency), however, is not robust against variations in the base-excitation dominant frequency.

Figure 10-1 Magnitude of the transfer function of the absolute lateral acceleration response of the roof and NSC for the uncontrolled (UNC) and the TMD-controlled (C) primary-secondary systems

To further investigate the TMD’s robustness for reducing the NSC acceleration demands, a roof pseudo floor response spectrum (FRS) is developed. For developing this pseudo floor spectrum, the NSC frequency, $\omega_{NSC}$, is varied from 10 to 30 rad/s. For each $\omega_{NSC}$, the pseudo floor response spectrum ordinate (the spectral acceleration ordinate at that specific frequency) is defined as the maximum value of the transfer function over the frequency range of excitation. The pseudo FRS for the uncontrolled building and controlled one by the TMD (which is tuned to the second mode of the building) are depicted in Figure 10-2. An evaluation of this figure shows that the TMD can reduce FRS values at the NSC frequencies close to the TMD frequency but not for other frequencies. In other words, the operating range of the TMD is relatively narrow. If the maximum value of the FRS over the entire frequency range of interest is considered in design, which is usually the case in practice, the TMD’s effectiveness is significantly de-emphasized. As seen in Figure 10-2, the maximum value of the FRS for the uncontrolled and controlled cases is 68.2 and 61.3 respectively, implying a slight improvement only.
10.3 Designing of a robust partial mass isolation (PMI) system for improving the seismic response of nonstructural components

The PMI technique was introduced in Chapter 9. This technique through isolating a portion of story masses can provide a building with multiple vibration suppressors without the installation of any additional mass in the building, which is common in traditional TMD systems. In Chapters 7-9, the effectiveness and robustness of different PMI configurations for mitigating fundamental-mode dominate response were addressed. This section uses a PMI system to protect high-frequency NSCs in buildings. It is assumed that isolated components (ICs) with an identical isolated mass ratio (IMR) of 5% are implemented at all stories. This configuration provides an overall isolated mass equal to the TMD’s mass studied in Section 10.2. Multiple mass dampers provided by isolating masses at different stories facilitate the robust design of the PMI system. Relatively light sections of buildings such as architectural double skin facades are potential candidates to be isolated from the superstructure to provide the required 5% IMR at all stories for this PMI system. Given that the isolated parts will play the role of sacrificial TMDs tuned to the higher modes of the building, using accelerative-sensitive components as the isolated parts is not recommended.

Designing a robust PMI system is performed through minimizing the maximum value of the pseudo FRS over the NSC frequency range of interest. A total of 24 parameters (stiffness coefficient and damping ratio of ICs at each story) should be optimized. For the optimization process a Genetic Algorithm approach is used. For comparison purposes, a similar strategy is
adopted to examine the possibility of designing a robust TMD with a mass ratio of 5%. For the TMD, the damper’s stiffness coefficient and damping ratio are the two parameters to be optimized.

Figure 10-3 illustrates the roof FRS for the PMI and TMD systems designed following the abovementioned robust design approach.

![Graph illustrating roof pseudo floor response spectra for PMI and TMD systems](image)

*Figure 10-3 Roof pseudo floor response spectra for the PMI and TMD systems designed to be robust against variation in NSC frequency*

As seen in Figure 10-3, the pseudo FRS curve for the PMI system is relatively flat meaning that for this case, the response of roof-mounted NSCs is less influenced by the NSCs fundamental frequency value. In other words, the system is robust with respect to changes in the NSC fundamental frequency. However, the single TMD is not able to provide the required level of robustness. According to Figure 10-3, for the TMD case at a frequency near the second-mode of the primary building, a significant spike is observed in the pseudo FRS. The maximum value of the pseudo FRS over the entire frequency range for the robust PMI system is limited to 31.5, which is significantly smaller than that of the building controlled by the TMD, which is 57.7.

Another significant disadvantage of a single TMD is the performance sensitivity to the tuning frequency such that a detuning can significantly degrade its seismic performance. This is particularly important because of the uncertainties associated in the estimation of the primary building modal frequencies, and also because of the frequency shift (due to building inelasticity) experienced by the building when exposed to severe earthquake ground motions. A PMI system can partially mitigate this drawback through tuning the ICs to different frequencies. In the following paragraph the design process of such a PMI system is described.

10-5
The PMI system is optimized once more to minimize the maximum value of the roof pseudo FRS when the superstructure stiffness, $k_s$, is varied by ±50%. The $k_s$ value is varied from $0.5k_d^d$ to $1.5k_d^d$ with an increment of $0.25k_d^d$ (i.e., a total of five $k_s$ values are considered), where $k_d^d$ is the superstructure design (mean) stiffness or the stiffness associated with the fundamental frequency of 5.2 rad/s. Varying $k_s$ can simulate the uncertainties in the estimation of superstructure frequencies and also the frequency shift present when the building experiences inelastic actions during a severe ground motion. The PMI system parameters are optimized following the aforementioned design strategy using a Genetic Algorithm. Figure 10-4 illustrates roof pseudo FRS when different values of $k_s$ are assumed. Over the range of NSC period considered, the maximum value of the pseudo FRS corresponding to different values of $k_s$ is rather consistent (its magnitude is approximately 50 for all cases). For the evaluation purposes, similar results for the uncontrolled building are also shown. An evaluation of the results shows that when the envelop FRS of all $k_s$ values is considered, the PMI system can reduce the maximum value of the transfer function over the NSC frequency range of interest (i.e., 10-30 rad/s) by 26%.

Figure 10-4 Roof pseudo spectra for the PMI-controlled and uncontrolled buildings exposed to variation in NSC frequency and variation in the superstructure stiffness, $k_s$

($k_s$ is varied from 0.25 to 1.5 design $k_s$, and each curve corresponds to a $k_s$ value)

10.4 Conclusions and Future Works

This chapter evaluates the effectiveness and robustness of a conventional tuned-mass-damper (TMD) and the proposed partial mass isolation (PMI) system for reducing the seismic response of a single-degree-of-freedom NSC attached to the roof floor of a 12-story building. The response
quantity to be evaluated is the magnitude of the transfer function of the absolute acceleration response of the NSC. First, it is assumed that the NSC is tuned to the building second-mode. It is shown that an optimal TMD tuned to the second mode of the building can significantly reduce the NSC response when the base-excitation dominant frequency is in the vicinity of the building second mode. However, the effectiveness of such a TMD is rapidly deteriorated by variations in the base-excitation frequency. In other words, this optimal TMD is effective for a specific design point (i.e., tuning base-excitation) but is not robust against variations in the dominant frequency of the building and ground excitation. As another important disadvantage, such a TMD is effective only for NSC frequencies in the vicinity of the building second mode but not for NSC beyond this frequency range. A Genetic Algorithm is used to design a robust PMI system for protecting nonstructural components (NSCs) with periods in the range (4-200) \( \pi \) rad/s, wherein most typical NSCs are situated. An evaluation of the pseudo roof floor spectrum of the building equipped with the robust PMI system illustrates that this system is equally effective for all NSCs in the frequency range of interest.

In this chapter, the frequency response of the system is selected as the objective function to be optimized. Future works should consider the seismic responses of buildings under ground motion excitations. In this study, the building is assumed to respond elastically, and for simulating the inelastic behavior of the building the lateral story stiffness is simply varied within a given range. The use of the system for protecting NSCs in inelastic building models should be investigated as well. In such studies, buildings with different heights (i.e., fundamental periods) should be used.

10.5 References


Chapter 11

Summary and Conclusions
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Chapter 11

Summary and Conclusions

11.1 Introduction

Post-earthquake reconnaissance following past earthquakes in the US and other seismic-prone countries illustrates that the majority of building losses (injury, death, dollar loss and downtime) resulted from the direct or indirect consequences of damage to nonstructural components (NSCs) and building contents. The reconnaissance reports illustrate that NSCs damages can severely compromise a building functionality, even if the building does not suffer significant structural damages. Based on the controlling failure modes, NSCs can be classified as either primarily displacement/deformation-sensitive or acceleration-sensitive components. This study focuses on acceleration-sensitive NSCs, and more specifically horizontal seismic demand on acceleration-sensitive NSCs that can be reasonably modeled as single-degree-of-freedom (SDOF) systems.

In the first part of this dissertation, acceleration responses of a wide variety of instrumented buildings and code-based designed building models are evaluated to: (i) identify the most salient limitations of using simplified numerical models for the quantification of seismic demands on NSCs, (ii) quantify the most influential parameters that control NSCs responses, (iii) evaluate the design equivalent static equations of ASCE 7-16 for acceleration-sensitive NSCs, (iv) assess alternative design equivalent static equations proposed as part of a recent project sponsored by the Applied Technology Council (Project ATC-120), and (v) develop modifications and improvements to the proposed ATC-120 equations.

In the second part of this dissertation, modern seismic protection techniques are studied that can decrease seismic input demands to a building, as opposed to modifying the seismic resistance of a building, which is the approach taken in current US design seismic provisions. The conventional base-isolation and tuned-mass-damper concepts are utilized to develop an innovative
seismic control system (i.e., the partial mass isolation, PMI system) that can reliably enhance the seismic performance of the structural elements and NSCs so that the building can be occupied and remain functional immediately after a design earthquake. The practicality, limitations, effectiveness, and robustness of the PMI system for protecting the structural and nonstructural components of building structures are discussed and evaluated.

11.2 Part I: Improved seismic design of acceleration-sensitive NSCs

11.2.1 Summary

Numerous studies have been conducted in the past three decades on the quantification of seismic demands on acceleration-sensitive NSCs. However, many of these works have assumed a linear-elastic behavior for the NSCs and/or their supporting buildings. The studies that have focused on the inelastic supporting buildings have been mostly based on simplified SDOF models or multistory generic frames with the assumption of an elastic NSC behavior. These models, while providing valuable insight into understanding the influential parameters on NSCs seismic demands and behavioral patterns, may not adequately represent the characteristics present in the responses of actual buildings. Therefore, there has been a continuous skepticism on the results of these simplified models in the practicing engineering communities. In the current ASCE 7-16 equivalent static equations for designing NSCs, the effect of NSC inelasticity [and also NSC viscous damping ratio] is taken into account by a response modification factor, $R_p$, which is based on engineering judgment rather than experimental or numerical studies. There has been no clear understanding of the consequences of designing NSCs using these $R_p$ factors (these consequences include NSC inelastic displacement and ductility demands). Hence, there was a need to quantify these effects when using the prescribed $R_p$ values, which are as large of 6 and 12 for some NSCs.

In the first part of this dissertation, which includes Chapters 2 through 5, recorded floor acceleration responses of a wide variety of instrumented buildings in past earthquakes and simulated seismic responses of several numerical building models are evaluated to identify the most important limitations of using the aforementioned simplified numerical models.

Floor response spectra of a total of 118 instrumented building-directions in California are evaluated. The selected buildings encompass a wide range of supporting building characteristics
(e.g., lateral-load resisting system and modal periods), and recorded ground motion characteristics (e.g., intensity levels and frequency contents). The primary objective of this evaluation is to identify and quantify the most important parameters that can significantly influence the magnitude of NSCs acceleration demands but are not explicitly considered in the simplified ASCE 7-16 $F_p$ equivalent static equation (Eq. 13.3-1) and are also difficult to capture using numerical models. Additionally, an objective is to evaluate and validate observations from numerical studies included in the literature regarding shortcomings associated with the ASCE 7-16 $F_p$ equation.

The numerical building models used as part of this study are code-based designed (archetype) special steel moment resisting frame (SMRF) and special reinforced concrete shear wall (RCSW) buildings with different heights varying from one- to 12-stories. Besides the baseline version of the archetype buildings, an overdesigned version, which may be in a better agreement with daily design, is also evaluated. Input ground motion excitations used for numerical analyses are a set of 20 spectrum-compatible records and a set of 44 magnitude-scaled far-filed records. Nonlinear response history analyses are conducted on the archetype buildings exposed to various ground motion intensity levels varying from 0.25 DE to 2.0 DE (design earthquake).

Most research works dealing with the quantification of seismic demands on NSCs are based on two basic premises that the viscous damping ratio of NSCs is 5% and they respond elastically. Recently in the ATC-120 Project, attempts were made to develop improved NSCs equivalent static design equations. In this project, discussions were ensued on whether NSCs (specifically electrical and mechanical equipment) may exhibit viscous damping ratios well below the nominal 5%. In addition, the inelastic design of NSCs, which was already taken into consideration in the ASCE 7-16 provisions through the empirical $R_p$ factor, was revised based on numerical analyses. As part of this dissertation, first, the influence of NSCs inelastic behavior on their seismic demands is investigated. Inelastic floor spectra for different floor levels of several archetype buildings are developed assuming different component viscous damping ratios and target ductility values. Seismic force and displacement demands on NSCs are evaluated following four different primary-secondary system scenarios namely (i) elastic NSC–elastic building; (ii) elastic NSC–inelastic building; (iii) inelastic NSC–elastic building; (iv) inelastic NSC–inelastic building. The first part of this dissertation ends with an evaluation of the equivalent static equation by ATC-120 Project. Additional potential modifications and improvements to this equation are proposed.
11.2.2 Conclusions of the first part of this dissertation

The most important conclusions of the first part of this dissertation are presented next. Some of these results were previously obtained by other researchers primarily using simplified numerical building models (i.e., generic frames) and are available in literature. This dissertation corroborates these results using the responses of instrumented and code-based designed buildings, which are believed to be more reliable. Furthermore, given that one primarily objective of this dissertation is to evaluate and improve the current equations for designing acceleration-sensitive NSCs, there is a need to quantify the influential parameters on NSC seismic demands using the responses of code-based design buildings. Therefore, some of the previous results should be re-produced.

11.2.2.1 The most salient conclusions of studying instrumented buildings are summarized below:

1) The conducted evaluation on the floor spectra of the instrumented buildings reveals significant behaviors that are not consistent with the results obtained in the past using equivalent simplified 2D numerical building models. In many studied instrumented buildings, the shape and magnitude of the floor spectra significantly depart from those obtained based on numerical models of a given type of a regular lateral-load resisting system. The primary reasons for these inconsistencies are the in-plane flexibility of the floor system diaphragm, torsional responses of the supporting building, and the supporting building’s vertical mass or stiffness irregularity:

1-1) The in-plane diaphragm flexibility and torsional responses of the supporting buildings can significantly alter the shape and magnitude of the floor spectra with respect to those obtained from equivalent numerical building models that incorporate rigid diaphragms and symmetry in strength and stiffness of the lateral-load resisting system elements. In buildings with perimeter lateral-load resisting elements, the in-plane diaphragm flexibility can amplify peak floor acceleration (PFA) and peak component acceleration (PCA) demands at the floor mid-span with respect to values at the floor edges. For buildings with a core lateral-load resisting system a reverse trend may be observed. This amplification in single-story buildings with plywood diaphragms can be as large as 5.0, whereas for the studied multistory buildings it is bounded to 2.0. Torsional responses of the supporting building, even in nominally regular buildings, can increase the floor acceleration responses as well as acceleration demands on
NSCs that are located in the floor periphery. This amplification is bounded to 1.53 for the studied instrumented buildings.

1-2) Although torsional amplification is highlighted more in torsionally irregular structures, for several buildings that are nominally symmetric in plan and layout of seismic force resisting elements, torsional responses are identified. Torsional effects in the torsionally regular buildings could be attributed to a variety of sources such as local yielding or the asymmetric yielding of the building, accidental torsional moments caused by eccentricities between the centers of rigidity and mass that exist because of uncertainties in the distribution of the mass and stiffness of the buildings, as well as the torsional components of earthquake ground motions. Results of the conducted evaluation illustrate that in most cases, amplification due to the torsional effects and in-plane diaphragm flexibility do not occur simultaneously. In other words, the in-plane diaphragm flexibility can mitigate torsional responses at the expense of amplified responses at the middle/edge of floors.

1-3) Relatively low or large normalized spectral acceleration responses, $F_S a/P_G A$, are observed at the vicinity of the modal periods of several instrumented buildings that are not consistent with the trend observed in the responses of typical numerical building models and other studied instrumented buildings. In some of these cases, the magnitude of $F_S a/P_G A$ at tuning situation is larger or smaller than the corresponding mean value of $F_S a/P_G A$ of all instrumented building-directions by factors larger than 10. The relatively low normalized spectral ordinates at tuning situations are due to the low energy content of the recorded ground motion at those specific NSC periods. The significantly large normalized responses generally occur in instrumented buildings that experienced low-intensity ground motions with a consequent elastic or near-elastic behavior. However, it is observed that additional plan or vertical mass and stiffness irregularities might have further amplified these large normalized responses. For example, in some of these instrumented buildings large normalized PCA responses are observed at top floor levels with a significantly smaller mass (e.g., by a factor of 10) than the typical floors mass. These large responses are most likely because of tuning of the top story to the modal periods of the rest of the building (the building without this story).

2) It is postulated in this dissertation that buildings with significant in-plane diaphragm flexibility, significant torsional responses, or the mentioned special behaviors should not be used
to establish the basis for NSCs design equations. The design equations— as they apply to new buildings – should be based on the responses of regular, modern code-compliant building designs. Then, correction factors could be incorporated into the basic design equations to account for the effects of the mentioned inconsistencies.

3) The roof floor motions obtained from regular instrumented buildings that are not greatly influenced by the abovementioned causes are evaluated to identify and quantify the effect of the supporting building lateral-load resisting system and modal periods on the floor spectra. This evaluation shows a significant difference between the shape and magnitude of the floor spectra for shear-dominated systems (e.g., MRFs) and flexural-dominated systems (e.g., tall SWs), as well as for short-period and long-period buildings. These observations illustrate that the floor spectra ordinates strongly depend on the type of lateral-load resisting system, and modal periods of the supporting building, which are not explicitly taken into account in the current ASCE 7-16 equivalent static approach. This observation suggests that one may need to consider using the dynamic analysis methods provided in Section 13.3.1.4 of the ASCE 7-16, which explicitly incorporate the characteristics of the supporting building.

4) In buildings with below-grade stories (basements), the seismic base location influences the estimation of acceleration demands on NSCs provided by the ASCE 7-16 Eq. 13.3-1. The acceleration responses of two parallel ground motion sensors, one installed at the ground floor level and the other one at the foundation level, in buildings with below-grade stories can be used to evaluate the seismic base location. An evaluation of the responses of several instrumented buildings reveals significant amplifications in ground floor acceleration responses with respect to the floor levels below ground (in some cases amplification factors up to 2.0 are observed even with the presence of perimeter basement concrete walls). This observation implies that the conditions to establish the seismic base at the ground level are not satisfied. For buildings with basements, a soil-structure-interaction analysis incorporating the characteristics of the adjacent soil is required to estimate the location of the seismic base. In the absence of such analyses, if doubt exists as where to locate the seismic base, strong consideration should be given to establishing the seismic base at the foundation level when using the ASCE 7-16 Eq. 13.3-1.

5) It is shown that many of the instrumented buildings experienced relatively small ground motion intensities. For example, in 85% of the cases the recorded PGA was lower than
0.5PGA_{\text{Design}}, where PGA_{\text{Design}} is 0.4S_{DS} for each building site. Hence, it is reasonable to infer that most of these buildings behaved in their linear-elastic range. A consequence of this linear behavior is the presence of relatively large normalized peak floor acceleration (PFA/PGA) and normalized peak component acceleration (PCA/PGA) responses; for example, at the roof level of the studied instrumented buildings, the PFA/PGA and PCA/PGA responses as large as 5.5 and 36.0 are observed, respectively. If these normalized acceleration responses are selected to evaluate the ASCE 7-16 equation in its normalized format (i.e., Equation 4-2a), they can exceed the design PFA/PGA and PCA/PGA by factors up to 1.8 and 9.0, respectively. The effect of ground motion intensity on floor response spectra of the instrumented buildings is investigated revealing that as the ground motion intensity increases, normalized acceleration responses significantly decrease; for example, when only the instrumented buildings with a PGA/PGA_{\text{design}} \geq 0.75 are considered, PFA/PGA and PCA/PGA responses at the roof level are limited to 2.0 and 7.5, respectively, which are values significantly lower that the previously discussed 5.5 and 36.0 normalized responses. This latter observation demonstrates the drawbacks of an evaluation of the ASCE 7-16 F_p equation in its normalized format based on linear-elastic models or instrumented buildings that have experienced relatively small intensity ground motions.

6) The results of the conducted evaluation on the instrumented buildings responses illustrate significant shortcomings associated with the two components of the ASCE 7-16 equation. It reveals that, unlike the current ASCE 7-16 approach, the component amplification factor (PCA/PFA) is a function of the ratio of NSC period to the modal periods of the supporting building; ground motion intensity level; and the NSC location along the building height. It also illustrates that the ASCE 7-16 period threshold used to determine the component amplification factor warrants modifications.

11.2.2.2 The most salient conclusions of the studying archetype buildings are summarized below:

1) As an initial step, challenges of using spectrum-compatible ground motions in linear and nonlinear responses history analyses are addressed. It is shown that using the spectral matching technique may not completely remove the record-to-record variability, especially in higher-mode dominated responses, such that still a relatively large number of spectrum-compatible records should be used and mean responses should be evaluated. Therefore, this approach may not
significantly decrease the number of required records, and consequently, the time involved for response history analyses. However, when using a set of historical records that are amplitude-scaled, individual spectra can significantly exceed the target spectrum especially at short periods, which can tend to overstate the importance of higher mode responses.

2) The evaluation of ASCE 7-16 equations is conducted using the responses of the archetype buildings at the DE level and assuming an elastic NSC having a 5% viscous damping ratio. The peak values of floor spectra at the higher-mode and first-mode regions are used for the evaluation of the design PCA. The most salient results of this evaluation, which corroborates results of previous studies, are summarized below:

2-1) The effect of supporting building inelastic behavior, lateral-load resisting system, and fundamental period should be explicitly incorporated into NSCs design equations, as they are significantly influential. Unlike the ASCE 7-16 provisions, the value of the component amplification factor highly depends on the relative height of the floor of attachment of NSCs.

2-2) Simulation results of the archetype buildings at the DE level, illustrate the tendency of the ASCE 7-16 in-structure amplification factor, \([1 + 2(z/h)]\), to significantly overestimate demands at all floor levels and the ASCE 7-16 component amplification factor, \(a_p = 2 \frac{1}{2}\), to in many cases underestimate the calculated component amplification factors. Furthermore, the product of these two amplification factors (that represents the normalized PCA response) in some floor levels of the archetype buildings exceeds the ASCE 7-16 equation by a factor up to 1.80.

3) Supporting building inelasticity in most cases has the effect of reducing seismic force demands on NSCs, especially for tuned NSCs. This beneficial effect, which depends on the floor relative height and NSC characteristics, is maximum for a roof-mounted low-damping elastic NSC tuned to the building fundamental mode. For some non-tuning NSCs, the building inelasticity can increase demands on NSCs. This adverse effect is more highlighted for low-damping NSCs and for building with a weak-story mechanism (localized plasticity).

4) The value of the supporting building viscous damping is an influential parameter on the seismic responses of tuned NSCs that are mounted on elastic buildings. However, for NSCs mounted on inelastic buildings, this effect is insignificant at all NSC periods, except for those in
the vicinity of the building higher modes. Even for these latter NSCs, the effect of supporting building viscous damping is de-emphasized when the supporting building responds inelastically.

5) The results show that even a mild level of inelasticity of a tuned NSC, especially to the building fundamental mode, can significantly decrease its force and displacement seismic demands (i.e., floor spectral acceleration and displacement values). Because of the semi-harmonic nature of floor motions, these reductions are more significant than reductions observed in typical inelastic ground spectra. For non-tuning NSCs with periods in between the modal periods of the supporting building, component inelasticity can increase their demands. At short-period ratio NSCs (i.e., rigid NSCs), the component inelasticity leads to significantly large ductility demands, and is not recommended in design. At long-period NSC ratios, the well-known equal displacement rule applies.

6) Parameters denoted as the component response modification factor, $R_{cc}$, and inelastic displacement ratio, $C_{cc}$, are defined to quantify the effect of NSC nonlinearity on its seismic force and displacement demands, respectively. Results show that $R_{cc}$ and $C_{cc}$ factors are functions of the component characteristics (i.e., tuning period ratio, viscous damping ratio, and target ductility); and at a lesser extent, of the supporting building characteristics (i.e., level of inelastic behavior; type of lateral-load resisting system, and fundamental period); the ground motion characteristics; and the vertical location of NSC within the building. The largest beneficial effect of NSC inelasticity is obtained for a roof-mounted low-damping NSC tuned to the building fundamental mode where the supporting building responds elastically.

7) Evaluations of the responses of archetype and instrumented buildings consistently reveal that several influential parameters on NSC seismic demands are not explicitly accounted for in the current ASCE 7-16 provisions. These parameters are the lateral-load resisting system, fundamental period, and the level of inelastic behavior of supporting building, 3D effects (in-plane flexibility of floor diaphragm and torsion), the NSC viscous damping ratio. The parameters that are “only approximately” incorporated (in many cases not adequately) are the tuning ratio of NSCs (rigid vs flexible component), the inelastic behavior of NSCs, the effect of the floor relative height, and the definition of seismic base in buildings with basement(s).
8) The last section of this part of the dissertation presents an evaluation of the recently proposed equation by ATC-120 Project for designing acceleration-sensitive NSCs. This evaluation is conducted for the four previously mentioned primary-secondary scenarios. The floor acceleration motions obtained from different floor levels of the baseline and overdesigned archetype buildings are used in the conducted evaluation. Potential improvements to the ATC-120 equations are proposed. The most important recommended improvement is regarding the $R_{\mu b}$ value. The proposed value for this parameter is the same for all floor levels. Results of this study suggest that the value of $R_{\mu b}$ may need to reduce from top to bottom floors. If the objective is to meet the criteria adopted when developing these equations (i.e., the limiting the NSC ductility to the predefined values), the following modifications are also recommended:

(i) increasing the upper limit of $PCA/PGA$ from 5.0 to 6.0. This upper limit is reached primarily when the NSC is elastic; (ii) incorporating an additional parameter (i.e., damping modification factor, DMF) to account for the NSC viscous damping ratios other than 5%. Based on the preliminary evaluations conducted in this study, and also some studies performed as part of the ATC-120 project, the value of this parameter for an elastic NSC with 2% damping is 1.6 whereas its value for an inelastic NSC with a target ductility of 2.0 and viscous damping ratio of 2% is the square root of 1.6 (i.e., 1.26). However, an alternative solution is to ignore modifications listed under (i) and (ii) and accept an increase in the ductility demands on NSCs with respect to the target values. This approach needs accommodating a larger NSC ductility demand. Adopting such an approach implies that no component remains in the elastic behavior range. This approach is further justifiable if one considers the following discussion:

If the force-based design approach is adopted, because of the 3D effects discussed in Chapter 2 of this dissertation, additional factors greater than 1.0 (in some instances as large as 1.3) may need to be applied on the baseline equations proposed by ATC-120. Applying the parameters incorporating 3D effects, NSC viscous damping deviation from the nominal 5%, simultaneously, can lead to very large design values, especially for elastic NSCs. For example, for an elastic NSC with 2% damping mounted on a floor with a flexible diaphragm system, two amplification factors of 1.5 and 1.3 should be simultaneously incorporated into the design equations. In other words, and additional amplification factor of 1.95 could be applied. This may significantly increase construction costs. However, as an alternative, NSCs can be designed for the baseline equations
without any further amplification due to 3D effects and/or deviation of NSC damping from 5%, if the ductility demand of NSCs for these cases can be controlled. If it is beyond admissible values prescribed in the design criteria, the design forces should be increased.

11.2.3 Future works

Future research works in this area should consider developing code-based designed models to (i) provide a more reliable quantification of the effects of 3D behaviors on NSC seismic demands (ii) estimate the location of seismic base in typical buildings with basement(s). Prediction equations should be developed for floor spectra’s damping modification factor to quantify the effect of variation of NSC viscous damping ratio from the nominal 5% value. The current NSC design equations, and also many research works, focus on the NSC force (spectral acceleration) demands whereas less attention is devoted to NSC displacement demands. Future research works should study NSC displacement demands in more detail. The study of the instrumented buildings showed that the in-plane diaphragm flexibility effects in typical single-story structures can cause a significant amplification in PFA and PCA responses of the middle of a roof supported at its two ends with respect to those of the roof edges. These amplifications could be as large as 5.0. However, the studied single-story buildings were mostly exposed to low intensity ground motions. To develop improved equations for designing NSCs attached to these structure, numerical building models should be developed and exposed to ground motions at the DE level, in which nonlinearity might occur in the lateral-load resisting elements or a floor system diaphragm. Numerical buildings models are needed to provide a revised definition of the seismic base location for buildings with basement(s).

11.3 Part II: Innovative control systems for protecting buildings and NSCs

11.3.1 Introduction

In the second part of this dissertation, which includes Chapters 6 through 10, modern seismic protection techniques are studied that can decrease seismic input demands to a building, as opposed to modifying the seismic resistance of a building, which is the approach taken in current design provisions such as ASCE 7-16. The conventional base-isolation (BI) and tuned-mass-damper (TMD) concepts are utilized to develop an innovative seismic control system that can reliably
enhance the seismic performance of the structural elements, NSCs, and building contents so that the building can be occupied and remain functional immediately after a design earthquake.

In the proposed system, which is referred to as a multi-floor isolation (MFI) system in Chapter 9 and a partial mass isolation (PMI) in other chapters, different portions of story masses are isolated from the superstructure to act as inherent seismic energy suppressors. As part of this dissertation, the most important drawbacks of using conventional TMD and BI systems are addressed, and it is stated how the proposed PMI approach can partially resolve these drawbacks. The PMI system is examined in linear elastic shear-building models with six, 12, and 20, 40 stories representing low-to high-rise building. Optimization is carried out on the PMI system’s parameters to minimize average normalized root-mean-square of inter-story drift responses of the structural frame under earthquake excitations with different frequency contents, while constraints are specified to control the isolated components (ICs) seismic responses. The ground excitation is modeled as a Kanai-Tajimi filtered Gaussian white noise process with narrow- or broad-band characteristics. For the optimization process, either parametric studies or a Genetic Algorithm is used. PMI configurations with different isolated mass ratios (IMRs), identical and dissimilar ICs along the building height, and different number of ICs are studied.

The seismic effectiveness and robustness of the PMI system are evaluated. In this evaluation, effectiveness relates to the ability of the system to significantly mitigate inter-story drift demands, and hence, improve seismic performance with respect to the uncontrolled case. Robustness refers to the ability of the system to provide a stable performance in the presence of epistemic uncertainties and aleatory variabilities. For the robustness evaluation, sensitivity analyses are conducted by incorporating variations in structural design parameters (i.e., the superstructure stiffness coefficient and damping ratio, and the ICs stiffness coefficient and damping ratio) as well as in ground motion characteristics (i.e., the predominant frequency and damping ratio of the Kanai-Tajimi excitation). A relatively large variation of +/-50% is assumed for these parameters.

The practicality and limitations of the PMI system is discussed. The effectiveness and robustness of the PMI system for protecting the structural and nonstructural components of building structures are evaluated.
11.3.2 Conclusions of the second part of this dissertation

The most important conclusions of the second part of this dissertation are presented next:

1) The approach implemented to model the superstructure viscous damping can significantly impact the fundamental- and higher-mode dominated responses of BI buildings even when the superstructure viscous damping ratio is as low as 2%. This sensitivity is more pronounced for higher-mode dominated responses (e.g., short-period floor spectral accelerations). A modified Rayleigh damping approach is proposed that can provide a reliable (and conservative) estimation of fundamental- and higher-mode dominated responses of BI buildings.

2) A PMI system could effectively integrate the benefits of conventional TMD and BI techniques while resolving some of their deficiencies particularly in high-rise buildings (e.g., practical and architectural challenges associated with the heavy additional mass in a TMD, and problems due to the inherent flexibility, heavy loads and overturning moments imposed on isolator bearings in a base-isolated tall building).

3) Three different ranges for the IMR could result in an iPMI system with behavior consistent with a TMD, a BI, or a TMD-BI hybrid system. It is demonstrated that (i) ICs at a low IMR can provide a building with multiple inherent mass dampers without the weight restrictions of common TMDs; (ii) ICs at a high IMR can divide a building into a relatively stiff super-frame and flexible ICs, such a scenario conceptually behaves similarly to an ideal BI, except that the structural frame remains almost stationary as opposed to the entire structure experiencing rigid-body displacements relative to the base; (iii) an iPMI system at intermediate IMRs ranks between a common TMD and an ideal BI system.

4) All three considered passive control strategies are effective under broad-band excitations in the test-bed buildings evaluated in this study. However, under narrow-band excitations, the control systems are effective only if the uncontrolled superstructure is stiffer than the soil profile. In all test-bed buildings, the most efficient passive control system is associated with the near-resonance case.

5) A BI-like system for a structure located on a soft soil site (i.e., under a narrow-band excitation) is highly effective for the resonance situation. However, the performance of this system is very sensitive to the dynamic characteristics of the fixed-base superstructure as well as the soft
soil. This observation implies that any misestimation of these influential parameters could cause significantly large structural responses, especially isolator drifts. Hence, the application of a BI-like system for a soft soil profile needs particular care, and it is not recommended.

6) In balancing ICs’ seismic responses and the global inter-story drift and lateral-floor acceleration demands on the structure, and taking into account structural and architectural constraints, applying an intermediate IMR (25%-50%) is recommended as an appropriate range for the implementation of the iPMI technique. Such a system, as a partial isolation technique, would retain significant advantages of traditional TMD and BI techniques (i.e., inter-story drift and acceleration response reduction of the entire building) while preventing significant displacement and acceleration demands on the ICs.

7) The GA-optimized PMI (gPMI) system, achieved by tuning ICs to different modes of vibration, could mitigate structural responses controlled by higher modes as well as those controlled by the structure’s fundamental mode. Therefore, the gPMI system slightly outperforms the iPMI configuration in terms of inter-story drift response reduction. However, this improvement is achieved at the expense of large ICs’ displacement and acceleration demands.

8) The iPMI approach, although effective, considerably influences the design, construction, and serviceability of a building at multiple floors while a conventional passive control system (e.g., a TMD or a BI system) affects only one specific floor. To address this deficiency, the seismic performance of the PMI technique with identical ICs at different subsets of stories (i.e., identical ICs at only roof, top two stories, etc.) is investigated. It is shown that ICs at a few top stories contribute the most to the PMI system efficiency. Hence, the number of ICs can be significantly reduced without a significant reduction in the efficiency of the PMI technique.

9) By mitigating seismic-induced inter-story drift demands as the primary performance objective, the PMI system has proved to be useful to mitigate seismic-induced floor acceleration demands, as well drift and acceleration demands on the isolated portions of a floor mass.

10) The effectiveness (i.e., the minimum value for the $f_{\text{drift}}^3$ at the design point) of the PMI system increases with an increase in the IMR and tends to saturate at IMR values approximately greater than 50%.
11) As an initial step toward designing a robust PMI system, an evaluation of the magnitude of the frequency response of the superstructure displacement at the roof level is performed. This evaluation suggests that PMI configurations with low IMRs (e.g., 5% and 10%) although effective in reducing the superstructure inter-story drift response at the resonance condition, do not provide adequate effectiveness when exposed to relatively small deviations (i.e., 10%) in the predominant frequency of the ground excitation. In terms of robustness, configurations with intermediate IMRs (e.g., 25% and 50%) provide significant seismic performance improvement with respect to the uncontrolled case for a wide range of the excitation predominant frequencies.

12) Simulation results illustrate that configurations with low IMRs although effective at the design point (e.g., using a 5% IMR can reduce $J_{\text{drift}}^S$ by 52%), can exhibit a decrease in effectiveness in the presence of changes in the design parameters. Overall, PMI configurations with intermediate IMRs (e.g., 50%) show a more stable/robust performance. For example, for a simultaneous deviation of 50% in the superstructure stiffness coefficient and damping ratio, as well as in the ICs stiffness and damping ratio, the worst $J_{\text{drift}}^S$ value for the PMI systems considered with IMRs of 5%, 50% and 90% is 0.99, 0.32, and 0.40, respectively, meaning that the configuration with an IMR of 50% exhibits the most robust performance. These results are consistent with those obtained from the evaluation of the roof displacement frequency response.

13) Additional studies were conducted to evaluate the optimum number and placement of ICs along the building height. Based on the optimization criteria adopted in this study, for a given number of ICs, the ICs can be simply located at the top stories. It is also shown that ICs can be distributed over the height of the building in such a way that they experience uniform acceleration responses.

14) Given the aforementioned observations related to the optimum placement of ICs along the height and values of IMR that result in more effective and robust designs, a Genetic Algorithm is used to design a robust system for a PMI configuration with top-10 ICs and an IMR of 50% in which the ICs can have different properties along the height. An assessment of the behavior of this system with respect to the response of an optimally-designed system with identical ICs illustrates that in terms of effectiveness, a near-optimal solution can be achieved by using identical ICs; however, such a system is unable to provide a robust design.
15) The conducted study illustrates that a PMI system with intermediate IMRs (e.g., 50%) and a few ICs at upper stories (e.g., top-half) can provide an effective and robust system to mitigate seismic demands. The main advantage of this system over conventional passive control strategies relies on its ability to behave almost equally effectively over a wide range of inherent uncertainties without compromising efficiency significantly.

16) The effectiveness and robustness of a conventional TMD and the proposed PMI system for reducing the seismic response of a single-degree-of-freedom NSC attached to the roof floor of a 12-story building are investigated. The response quantity to be evaluated is the magnitude of the transfer function of the absolute acceleration response of the NSC. It is shown that an optimized TMD tuned to the second mode of the building can significantly reduce the NSC response in the vicinity of the building second-mode but its effectiveness is deteriorated by variations in the base-excitation frequency. In other words, an optimal TMD is effective for a specific design point but is not robust against variations in the characteristics of the building and ground excitation. As another important disadvantage, this TMD system is effective only for NSC frequencies in the vicinity of the building second-mode but not for NSC beyond this frequency range. A Genetic Algorithm is used to design a robust PMI system for protecting NSCs with periods in the range \((4-200) \pi \text{ rad/s}\), wherein most typical NSCs are situated. An evaluation of the pseudo floor spectra of the robust PMI system illustrates that this system is equally effective for all NSCs in the frequency range of interest.

11.3.3 Future works

The effect of isolating building components on the vertical acceleration responses should be also studied. The PMI system’s efficiency in structural models with nonlinear seismic behavior should be examined as well. The effect of torsional responses caused by the out-of-phase movements of ICs should be investigated using three-dimensional models. The aesthetical aspects and possible environmental benefits of the proposed PMI system should be considered. For example, if isolated units (floors) are capable of accommodating large movements in the horizontal directions, the proposed technique can optimally use the sunlight during the daytime and save the building energy consumption. This latter characteristic can justify the initial cost of this relatively new control system, given that its structural control improvement component is achieved only during rare ground motions in a building lifetime.