University of Nevada, Reno

Fundamental Dynamics and Performance Assessment of Three-Dimensional Seismic Isolation

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

by

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Abstract

Seismic isolation is an effective technique used to mitigate effects of ground shaking and achieve higher seismic performance in building structures. Recent research has suggested that vertical excitation has significant effects on the structural response during an earthquake. In this dissertation, the dynamic response and control of buildings with 3-dimensional (3D) isolation systems is explored. First, the fundamental dynamic response of a simplified model consisting of a rigid block resting on isolation bearings is studied. The overall structural response of the 3D isolated block is evaluated through drift ratio, vertical and horizontal displacement and acceleration amplification factors. A parametric study is carried out to evaluate the effect of different site conditions, building aspect ratio and 3D isolation parameters on the structure and bearing response. The results show that effective 3D isolation is achieved by designing the system with vertical isolation period in the range of 0.5 – 1.0 sec. Also, the horizontal ($T_H$) and vertical ($T_V$) isolation periods should not be closely coupled, and $T_H$ should be selected to be much longer than $T_V$. Structural responses are seen to increase for increasing site class (increasingly softer soil) over the entire parameter range. Increasing horizontal $\zeta_H$ and vertical damping ratios $\zeta_V$, leads to decreasing drift ratios and vertical bearing displacement. However, increasing $h/b$ leads to an increase in rocking, which consequently impacts the relative drift ratio between the top and the base and the vertical bearing displacement.

Next, a concept is proposed for 3D isolation that combines elastomeric bearings to resist horizontal ground shaking in series with bilinear liquid spring (BLS) - controllable magnetorheological fluid dampers (CMRD) to resist vertical shaking. The simplified rigid
block model is extended to predict the response of the building isolated with BLS-CMRD devices under earthquake loading. BLS-CMRDs are simulated through a combination of nonlinear springs, viscous and hysteretic (semi-active) damping. A new Disp/Vel-based Control strategy is proposed that adjusts the input current according to the instantaneous vector combination of feedback displacement and velocity of the damper. Two variations of the control strategy are explored. First, with Linear Current Variation, the current is activated when a threshold lower bound vector magnitude is reached, and maximum current is applied when a threshold upper bound magnitude is exceeded. Second, the simplified ON-OFF strategy uses a single threshold vector magnitude that triggers the maximum current to turn on when the instantaneous vector magnitude exceeds the threshold, and turn off otherwise. Results of this analysis show that for ground motions that exceed the design level, Disp/Vel-based Control is effective to moderate the level of energy dissipation, keep device vertical displacement within the design stroke limit, and attenuate vertical acceleration below PGA. In addition, Disp/Vel-based control reduces all responses relative to the well-known clipped optimal strategy used for structural control.

Finally, the promising findings regarding 3D isolation are validated through a detailed study of the structural response of a real multi-story frame building with 3D isolation. Three hypothetical steel buildings of different aspect ratio with special concentric braced-frame lateral systems have been designed for this purpose. The seismic response of the buildings with fixed-base, horizontal (Hz) isolation and 3D isolation are compared to evaluate the effectiveness of 3D isolation to mitigate the vertical ground shaking. Also, responses predicted by a 3D rigid block model are compared to the flexible building response to identify the limitations of rigid block in predicting both global and local
structural responses. The simplified rigid block model estimates reasonably well the global structural responses, but does not account for slab vibrations. Overall, 3D isolation with a relatively short isolation period (0.5 sec) is found to be adequate to significantly mitigate the vertical acceleration in a flexible frame building, including the amplification at mid-slab relative to adjacent columns, without compromising the usual reductions in horizontal story drifts and floor accelerations. Aspect ratio was shown to insignificantly influence the structural response of 3D isolated buildings for the relatively short, stiff buildings considered in the study.
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Chapter (1) – Introduction

1.1 Motivation and Background

1.1.1 Previous studies investigating vertical vibration

Seismic isolation systems for buildings are designed to minimize structural and non-structural damage during and after an earthquake event. Most of the time, design and analysis focuses on evaluation of horizontal response only. One of the earliest studies to investigate three-dimensional (3D) ground shaking, Hwang and Hsu conducted bilateral, and triaxial shaking table tests to study the seismic response of a three-story base-isolated steel structure. The vertical ground acceleration increased the maximum story accelerations of the superstructure, whereas the maximum horizontal acceleration measured at the base floor was not significantly affected [Hwang and Hsu, 2000]. Unfortunately, vertical responses of the building due to vertical shaking were not reported.

Recent studies have suggested that even when horizontal shaking is mitigated by using seismic isolation, vertical excitation has the potential to cause damage to building structures during earthquakes, especially non-structural components and contents. Sato et al. [2011] conducted a full-scale shaking test of a four-story reinforced concrete medical building that was isolated with elastomeric bearings. The building was subjected to three-dimensional (3D) ground motions at Japan’s E-Defense shake table facility. The test showed that vertical accelerations at locations adjacent to columns were amplified relative to the shake table by a factor of 1.5, and further magnified by factors of 1.8-2.6 from the column locations to the centers of slabs when subjected to the 1995 Kobe at Kobe JMA station earthquake record. Moreover, for the 1940 El Centro recorded ground motion, the
peak vertical acceleration recorded at the table (0.5 g) was amplified to approximately 4 g at the center of the roof slabs. Disruption and damage to building contents and equipment was significant; however, damage to nonstructural components such as ceilings, piping and sprinkler systems, and partition walls and sliding doors was not observed.

Subsequently, a full-scale shaking test of a five-story steel moment frame building was carried out at E-Defense by US and Japanese researchers [Guzman and Ryan, 2015]. The building was tested with two different isolation systems – both of which were vertically stiff due to the presence of sliding bearings – and in the fixed-base configuration. An integrated ceiling, partition wall, and fire sprinkler piping (CPP) system was installed in the building to assess the performance of nonstructural components during 3D ground motion. In addition, the response of unattached contents was evaluated in two enclosed rooms. Vertical accelerations at column locations were amplified relative to the shake table by factors generally less than 2, while accelerations at mid-slab were amplified by average factors of approximately 3 on the second floor to 6 on the roof. Column acceleration amplification factors were low and insensitive to the height as columns were relatively rigid and could transfer the motion with little distortion. However, mid-slab vertical accelerations were dominated by slab vibration period that ranged from 0.08 to 0.14 sec in the test building [Ryan et al. 2016]. Field observations in real buildings validated the slab vibration frequencies observed in the test structure. The vibration period of composite floor systems in the field range from 0.08 to 0.25 sec, as measured through low-level forced vibration with an eccentric mass shaker [Hicks, 2004] or determined by expert opinion [Murray et al., 1997 and Allen and Pernica, 1998].
The large vertical accelerations measured in the building led to CPP component damage. This damage was qualitatively evaluated by inspection of all available video footage, and observed damage states were compared to recorded peak horizontal floor and vertical slab accelerations parameters for every earthquake simulation. Damage was shown to be closely related to vertical accelerations recorded in the center of the floor slabs. Damage to nonstructural components was classified as slight for vertical acceleration between 2-3g, moderate for 3-5g, and extensive for vertical acceleration above 5g.

Despite these recent experiments, significant damage to nonstructural components in isolated buildings during past earthquakes has not been reported. However, vertical response generally has not been measured to determine whether vertical accelerations have been significant. In fact, most instrumented buildings do not include the right kind of sensors to measure the slab vibration [Ryan et al. 2016]. Bozorgnia et al. [1998] examined 12 buildings shaken in the 1994 Northridge earthquake (both base-isolated and conventional) that all included some instrumentation to measure vertical shaking. Three of these buildings contained vertical accelerometers both at the ground level and on the roof slab away from a column. The ratios of peak vertical accelerations recorded in the roof to those at the ground level were 3.75, 6.4, and 2.4. In addition, Lew and Hudson [1999] examined several instrumented buildings and observed that vertical accelerations transmitted from the ground to the building were not affected by the presence of an isolation system.

Researchers have begun to consider strategies to mitigate vertical acceleration amplification due to the vertical component of ground motion. Guzman and Ryan [2017]
evaluated parameters that influence the vertical response of the floor system, including mass eccentricities, modeling assumptions of the isolators, and slab flexibility. A hypothetical three-story building with and without base isolation was investigated. Most floor system parameter variations led to slightly altered vertical modal frequencies, and these modified frequencies had an unpredictable influence on slab amplification factors. Thus, modifying the slab design is unlikely to be an effective mitigation strategy.

More effective strategies are needed to mitigate vertical vibration in isolated buildings or other applications where it is important to protect nonstructural components and contents. One possible way to achieve this is through combined horizontal and vertical isolation, or 3D isolation of the whole building. Thus, a research need is to develop functional 3D isolation systems for high performance applications.

1.1.2 Three dimensional isolation systems

Nowadays, 3D isolation systems have been investigated to fulfill nuclear facility design requirements. A 3D base isolation system can be achieved through specially designed 3D isolation devices or by combining vertical base isolation devices and horizontal base isolation devices in series [Inoue et al., 2004]. Suhara et al. [2003, 2005] developed 3D isolation by combining laminated rubber bearings as horizontal isolators in series with rolling seal-type air springs as vertical isolators. The rolling seal-type air spring is a steel/concrete cylinder lowered into an air cavity and attached with a rolling rubber seal. The proposed device was examined for earthquake excitation, and it performed as designed. Kashiwazaki et al. developed and tested a hydraulic system used for vertical isolation, also connected in series with laminated rubber bearings. The hydraulic system
consisted of load carrying hydraulic cylinders filled with nitrogen gas, to which fluctuating pressure could be transmitted by the attached accumulator units [Kashiwazaki et al., 2003].

Kageyama et al. proposed a 3D isolation system consisting of cable-reinforced air springs. The 3D air spring was composed of an inner cylinder attached to the base and an outer cylinder attached to the structure separated by an air cavity bounded by a flexible rubber sheet [Kashiwazaki et al., 2003 and 2004]. A shaking table test proved the feasibility of the developed system. Shimizu Corporation also developed a commercial solution for 3D isolation [Takahashi et al., 2008 and Suhara et al., 2008]. The proposed 3D seismic isolation device consists of a laminated rubber bearing for horizontal isolation and multiple air springs for vertical isolation. A rocking suppression device with an oil damper is used to control rocking vibration. The proposed system has been implemented on a three-story apartment house building. The system is quite complicated and perhaps cost prohibitive for wider implementation.

Finally, some researchers have focused on 3D isolation for a special class of long-span articulated or hangar structures. Li et al. [2013] suggested a 3D isolation bearing composed of a frictional sliding device in the horizontal direction and helical springs or disk springs in the vertical direction. A long-span hangar structure with these bearings was simulated using multistory frame modeling. The hangar structure with 3D bearings achieved improved performance by effectively reducing the axial force and acceleration response, while the displacement of the bearing remained within its predetermined range. Xu et al. [2012a and 2012b] proposed a new multidimensional earthquake isolation and mitigation device for protecting long-span reticulated structures due to strong earthquake
motions. The developed device consisted of a viscoelastic bearing and several viscoelastic dampers. The proposed device provided substantial energy dissipation as the force–displacement hysteretic loops were observed to be full smooth ellipses.

All previous studies focused on the development of vertical or 3D isolation devices or systems. The proposed systems have vertical isolation periods on the order of 1–2 s and generally utilize dampers (oil dampers or viscous wall dampers) and rocking suppression devices to control both vertical and rocking displacements. However, little work has been done to investigate the fundamental dynamics of 3D isolation systems to select target design parameters. Therefore, research is needed to identify governing parameters of 3D isolation system that optimize the overall response of the structure, considering the tradeoffs between different displacements and accelerations.

1.1.3 Control Strategies
Effective use of controllable damping helps the isolation system to accommodate a wider range of demands, and especially high intensity demands. Such controllable damping may be incorporated into vertical isolation systems to improve their ability to mitigate vertical shaking due to a variety of earthquakes. Several semi-active control algorithms have been proposed, such as bang-bang, decentralized bang-bang and clipped optimal [Dyke and Spencer, 1997]. Leitmann [1994] applied Lyapunov's direct approach for the design of a semi-active controller. This algorithm is classified as a bang-bang controller, and is dependent on the sign of the measured control force and the states of the system. These states are estimated based on available measurements (e.g., device displacement, device forces and structural accelerations) using the Kalman filter. The effectiveness of the
algorithm depends on balanced measurements of the responses throughout the full structure [Leitmann, 1994]. McClamrock and Gavin [1995] also used Lyapunov's direct approach to develop the decentralized bang-bang control law. In this approach, the Lyapunov function was chosen to represent the total vibratory energy in the structure (kinetic plus potential energy), which requires only the floor velocities and applied forces to be measured.

Dyke et al. [1996a, 1996b, 1996c and 1996d] proposed the clipped optimal control approach, and found it to be effective for use with MR dampers. Clipped optimal control uses a linear optimal controller that calculates a desired control force based on measured structural responses and the measured applied force. Through a force feedback loop, commands direct the MR damper to generate approximately the desired optimal control force. The bang-bang, decentralized bang-bang and clipped optimal control approaches were compared on a three story structure with MR dampers [Dyke and Spencer, 1997]. All three control algorithms varied in effectiveness for reducing different response parameters, and tradeoffs were observed. Compared to the other algorithms, bang-bang control achieved, on average, a balanced (but small) reduction of the peak displacement, interstory displacement and acceleration. However, the largest reduction in the peak acceleration was achieved with the decentralized bang-bang approach while the largest reductions in the peak roof and interstory displacements were achieved with the clipped optimal control.

Previous control strategies were all ON-OFF strategies, meaning either full current or no current was applied. Performance might be improved by using strategies that impose smooth variation to the control force through variable current input rather than abrupt changes. Also, variable current usage will impact the energy consumption for operating the
isolator relative to full current application. Thus, research need is to develop new control strategies that can take advantage of variable input current to keep forces and displacement within isolator capacity.

1.2 Dissertation Organization and Objectives

Research has been carried out to develop a new device – a bilinear liquid spring (BLS) - controllable magnetorheological fluid damper (CMRD) – to be used as a vertical isolator in 3D isolation system. In conjunction with this effort, this dissertation focuses on the overall system response of a building with 3D isolation to inform the design of the device. First, fundamental dynamics of 3D isolation is investigated to develop the device design parameters. Then, a new control strategy is developed to best utilize the semi-active capabilities of the device. Finally, performance of 3D flexible building with rubber bearing as horizontal isolator combined in series with BLS-CMRD as vertical one subjected to 3D ground motion is studied.

Following this introduction, three main chapters (Chapters 2-4) are presented, concluding with a final dissertation summary and overall conclusions chapter (Chapter 5). Each of the three main Chapters (2-4) corresponds to a stand-alone journal article, and thus they can be read in any order. The paper in Chapter (2) was published online in 2018, while the remaining two articles (Chapters 3 and 4) have been submitted for review and possible publication as of the filing of this dissertation. The title, authors, corresponding publication, description and objectives for each chapter are discussed in turn:

Chapter 2 presents a paper entitled “Parameters affecting dynamics of three-dimensional seismic isolation” by Eltahawy, Ryan, Cesmeci and Gordaninejad, published
in *Journal of Earthquake Engineering* in Nov. 2018. The paper presents the dynamic response of 2D isolated rigid block, subjected to variations of the horizontal and vertical isolation stiffness and damping parameters to better understand the dynamics and develop and underlying design philosophy for 3D isolation. The objectives of this paper are to identify governing parameters of 3D isolation system that optimize the overall response of the structure, considering the tradeoffs between different displacements and accelerations. Parameters that define the 3D isolation system include horizontal isolation period $T_{H}$, vertical isolation period $T_{V}$, damping ratios $\zeta$ and bilinear stiffness. Other relevant variables include site parameters, target spectra, and building height/width or aspect ratio $h/b$.

Chapter (3) presents a paper entitled “Displacement/velocity-based control of a liquid spring - MR damper for vertical isolation” by Eltahawy, Ryan, Cesmeci and Gordaninejad, submitted to *Structural Control and Health Monitoring* in Sep. 2018. In this paper, a new control strategy is proposed for the semi-active BLS-CMRD device when subjected to vertical ground shaking. In this strategy, the input current, which increases the controllable damping force, is set proportional to a vector valued function of the feedback displacement and velocity. The objective of this study is to optimize parameters to achieve the best performance with the proposed control strategy. Also, two variations of the proposed control approach are evaluated against clipped optimal control strategy. This research was conducted alongside development and testing of a quarter scale BLS-CMRD device in the Large Scale Structures Laboratory at University of Nevada, Reno.

Chapter (4) presents a paper entitled “Performance of Flexible Frame Building with Horizontal and 3D Seismic Isolation when subjected to 3D Ground Shaking” by Eltahawy
and Ryan submitted to *Earthquake Spectra* in Dec. 2018. Although the rigid block model provides good information about the dynamics of 3D isolation, certain important aspects of the response are neglected such as the vibration of the flexible floor systems. The objective of Chapter (4) is to evaluate the structural response and mitigation of vertical vibration in realistic flexible buildings with 3D isolation. Three hypothetical steel buildings with different aspect ratios that use special concentric braced-frame lateral systems have been designed and modeled. The seismic response of the buildings in fixed base, with horizontal (Hz) isolation, and with 3D isolation are compared. The effect of 3D isolation in mitigating the effect of vertical ground shaking is investigated, while qualifying the potential increase in horizontal response parameters (drift, acceleration) that should be considered in design. Also, responses predicted by a 3D rigid block model are compared with the responses of one of the flexible buildings to identify the limitations of rigid block in predicting both global and local structural responses. Finally, the influence of aspect ratio effect on the structural response of the building with 3D isolation is evaluated.

1.3 References


Kashiwazaki A., Shimada T., Fujiwaka T. and Moro S. [2003] "Study on 3-dimensional base isolation system applying to new type power plant reactor (hydraulic 3-dimensional base isolation system: No.1)," Proc. of the 17th International Conference on Structural Mechanics in Reactor Technology, Prague, Czech Republic.


Suhara J., Tamura T., Ohta K., Okada Y. and Moro S. [2003] "Research on 3-D base isolation system applied to new power reactor 3-D seismic isolation device with
rolling seal type air spring: Part 1," Proc. of the 17th International Conference on Structural Mechanics in Reactor Technology (SMiRT 17), Prague, Czech Republic. 


Chapter (2) – Parameters Affecting Dynamics of Three-Dimensional Seismic Isolation


Abstract

Seismic base isolation systems are mainly used to reduce seismic demands and minimize earthquake effect on structure performance. In this study, the fundamental dynamic response of structures with three-dimensional (3D) isolation systems is explored using a simplified rigid block model. A parametric study is carried out to evaluate the effect of different site conditions, structure properties and 3D isolation parameters on structure and bearing response. The results show that the acceptable range of 3D isolation periods is 0.5–1.0 s for vertical direction with horizontal isolation period around 3–4 times the vertical period and 20% damping in both directions.

2.1 Introduction

Earthquake shaking is one of the most critical problems that affect different structures. Earthquakes induce much larger structure acceleration and forces more than those developed from the static load only; therefore, seismic isolation systems are used to mitigate this effect. Seismic isolation provides a flexible interface that uncouples the structure from the ground. Typically, flexible isolators attenuate accelerations and forces caused by the earthquake through a lengthening of the structure period. The increased displacement demands as a result of the period lengthening are accommodated by the isolation devices. Several studies have examined seismic reliability and life cycle cost
analysis of base-isolated structures. These analysis results showed that seismic isolation systems generally provide significant improvements to overall system reliability [Alhan and Gavin, 2005; Chenet et al., 2007, Castaldo et al., 2016] and reductions in life cycle costs [Terzic et al., 2014, Terzic and Mahin, 2017, Cutfield et al., 2016; and Castaldo et al., 2016].

Traditional isolation systems control only the horizontal shaking and do nothing to mitigate vertical shaking. Recent studies illustrate that for seismic resiliency, vertical component of shaking needs to be considered as well as horizontal shaking, and vertical shaking greatly affects the performance of seismically isolated structures. Furukawa et al. conducted a full-scale shaking test of a four-story base-isolated reinforced concrete building. The test showed that vertical accelerations were significantly amplified relative to the ground in some cases in a building with rubber isolation bearings [Furukawa et al., 2013]. Another full-scale shaking test of a five-story base-isolated steel moment frame building was conducted by Ryan et al. [2016]. Observed damage was attributed entirely to large vertical input acceleration, because the horizontal structural acceleration was constrained to relatively low levels. Both tests clarified that nonstructural components and contents are greatly affected by vertical excitation intensity.

Nowadays, three-dimensional (3D) isolation system has been used to fulfill nuclear facility design requirements. A 3D base isolation system can be achieved through specially designed 3D isolation devices or by combining vertical base isolation devices and horizontal base isolation devices in series [Inoue et al., 2004]. Suhara et al. [2003, 2005] developed 3D isolation by combining laminated rubber bearings as horizontal isolators in series with rolling seal-type air springs as vertical isolators. The rolling seal-type air spring
is a steel/concrete cylinder lowered into an air cavity and attached with a rolling rubber seal. The proposed device was examined for earthquake excitation, and it performed as designed. Kashiwazaki et al. developed and tested a hydraulic system used for vertical isolation, also connected in series with laminated rubber bearings. The hydraulic system consisted of load carrying hydraulic cylinders filled with nitrogen gas, to which fluctuating pressure could be transmitted by the attached accumulator units [Kashiwazaki et al., 2003]. Kageyama et al. proposed a 3D isolation system consisting of cable-reinforced air springs. The 3D air spring was composed of an inner cylinder attached to the base and an outer cylinder attached to the structure separated by an air cavity bounded by a flexible rubber sheet [Kageyama et al., 2003 and 2004]. A shaking table test proved the feasibility of the developed system. Shimizu Corporation also developed commercial solution for 3D isolation [Takahashi et al., 2008 and Suhara et al., 2008]. The proposed 3D seismic isolation device consists of a laminated rubber bearing for horizontal isolation and multiple air springs for vertical isolation. A rocking suppression device with an oil damper is used to control rocking vibration. The proposed system has been implemented on a three-story apartment house building. The system is quite complicated and perhaps cost prohibitive for wider implementation.

Finally, some researchers have focused on 3D isolation for a special class of long-span articulated or hangar structures. Li et al. [2013] suggested a 3D isolation bearing composed of a frictional sliding device in the horizontal direction and helical springs or disk springs in the vertical direction. A long-span hangar structure with these bearings was simulated using multistory frame modeling. The hangar structure with 3D bearings achieved improved performance by effectively reducing the axial force and acceleration
response, while the displacement of the bearing remained within its predetermined range. Xu et al. [2012a and 2012b] proposed a new multidimensional earthquake isolation and mitigation device for protecting long-span reticulated structures due to strong earthquake motions. The developed device consisted of a viscoelastic bearing and several viscoelastic dampers. The proposed device provided substantial energy dissipation as the force–displacement hysteretic loops were observed to be full smooth ellipses.

All previous studies focused on the development of vertical or 3D isolation devices or systems. The proposed systems have vertical isolation periods on the order of 1–2 s and generally utilize dampers (oil dampers or viscous wall dampers) and rocking suppression devices to control both vertical and rocking displacements. However, little work has been done to investigate the fundamental dynamics of 3D isolation systems to select target design parameters. Zhou et al. [2016] examined the dynamic performance of several vertical and 3D isolation systems for potential application to modern nuclear facilities. An isolation system with vertical period (T_v) = 0.33 s was found to be feasible for the studied nuclear power plant model and could effectively reduce the vertical in-structure responses. Also, Zhou concluded that the rocking effect was obvious when T_v increased to 1.0 s, and vertical bearing displacements were at least as large as the horizontal displacements for T_v = 2.3 s. Finally, the authors [Eltahawy et al., 2017] presented preliminary results on the fundamental dynamic response of structures with both horizontal and vertical isolations based on only three ground motions. This article is an extension of that work and includes multiple ground motion ensembles for statistical analysis, as well as variations in soil conditions, structure properties and isolation characteristics.
The objective of this study is to identify governing parameters of 3D isolation system that optimize the overall response of the structure, considering the tradeoffs between different displacements and accelerations. Parameters that define the 3D isolation system include horizontal isolation period $T_H$, vertical isolation period $T_V$, damping ratios $\zeta$ and bilinear stiffness. Other relevant variables include site parameters, target spectra and building height/width or aspect ratio $h/b$. These parameters are varied widely to identify the parameters of an effective isolation system to mitigate both horizontal and vertical effects. The structural response is simulated as a two-dimensional (2D) rigid block on flexible isolators subject to both horizontal and vertical input excitations.

### 2.2 Site Parameters and Target Spectra

A hypothetical site location and soil type were defined to obtain target spectra representing the seismic hazard in the horizontal and vertical direction. The selected site is located in Los Angeles area on soil class (D) with Mapped Risk-Targeted Maximum Considered Earthquake (MCE$_R$), 5% damped spectral acceleration of $S_S = 2.11g$ at short periods and $S_1 = 0.74g$ at 1.0 sec period. MCE$_R$ horizontal and vertical target spectra were developed for this site according to ASCE 7-16 [ASCE, 2016] and FEMA P-750 [NEHRP, 2009], respectively. Equations to calculate spectra are illustrated in Figure 2-1 for the horizontal direction and Figure 2-2 for the vertical direction. In these equations, $S_{MS} = F_a S_S$ and $S_{M1} = F_v S_1$ where $F_a$ and $F_v$ are site coefficients for short periods and 1.0 sec periods, respectively. For site class (D) and $S_S \geq 1.25g$, $F_a = 1.0$, while $F_v = 1.5$ as $S_1 \geq 0.5g$. According to Section 23 of NEHRP [2009], $C_V$ is a vertical coefficient that depends on $S_S$ and site class. From Figures 2-1 and 2-2, the peak vertical to horizontal spectral acceleration ($V/H$) ratio is observed to be 0.8 $C_V$. For site class (D), $C_V$ varies from 0.7 for
$S_s \leq 0.2 \text{ g}$ to $1.5 \text{ for } S_s \geq 2.0 \text{ g}$; consequently, the V/H ratio ranges from 0.56 to 1.2. Based on $S_s$, the prescribed value is $V/H = 1.2$ for the given site. In addition, $V/H = 1.0$ and 0.75 were considered by modifying the target vertical spectra for the same horizontal spectra, to capture a wider range of ground motions and site conditions.

The developed horizontal and vertical MCE$_R$ spectra with 5% damping and various V/H ratio are presented in Figure 2-3. At longer periods, the vertical spectra are controlled by the requirement that vertical spectral acceleration does not fall below $\frac{1}{2}$ of the horizontal spectral acceleration [NEHRP, 2009]. This results in a step in the calculated vertical spectra midway through the descending range. The length of the step increases with decreasing V/H. The step terminates at $T_V = 0.5 \text{ sec}$ for all V/H, but begins at $T_V = 0.49 \text{ for } V/H = 1.2$, $T_V = 0.4 \text{ for } V/H = 1.0$, and $T_V = 0.275 \text{ for } V/H = 0.75$ (Figure 2-3). This step in the response spectrum does not represent the real motion, which consequently leads to difficulty in fitting the motions to the target spectra.

![Figure 2-1: Horizontal MCER spectrum according to ASCE 7-16 [ASCE, 2016]](image1)

![Figure 2-2: Vertical MCER spectrum according to FEMA P-750 [NEHRP, 2009]](image2)
2.3 Ground Motion Selection and Scaling

To study the effect of intense vertical shaking on the structural response, ground motions with relatively large vertical components were hand selected to represent the target spectra. Initially, 92 recorded motions were selected from the PEER NGA database [Chiou et al., 2008] by identifying ground motion triplets that by inspection best matched the shape and relative intensity of the target spectra for different V/H. These motions were amplitude-scaled to minimize the sum of the squared error between the response spectrum of the ground motion component and the target spectrum in each direction. The error was minimized over a period range from 1.5 to 4.0 sec in both horizontal directions, and from 0 to 2.0 sec in the vertical direction. The broad period range encompassed the parameter variation considered in the study. All three components were scaled by a single scale factor to preserve the relative component amplitudes of the original recorded motion as recommended by commentary of ASCE 7-16 [ASCE, 2016].
These 92 motions were filtered to select three subsets of 12 motions, one for each V/H ratio, to be used in the analysis. The number of motions per subset was limited because identifying motions that matched the target spectrum well in all directions using a single scale factor was found to be difficult. First, motions requiring scale factor > 3.0 were removed to exclude ground motion distortion effects that may come from excessive scale factors. Then multiple attempts were made by trial and error to choose a smaller set of 12 motions with median spectra that matched the target spectra in both horizontal and vertical directions. This process was repeated for each V/H ratio. The final ground motion suite selected for the site contains 36 scaled ground motions, divided into three subsets of 12 for different V/H ratios. The suite contains 18 unique records, as some records are repeated. Table 2-1 summarizes the selected ground motions and the calculated scale factors to match horizontal and vertical target spectra for V/H = 1.2, 1.0 and 0.75. Six motions are common in all three subsets, while five motions are used in both the first and second subsets, and only one motion is mutual in the first and third subsets.

Figure 2-4 shows the x, y and z components of the 12 scaled motions and median spectra, plotted against the target MCE_R horizontal and vertical spectra with V/H = 1.2. The X-component of the median spectrum is greater than Y-component for T_H < 2 sec, but the two are almost the same for T_H > 2 sec. For this reason, only X and Z-components are presented in the analysis hereafter. Figure 2-5 compares the target spectrum and the median spectrum of the 12 ground motions for different V/H ratios. As discussed previously, the step in the vertical target spectrum made matching the ground motions challenging. As a result, the discrepancy between the median and target spectra is relatively large and increases with decreasing V/H.
Table 2-1: Selected ground motions and scale factors for horizontal and vertical target spectra with different V/H ratios.

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
<th>Scale Factor</th>
<th>V/H = 1.2</th>
<th>V/H = 1.0</th>
<th>V/H = 0.75</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>San Fernando</td>
<td>1971</td>
<td>Pacoima Dam (upper left abut)</td>
<td></td>
<td>-</td>
<td>1.44</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Tabas, Iran</td>
<td>1978</td>
<td>Tabas</td>
<td>1.22</td>
<td>1.20</td>
<td>1.17</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>San Salvador</td>
<td>1986</td>
<td>Geotech Investig Center</td>
<td>2.67</td>
<td>2.59</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>LGPC</td>
<td>1.13</td>
<td>1.11</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Manjil, Iran</td>
<td>1990</td>
<td>Abbar</td>
<td>1.81</td>
<td>1.70</td>
<td>1.62</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Cape Mendocino</td>
<td>1992</td>
<td>Cape Mendocino</td>
<td>-</td>
<td>-</td>
<td>1.54</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Northridge</td>
<td>1994</td>
<td>LA - Sepulveda VA Hospital</td>
<td>2.16</td>
<td>2.12</td>
<td>2.07</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Northridge</td>
<td>1994</td>
<td>Beverly Hills - 14145 Mulhol</td>
<td>2.29</td>
<td>2.22</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Northridge</td>
<td>1994</td>
<td>Jensen Filter Plant Generator Building</td>
<td>1.32</td>
<td>1.30</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>Amagasaki</td>
<td>-</td>
<td>-</td>
<td>2.07</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU071</td>
<td>2.52</td>
<td>2.41</td>
<td>2.31</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU072</td>
<td>2.41</td>
<td>-</td>
<td>2.25</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU079</td>
<td>2.98</td>
<td>2.81</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>Chuetsu-oki</td>
<td>2007</td>
<td>Kashiwazaki Nishiyamacho Ikeura</td>
<td>-</td>
<td>-</td>
<td>2.03</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Chuetsu-oki</td>
<td>2007</td>
<td>Kashiwazaki NPP Unit 5: ground surface</td>
<td>1.65</td>
<td>1.63</td>
<td>1.60</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>L'Aquila, Italy</td>
<td>2009</td>
<td>L'Aquila - Parking</td>
<td>2.88</td>
<td>2.72</td>
<td>2.63</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>El Mayor-Cucapah</td>
<td>2010</td>
<td>Cerro Prieto Geothermal</td>
<td>-</td>
<td>-</td>
<td>1.60</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>El Mayor-Cucapah</td>
<td>2010</td>
<td>Calexico - Fire Station</td>
<td>-</td>
<td>-</td>
<td>2.69</td>
<td></td>
</tr>
</tbody>
</table>
2.4 Rigid Block Model

2.4.1 Model assumptions and equations

A simplified 2D rigid block model was developed to understand the fundamental dynamics of combined horizontal and vertical isolation. This block is supported on
isolation bearings at each base corner, represented by linear springs with total horizontal stiffness \((K_H)\) and vertical stiffness \((K_V)\), as shown in Figure 2-6. The block has mass \((m)\) and moment of inertia \((I_θ)\) lumped at the geometric center, which is the center of mass (CM). The model degrees of freedom (DOFs) are horizontal displacement \((U_X)\), vertical displacement \((U_Z)\), and rotation \((θ)\) of the block at its CM (Figure 2-6). The coupled equations of motion for the system subjected to horizontal and vertical ground accelerations \(\ddot{U}_{gx} (t)\) and \(\ddot{U}_{gz} (t)\) are:

\[
\begin{bmatrix}
m & 0 & 0 \\
0 & m & 0 \\
0 & 0 & I_θ \end{bmatrix}
\begin{bmatrix}
\ddot{U}_X (t) \\
\ddot{U}_Z (t) \\
\ddot{θ} (t) 
\end{bmatrix}
+ [C]
\begin{bmatrix}
\ddot{U}_X (t) \\
\ddot{U}_Z (t) \\
\ddot{θ} (t) 
\end{bmatrix}
+
\begin{bmatrix}
K_H & 0 & K_H * \frac{h}{2} \\
0 & K_V & 0 \\
K_H * \frac{h}{2} & 0 & K_H * \frac{h^2}{4} + K_V * \frac{b^2}{4} \end{bmatrix}
\begin{bmatrix}
U_X (t) \\
U_Z (t) \\
θ (t) 
\end{bmatrix}
= -
\begin{bmatrix}
m & 0 & 0 \\
0 & m & 0 \\
0 & 0 & I_θ \end{bmatrix}
\begin{bmatrix}
\ddot{U}_{gx} (t) \\
\ddot{U}_{gz} (t) \\
0 
\end{bmatrix}
\text{Eq. 2-1}
\]

The damping matrix \(C\) is defined based on Rayleigh damping calibrated for target damping ratios in the horizontal \(1^{st}\) and vertical \(2^{nd}\) or \(3^{rd}\) modes. According to the stiffness matrix, horizontal translation and rocking are coupled, while the vertical translation is uncoupled from the other two modes. The equations of motion are solved numerically in MATLAB using Newmark’s linear acceleration method (gamma \((γ)\) = 1/2 and beta \((β)\) = 1/6) with a time increment equal to the time step of the recorded ground motion.
2.4.2 Validation of rigid block modeling approach

A multi-story frame model was developed and analyzed using SAP2000 to validate the applicability of the simplified rigid block model to represent a flexible frame building with an isolation system. The model was a 5-story single bay plane frame with story height 3.6 m and bay width 9 m, consistent with an overall height-to-width or aspect ratio (h/b) = 2 of the rigid block. The frame was modeled using linear steel frame elements with lumped mass in the middle of each floor beam. Wide flange sections W1100x499 and W610x372 were used beams and columns, respectively. The two joints on each floor were connected with rigid constraint to eliminate local deformation; thus, the frame behaved as a shear frame with rigid beams. Lumped mass values were m = 175 kN sec²/mm at each floor, except the roof, where m/2 was applied. Linear link elements were applied at the frame base to model the isolation system. The stiffness of the links were defined to represent isolation period in both horizontal and vertical directions, while the rotation degree of freedom was fixed and the damping was set to be zero. Both rigid (R) and flexible (F) frame models were investigated to evaluate the influence of frame flexibility on accuracy.
of the rigid block model. The fixed-base properties were $T_{H,R} = 0.5$ sec, $T_{V,R} = 0.1$ sec for the rigid frame, while for the flexible frame were $T_{H,F} = 1.0$ sec, $T_{V,F} = 0.2$ sec.

The isolation system used in both models had $T_H = 3.0$ sec, and different values were considered for $T_V$: 0.1, 0.5, 1.0 and 2.0 sec. Constant damping of 5% was assigned for all higher modes. However, damping overrides were applied to the first three modes according to the following: 20% for both fundamental horizontal and vertical modes, and damping for the intermediate rocking mode calculated using the Rayleigh quotient. The damping in the multi-story frame model was selected to best match the applied viscous damping for the rigid block model, assumed to correspond to 20% damping ratio in both the horizontal and vertical modes based on a rigid superstructure assumption.

Table 2-2 compares the first three modal periods of the multi-story rigid frame and rigid block models for the different vertical isolation periods considered. The respective periods of the two different models differ by no more than 10%; except when $T_V = 0.1$ sec, the 2nd and 3rd mode periods differ by nearly 50% and 30%, respectively. The rigid block approach does not account for the superstructure flexibility that is present in the multi-story frame model; thus, the periods obtained from multi-story frame model exceeds the ones calculated from rigid block approach. As expected, the influence of the superstructure flexibility is more significant when the isolation period is smaller. Therefore, the modal periods of the multi-story frame and rigid block differ most when $T_V = 0.1$ sec; however, the discrepancy decreases when the vertical period increases.
Table 2-2: First three modal periods of the multi-story rigid frame and rigid block models

<table>
<thead>
<tr>
<th>Case</th>
<th>Multi-Story Rigid Frame Model</th>
<th>Rigid Block Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mode Period (sec)</td>
<td>Mode Period (sec)</td>
</tr>
<tr>
<td></td>
<td>Mode (1)</td>
<td>Mode (2)</td>
</tr>
<tr>
<td>$T_H = 3.0$ sec, $T_V = 0.1$ sec</td>
<td>3.1150</td>
<td>0.2721</td>
</tr>
<tr>
<td>$T_H = 3.0$ sec, $T_V = 0.5$ sec</td>
<td>3.3135</td>
<td>0.6778</td>
</tr>
<tr>
<td>$T_H = 3.0$ sec, $T_V = 1.0$ sec</td>
<td>3.9328</td>
<td>1.0876</td>
</tr>
<tr>
<td>$T_H = 3.0$ sec, $T_V = 2.0$ sec</td>
<td>5.9353</td>
<td>2.0375</td>
</tr>
</tbody>
</table>

The 1\textsuperscript{st} mode in the rigid block and multi-story frame models is mainly horizontal translation accompanied by a relatively small rocking angle as presented in Figures 2-7 (a) and 2-8 (a), respectively. The 2\textsuperscript{nd} mode in the rigid block model is governed by rocking with small horizontal translation (Figure 2-7 (b)), while the multi-story frame model sees height-based fluctuations in story drift ratio but the overall modes are comparable with the rigid block model (Figure 2-8(b)). In both models, the 3\textsuperscript{rd} mode is pure vertical translation without any rocking (Figures. 2-7 (c) and 2-8 (c)). The resulting mode shapes obtained from the multi-story frame and the rigid block model both have the characteristic that horizontal and rocking displacement are coupled, but vertical displacement is uncoupled from rocking. Therefore, the simplified rigid block model represents the dynamics of a multi-story frame model. Also, the coupling increases as $T_V$ increases, which is observed as increasing rocking in the horizontal mode and increasing horizontal translation in the rocking mode. In both rigid block and frame models, the 2\textsuperscript{nd} mode is the rocking mode and the 3\textsuperscript{rd} mode is the vertical mode up until $T_V = 1.0$ sec. However, the vertical and rocking mode switch to the 2\textsuperscript{nd} and 3\textsuperscript{rd} mode respectively when $T_V = 2.0$ sec (Table 2).
Figure 2-7: Mode shapes for rigid block model with different vertical isolation period $T_V$

a) 1$^{\text{st}}$ Mode Shape, b) 2$^{\text{nd}}$ Mode Shape and c) 3$^{\text{rd}}$ Mode Shape

Figure 2-8: Mode shapes for multi-story rigid frame model with different vertical isolation period $T_V$

a) 1$^{\text{st}}$ Mode Shape, b) 2$^{\text{nd}}$ Mode Shape and c) 3$^{\text{rd}}$ Mode Shape

Three representative ground motions (# 2, 7 and 12) scaled to the target spectra for $V/H = 1.2$ were applied to both the rigid block and multi-story frame models to compare their responses. History of roof drift ratio (in percent), vertical displacement, and total acceleration at the bearing are compared for the rigid block and multi-story frame models in Figures 2-9, 2-10 and 2-11, respectively, for each value of $T_V$ for ground motion (# 7).

For the rigid block model, drift ratio was computed as the ratio of relative horizontal displacement to the block height, which is shown to simplify to the rotation $\theta$:

$$\text{Drift ratio} = \frac{(U_{X_{\text{Top}}}-U_{X_{\text{Bottom}}})}{h} = \frac{(u_x - h/2 \theta) - (u_x + h/2 \theta)}{h} = -\theta \quad \text{Eq. 2-2}$$
In Eq. 2-2, $U_{X\text{Top}}$ is the horizontal displacement at the top of the rigid block and $U_{X\text{Bottom}}$ is the horizontal displacement at the bearing. For the multi-story frame model, the drift ratio was computed as total relative superstructure displacement from base to roof over total height. Visually, the two models are closely correlated, as the discrepancy between the response histories is minor. Also, for both models the rocking effect, as represented by drift ratio (noted earlier in the mode shapes), and the vertical displacement increases as $T_V$ increases in Figures 2-9 and 2-10, respectively. Discrepancy between the two models’ responses at $T_V = 0.1$ sec is noted, especially for vertical acceleration. Discrepancies in response of the two models shown in Figures 2-9, 2-10 and 2-11 coincide with discrepancy in the calculated periods, as discussed previously.

The following error measures were calculated for the three applied ground motions to better quantify the error in the rigid block model relative to the multi-story frame models. The normalized root mean square deviation (NRMSD), is the RMSD normalized by max peak-to-peak response:

$$\text{RMSD} = \sqrt{\frac{1}{N} \sum_{i=1}^{N} (r_{i,B} - r_{i,M})^2}$$  \hspace{1cm} \text{Eq. 2-3}

$$\text{NRMSD} = \frac{\text{RMSD}}{r_{\text{max},M} - r_{\text{min},M}}$$  \hspace{1cm} \text{Eq. 2-4}

where $r =$ response quantity of interest (drift, vertical displacement, acceleration), $i =$ $i^{th}$ step, $N =$ number of steps, $B$ refers to rigid block and $M$ refers to multi-story frame. NRMSD is a measure of average relative error in the response over the history. The peak error, or relative error in the peak responses, was computed as:

$$\text{Peak Error} = \frac{\max|r_{i,B}| - \max|r_{i,M}|}{\max|r_{i,M}|}$$  \hspace{1cm} \text{Eq. 2-5}
The average peak error and NRMSD obtained from the three applied ground motions for the drift ratio, vertical displacement and acceleration as a function of the vertical isolation period for the two frame models are presented in Figures 2-12 and 2-13, respectively. According to peak error and NRMSD calculations, the error in the rigid block model is maximized when $T_v = 0.1$ sec, for reasons discussed previously. The rigid block is a good simplified representative model of the response of a multi-story frame with horizontal and vertical isolation as the error quantity never rises above 25% for the peak error and 6% for NRMSD for $T_v \geq 0.5$ sec. However, the increase in the frame flexibility slightly increases the error between the rigid block and frame model. Furthermore, the correspondence between computed rotation $\theta$ of the rigid block and base to roof drift in the multi-story frame model confirms that the rotation of the rigid block is a good measure of overall drift, to be used with caution as the superstructure becomes more flexible.

Figure 2-9: Roof drift ratio of rigid block and multi-story frame for various $T_v$

Figure 2-10: Bearing vertical displacement for rigid block and multi-story frame for various $T_v$
2.5 Parametric Study of Rigid Block Model

A parametric study was carried out to investigate the effect of different parameters on the isolation performance. Considered parameters include $T_H$, $T_V$, $V/H$, site class, damping ratio ($\zeta$) and $h/b$.

2.5.1 Effect of Varying Both $T_H$ and $T_V$

The response trends of the rigid block are investigated for different combinations of horizontal and vertical isolation period. Peak responses are presented for the ground motion subset for $V/H = 1.2$ as a function of horizontal period $T_H$ varying from 0 to 5 sec. Peak responses are presented for each individual excitation; also shown are the median $\mu$, $\mu +$
1σ (84\textsuperscript{th} percentile), and µ - 1σ (16\textsuperscript{th} percentile) over the ground motion subset, where σ is the standard deviation, to summarize the variation with ground motions. In general, $T_H < 1$ sec represents the response of a comparable fixed base structure. As before, discrete values of $T_V = 0.1, 0.5, 1.0$ and 2.0 sec are considered, $\zeta = 20\%$ is applied in the horizontal and vertical modes and aspect ratio $h/b = 2$. The resulting peak horizontal base displacement (isolator displacement), peak drift ratio (in percent), and peak vertical displacement at the left bearing are shown in Figures 2-14, 2-15 and 2-16, respectively.

As expected, the bearing horizontal displacement increases with increasing $T_H$ (Figure 2-14) However, for a given $T_H$, bearing displacement decreases for higher values of $T_V$. Note that the bearing horizontal displacement is influenced by both $U_X$ and $\theta$. The rocking effect – represented by $\theta$ – increases with increasing $T_V$, which causes the bearing horizontal displacement to decrease while the roof horizontal displacement increases. The drift ratio (Figure 2-15) decreases with increasing $T_H$, but is also correlated to $T_V$. Also, the vertical bearing displacement is closely correlated to the drift ratio as shown in Fig. 16. Both the bearing vertical displacement and the drift ratio follow the trend for rocking $\theta$, and thus increase as $T_V$ increases. Furthermore, vertical displacement at the block center is unaffected by $T_H$, thus, variation in bearing vertical displacement with $T_H$ is due to rocking alone. Large drift ratios result even at short periods $T_H$ when vertical flexibility is introduced ($T_V = 0.5, 1$ or 2 sec). In each case, the drift ratio and vertical displacement are almost constant up to $T_H = T_V$. However, they eventually decrease as $T_H$ increases beyond $T_V$. This suggests that for effective design, $T_H$ and $T_V$ should not be closely coupled, and $T_H$ should be selected to be much longer than $T_V$. An isolated building designed per ASCE 7-16 is designed to remain linear under the design lateral forces, and not exceed a drift limit.
of 1.5% [ASCE 2016]. Therefore, the drift ratio demand of the rigid block is evaluated
gainst this code limit of 1.5%. Thus, $T_V = 0.5$ sec produces acceptable results for almost
all $T_H$, $T_V = 1.0$ sec produces acceptable drift ratios for $T_H > 3.5$ sec, and $T_V = 2$ sec
produces drift ratios higher than the ASCE limit for all values of $T_H$.

Figure 2-14: Horizontal base displacement (isolator displacement) for $V/H = 1.2$ and
different $T_V$

Figure 2-15: Relative drift ratio between top and bottom of the block for $V/H = 1.2$ and
different $T_V$

Figure 2-16: Vertical displacement at left bearing for $V/H = 1.2$ and different $T_V$

The $16^{th}$ and $84^{th}$ percentile curves represent bounds for the middle 68% of the
expected response. Responses with low dispersion (small difference between $16^{th}$ and $84^{th}$
percentile) are predicted with more confidence than those with high dispersion (large
difference between $16^{th}$ and $84^{th}$ percentile). The results show that both drift ratio and
bearing vertical displacement are predicted with increasing confidence as $T_H$ increases and as $T_V$ increases. However, the opposite is true for the bearing horizontal displacement. The choice of suitable $T_V$ will also be influenced by the ability of the isolation system to accommodate the displacement and reliably limit the maximum drift ratio below the acceptable design threshold; therefore, design within the low dispersion range is preferred.

Horizontal acceleration amplification factors, $A_{Amp,H}$ (peak horizontal acceleration in the structure normalized by horizontal peak ground acceleration or PGA) are illustrated in Figure 2-17 for the top (representative of the roof) and base of the block. $A_{Amp,H} < 1$ indicates that horizontal acceleration is attenuated or reduced compared to PGA. $A_{Amp,H}$ at the top exceeds that at the base when $T_H \leq 1.0$ sec and $T_V = 0.1$ sec, representative of fixed-base. However, when $T_H > 1.0$ sec and $T_V = 0.1$ sec – representative of a conventional isolated building - $A_{Amp,H}$ is nearly constant over the structure height due to the absence of rocking, and decreases with increasing $T_H$. For a 3D isolation system ($T_H > 1.0$ sec and $T_V > 0.5$ sec), $A_{Amp,H}$ at the top of the block is lower than $A_{Amp,H}$ at the base. The rocking of the structure causes this effect as the rocking mode acts in the opposite direction of the horizontal mode and thus counteracts the movement at the top as illustrated previously in Figure 2-7. The horizontal acceleration attenuates as $T_H$ increases; and a change in $T_V$ has insignificant influence on the horizontal acceleration after $T_H$ is sufficiently long (Figure 2-17). For example, at $T_H = 3.0$ sec, $A_{Amp,H} \approx 0.25$ regardless of vertical period.

The vertical acceleration amplification factor, $A_{Amp,V}$ (peak vertical acceleration at the block edge normalized by vertical peak ground acceleration or PGA$_Z$) is presented in Figure 2-18. In the range of typical horizontal isolation periods ($T_H \geq 2.0$ sec), $A_{Amp,V}$ is
not affected by $T_H$ variation, and is inversely proportional to $T_V$ as observed in Figure 2-18. $A_{Amp,V} < 1$ means vertical acceleration is attenuated or reduced compared to PGA$_Z$. For $T_V = 0.5$ sec, attenuation of vertical acceleration may be achieved but it is not reliable. However, for both $T_V = 1.0$ and $2.0$ sec, reliable attenuation of vertical acceleration can be achieved. Therefore, vertical acceleration attenuation can be achieved by increasing $T_V$ and hence the bearing flexibility, but the limiting factors are the vertical displacement and drift ratio.

In summary, 3D isolation helps limit horizontal and vertical accelerations and corresponding forces in the structure resulting from applied ground motion. Based on the results presented herein, the acceptable range of 3D isolation periods is concluded to be $T_V = 0.5$ sec and $T_H \geq 1.5$ sec or $T_V = 1.0$ sec and $T_H \geq 4.0$ sec, and intermediate periods as determined by interpolation. These ranges have been selected: 1) to keep drift ratio below ASCE limit [ASCE, 2016], 2) to attenuate both horizontal and vertical acceleration, and 3)
to achieve good separation of horizontal and vertical isolation periods so as to minimize rocking.

### 2.5.2 Effect of Varying Site Conditions

Peak responses for various V/H ratios and site classes are compared to investigate the variability of 3D isolation system response in different site conditions. As discussed previously in Section 2, \( V/H = 0.8 C_V \) where \( C_V \) is a vertical coefficient that depends on \( S_S \) and site class. Also, the target spectra vary as a function of site class. Next, two sets of analyses are presented to study the effect of different site conditions. First, rigid block responses are compared for \( V/H = 1.2, 1.0 \) and 0.75 for the same site parameters \( S_S, S_1 \) and site class D. For these analyses, different ground motions were selected and scaled to match the target spectra as described previously in Section 3. The rigid block was modeled with damping ratios \( \zeta = 20\% \) in the horizontal and vertical modes and aspect ratio \( h/b = 2 \).

Median (over the ground motion subsets) of the peak values of drift ratio, vertical displacement at the left bearing, \( A_{\text{Amp,H}} \) and \( A_{\text{Amp,V}} \) for different V/H ratios are presented in Figures 2-19 to 2-22. The responses are presented only for \( T_V = 0.5 \) and 1.0 sec based on the conclusions in the previous section. Drift ratio and bearing vertical displacement responses are seen to depend only weakly on V/H ratios (Figures 2-19 and 2-20), while the acceleration amplification factors are independent of V/H ratios (Figures 2-21 and 2-22). The obtained results may be a reflection of the ground motion selection and scaling procedures, and not a true evaluation of theoretical differences between different V/H ratios. Although best practice methods were used, several of the selected motions were common among multiple suites and the scale factors differed only slightly between suites,
as presented in Table 1. However, to fit any of the target spectra, the chosen ground motions all had large vertical intensities compared to typical motions in the database.

Some distinction between V/H = 0.75 and the two larger V/H ratios is apparent (Figures 2-19 and 2-20). In particular, drift ratio and bearing vertical displacement for V/H = 0.75 are lower than for V/H = 1.2 and 1.0 when \( T_H < 3.0 \) sec. The acceptable range of 3D isolation periods found in Section 5.1 for V/H = 1.2, based on the limitations specified previously, can be applied to all V/H ratios.

The second set of analyses – presented next – varies the site class while holding \( S_S \), \( S_1 \) and V/H constant. The original hypothetical site was classified as site class D (stiff soil). Site classes C (very dense soil/soft rock) and E (soft clay soil) are considered to investigate the isolation behavior with different soil types. In these analyses, ground motions selected previously for V/H = 1.2 were scaled to match the new target spectra presented in Figure
2-23. Again, the rigid block model was modeled with damping ratios $\zeta = 20\%$ in the horizontal and vertical modes and aspect ratio $h/b = 2$.

Median (over the ground motion subsets) of the peak values of drift ratio, vertical displacement at the left bearing, $A_{Amp,H}$ and $A_{Amp,V}$ for different site classes are presented in Figures 2-24 to 2-27. All responses are seen to increase for increasing site class (increasingly softer soil) over the entire parameter range. The increase in response in site class E compared to D is most pronounced, likely due to significant lengthening of the constant acceleration region of the horizontal spectrum (and corresponding lengthening of the “step” region of the vertical spectrum). The results suggest that a more limited range of horizontal and vertical isolation periods would satisfy acceptable drift limits for site class E due to the change in spectral shape.

![Figure 2-23: Horizontal and vertical MCE spectra with 5% damping for different site class](image-url)
The results suggest that the key isolator and structural response parameter trends are not much affected by site parameters $S_S$ and $S_1$; rather, governing system parameters and site class – which highly influences spectral shape – are much more influential. The subsequent analyses will consider only ground motions selected and scaled to match target spectra with $V/H = 1.2$ and for site class D.

### 2.5.3 Effect of Viscous Damping

The influence of variable viscous damping on the response of the rigid block is investigated next. For this analysis, isolation periods were selected as $T_H = 3.0$ and 5.0 sec and $T_V = 0.5$ and 1.0 sec (all four combinations were considered), and the aspect ratio of the rigid block model was $h/b = 2$. Median (over the ground motion subsets) of the peak values of drift ratio, left bearing vertical displacement, $A_{Amp,H}$ and $A_{Amp,V}$ are presented in Figures 2-28 to 2-31. The damping ratio in the vertical mode ($\zeta_V$) was varied from 10% to
40% and the damping ratio in the fundamental horizontal mode ($\zeta_H$) was varied discretely as 10, 20, 30 and 40%. Recall that damping in the 3rd (rocking) mode is determined by Rayleigh quotient.

![Figure 2-28: Relative drift ratio between top and bottom of the block for different damping ratio](image)

![Figure 2-29: Vertical displacement at left bearing for different damping ratio](image)

The drift ratio is relatively insensitive to $\zeta_V$ when $T_V = 0.5$ sec; however, it decreases slightly with increasing $\zeta_V$ when $T_V = 1.0$ sec. Increasing horizontal damping ratio $\zeta_H$ is more effective to reduce the drift than increasing vertical damping, especially for the shorter horizontal isolation period $T_H = 3.0$ sec. (Figure 2-28). Increasing damping up to 40% in both directions ($\zeta_H = 40\%$ and $\zeta_V = 40\%$) is still not sufficient to reduce the drift to the ASCE limit for the combination of $T_H = 3.0$ sec and $T_V = 1.0$ sec. This implies that the
horizontal period $T_H$ should be further lengthened to be paired with $T_V$ as long as 1.0 sec. Vertical bearing displacement is more sensitive $\zeta_V$, and increasing $\zeta_V$ is the most effective way to reduce bearing displacement (Figure 2-29). $A_{\text{Amp},H}$ has a decreasing trend with increases in both $\zeta_H$ and $\zeta_V$; however, the effect is rather minimal for all parameter combinations except $T_H = 3.0$ sec and $T_V = 1.0$ sec (Figure 2-30). Thus, for most reasonable systems, horizontal accelerations cannot be reduced much by increasing the damping. $A_{\text{Amp},V}$ is completely insensitive to $\zeta_H$, and can be reduced only slightly by increasing $\zeta_V$ (Figure 2-31).

![Figure 2-30: $A_{\text{Amp},H}$ at the block base for different damping ratio](image1)

![Figure 2-31: $A_{\text{Amp},V}$ at the block edge different damping ratio](image2)
Additional analyses are conducted for different combinations of $T_H$, $\zeta_H$ and $\zeta_V$ over the range $T_V = 0.5$ to $1.0$ sec, and the resultant drift ratio and vertical bearing displacement are plotted in Figure 2-32. Both drift ratio and bearing vertical displacement are inversely proportional to $T_H$, $\zeta_H$ and $\zeta_V$ (meaning that the drift ratio and vertical displacement can be reduced by increasing any of these parameters). All studied combinations of periods and damping ratios satisfy the ASCE drift ratio limit except moderate horizontal isolation period ($T_H = 3.0$ sec) and baseline damping ($\zeta_H = \zeta_V = 20\%$). In this case, the drift limit can be satisfied by limiting the vertical isolation period $T_V < 0.8$ sec. However, vertical displacement demand is most effectively reduced by decreasing the vertical period as illustrated in Figure 2-32.

Figure 2-32: Drift ratio and left bearing vertical displacement for different combination of $T_H$, $\zeta_H$ and $\zeta_V$

2.5.4 Effect of Superstructure Aspect Ratio

Recall that all respond trends thus far have been evaluated for a rigid block with aspect ratio $h/b = 2$. Next, the response trends of the rigid block are investigated for different structure aspect ratio ($h/b$) with various isolation properties. Peak responses are presented as a function of $h/b$ varying from 0.5 to 5, where $b$ is kept constant (10 m) and $h$
varies accordingly. Four isolation system parameter combinations were used in this analysis, $T_H = 3.0$ or $5.0$ sec, $T_V = 0.5$ or $1.0$ sec and $\zeta_H = \zeta_V = 20\%$. Median (over the ground motion subset) of the peak values of drift ratio and left bearing vertical displacement for different h/b ratios are presented in Figure 2-33. As expected, an increase in aspect ratio increases the rocking effect, which consequently leads to an increase in both drift ratio and vertical bearing displacement (Figure 2-33). However, the increase in these responses with aspect ratio is sublinear, which means that the rate of increase slows as aspect ratio. In fact, for some parameter combinations ($T_V = 1.0$ sec) the drift ratio and vertical bearing displacement reach a peak for h/b in the range of 2 to 4, and decrease thereafter.

Both drift ratio and vertical bearing displacement depend strongly on rotation $\theta$, which is influenced by the structure dimension and coupling between horizontal and rocking modes, as illustrated previously. Therefore, modal analysis was carried out for ground motion (# 13) to understand the dynamics related to aspect ratio. The modal coordinate $q$ represents the contribution factor of each mode to the total deformation. Response histories of the modal coordinates histories $q_1$ and $q_3$, which represent the horizontal and rocking modes, respectively, are presented in Figure 2-34 for $T_V = 0.5$ sec and Figure 2-35 for $T_V = 1.0$ sec, while $T_H = 3.0$ sec for both. The modal coordinate $q_i(t)$ is multiplied by the mode shape $\phi_i$ and summed over all modes to evaluate total displacement and rotation. Note that the mode shapes were normalized such that $\phi_i^T m \phi_i = 1$. In general, the modal coordinate $q_1$ decreases and $q_3$ increases as h/b increases, which is due to increased coupling between the horizontal mode and rocking mode. For $T_V = 0.5$, the changes in intensity of the modal coordinates are relatively small, and both drift ratio
and vertical bearing displacement increase as h/b increases. However, for $T_V = 1.0$ sec, $q_1$ decreases and $q_3$ increases significantly and $q_1$ and $q_3$ are out of phase; consequently, the drift ratio and vertical bearing displacement decrease at $h/b > 3.0$. Increasing aspect ratios do not appear to be a concern for escalating growth in the rocking response; however, for any aspect ratio selecting a shorter $T_V$ will help limit the vertical displacement demand.

**Figure 2-33:** Drift ratio and left bearing vertical displacement for different h/b

**Figure 2-34:** Modal history analysis results at $T_V = 0.5$ sec for different h/b

**Figure 2-35:** Modal history analysis results at $T_V = 1.0$ sec for different h/b

### 2.5.5 Effect of Bilinear Stiffness

If the design ground motion is exceeded, rocking of a building with 3D isolation may cause bearing vertical displacement demands large enough to induce tension in the
isolation devices. This is associated with an overturning concern when one side of the building engages in uplift. The authors hereby propose the concept of increasing the isolator resistance to tension relative to compression as an added safeguard against overturning. This can be achieved by designing a vertical or 3D isolator to have bilinear stiffness, with a tension stiffness $K_T$ that differs from the compression stiffness. The objective of a bilinear stiffness is to constrain the peak tensile displacement to very small values – an indication that activating the tension stiffness has arrested the overturning and prevented large tensile displacement demands – while simultaneously attenuating the vertical acceleration.

Additional analyses were conducted to investigate the effectiveness of the proposed concept for controlling or reduce the rocking behavior. The system parameters were selected as: rigid block aspect ratio $h/b = 2$, isolation properties $T_H = 3.0$ sec and $T_V = 0.5$ sec, $\zeta_H = 20\%$, $\zeta_V = 40\%$. The static displacement of the rigid block due to self-weight or gravity loading is 0.06 m, while the median bearing vertical displacement demand for this system (which can be qualitatively deduced from Figure 2-29) is 0.05 m. To evaluate the response when the bearings are consistently subjected to tension, the ground motion subset was scaled up by a factor of 2.0. Median peak values of left bearing vertical displacement and $A_{Amp,V}$ for varying tension stiffness are presented in Figure 2-36. The tension stiffness ($K_T$) is given as a multiple of the compression stiffness ($K$). The compression displacement is almost independent of $K_T$, however, tensile displacement slightly decreases as $K_T$ increases, suggesting that the strategy is moderately effective. The vertical acceleration amplification $A_{Amp,V}$ increases with increasing $K_T$, but vertical acceleration attenuation can be achieved by keeping $K_T$ below 8$K$. 

Figure 2-36: Vertical displacement at left bearing and $A_{\text{Amp},V}$ at block edge for different tension stiffness

2.6 Conclusion

A simplified rigid block model supported on isolation bearings at the block edges, and subjected to horizontal and vertical ground motion input, was used to investigate the fundamental dynamic response trends of a structure with a 3D isolation system. Furthermore, a multi-story frame model was developed to validate the numerical simplified rigid block model. Finally, a parametric study was carried out to evaluate the effect of different site conditions, structure properties and 3D isolation parameters on the structural and isolator responses. This study has led to the following conclusions:

- The rigid block model was shown to be a good simplified model to explore the fundamental dynamic behavior of a 3D isolated structure subjected to horizontal and vertical ground motion excitation. However, the rigid block model doesn’t account for variation in superstructure stiffness, and should be used with caution to represent flexible superstructures.

- The rotation $\theta$ in rigid block model is shown to be an effective “first order” prediction of the drift.
• The coupling between horizontal translation and rotation of the rigid block increases with increasing vertical isolation period $T_v$, which leads to increasing rotation and an associated increase in horizontal displacement at the top of the block (roof level) and a decrease at the base of the block (bearing level).

• Large drifts result even at low values of horizontal period $T_h$ when vertical flexibility is introduced ($T_v = 0.5, 1$ or $2$ sec). The drift ratio and bearing vertical displacement are almost constant up to $T_h = T_v$. However, both responses decrease when $T_h$ increases beyond $T_v$. This suggests that for effective design, $T_h$ and $T_v$ should not be closely coupled, and $T_h$ should be selected to be much longer than $T_v$.

• The key isolator and structural response parameter trends are not much affected by V/H ratio; rather, governing system parameters are much more influential. Although, the obtained results may be an artifact of the applied ground motions during the analysis, which had similar scale factors when scaled to fit the target spectra with different V/H. However, the chosen ground motions all have large vertical intensity relative to the horizontal.

• Structural responses are seen to increase for increasing site class (increasingly softer soil) over the entire parameter range. The increase in response in site class E compared to D is most pronounced, likely due to significant lengthening of the constant acceleration region of the horizontal spectrum (and corresponding lengthening of the “step” region of the vertical spectrum).

• Increasing horizontal and vertical damping ratios $\zeta_h$ and $\zeta_v$, leads to decreasing drift ratios and vertical bearing displacement. Increasing the damping ratios is more effective in limiting the drift and bearing displacement demands when $T_v = 1.0$ sec.
than when $T_V = 0.5$ sec. However, both horizontal and vertical acceleration amplification factor decrease with increasing $\zeta_V$.

- Increasing h/b leads to an increase in rocking, which consequently impacts the relative drift ratio between the top and the base and the vertical bearing displacement. However, for $T_V = 1.0$ sec, the drift ratio and vertical bearing displacement decrease at h/b > 3.0 due to increased coupling between the horizontal mode and rocking mode as well as those modes becoming out of phase. Although increasing aspect ratios do not appear to be a concern for escalating growth in the rocking response, selecting shorter $T_V$ will help limit the vertical displacement demand for any aspect ratio.

- Based on the results presented in this study, the acceptable range of 3D isolation periods is concluded to be $T_V = 0.5$ sec and $T_H \geq 1.5$ sec or $T_V = 1.0$ sec and $T_H \geq 4.0$ sec and $\zeta_H$ and $\zeta_V \geq 20\%$. These ranges have been selected: 1) to keep drift ratio below ASCE limit [ASCE, 2016], 2) to attenuate both horizontal and vertical acceleration, and 3) to achieve good separation of horizontal and vertical isolation periods so as to minimize rocking.

2.7 Funding

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Chapter (3) – Displacement/Velocity-based Control of a Liquid Spring - MR Damper for Vertical Isolation

*Note. This chapter is a stand-alone paper submitted on Sep., 2018 for Structural Control and Health Monitoring.*

**Abstract**

Seismic isolation is an effective technique used to mitigate effects of shaking and helps to achieve higher seismic performance. Recent research has suggested that vertical excitation has significant effects on structures behavior during earthquake. In this study, combined horizontal and vertical isolation is proposed that combines an elastomeric bearing to resist horizontal ground shaking in series with a bilinear liquid spring (BLS) - controllable magnetorheological fluid damper (CMRD) to resist vertical shaking.

A numerical model of simplified rigid 2D block was developed to predict BLS-CMRD response under earthquake loading. BLS-CMRDs were simulated through a combination of nonlinear springs, viscous and hysteretic (semi-active) damping. A Disp/Vel-based Control strategy was proposed that adjusts the input current according to the instantaneous vector combination of feedback displacement and velocity of the damper. Two variations of the control strategy were explored. First, with Linear Current Variation, the current is activated when threshold lower bound vector magnitude is reached, and maximum current is applied when threshold upper bound magnitude is exceeded. Second, the simplified ON-OFF strategy uses single threshold vector magnitude that triggers the maximum current to turn on when the instantaneous vector magnitude exceeds the threshold, and turn off otherwise.
Results show that for ground motions that exceed the design level, Disp/Vel-based Control is effective to moderate the level of energy dissipation, keep device vertical displacement within the design stroke limit, and attenuate vertical acceleration below PGA. In addition, Disp/Vel-based control reduces all responses relative to the well-known clipped optimal strategy used for structural control.

3.1 Introduction

Seismic isolation systems for buildings are designed to minimize structural and non-structural damage during and after an earthquake event. However, recent studies have suggested that vertical excitation has significant effects on the behavior of building structures during earthquakes, especially on non-structural elements. Furukawa et al. conducted a full-scale shaking test of a four-story base-isolated reinforced concrete building. The test showed that vertical accelerations were significantly amplified relative to the ground in some cases in a building with rubber isolation bearings [Furukawa et al., 2013]. Another full-scale shaking test of a five-story base-isolated steel moment frame building was carried out by Ryan et al. [2016]. Observed non-structural and content damage was attributed to large vertical input acceleration, because the horizontal floor accelerations were constrained to relatively low levels. Both tests clarified that non-structural component and content responses are greatly affected by vertical excitation intensity.

As a result, researchers have proposed to combine both horizontal and vertical isolation in series or develop 3D isolation devices to achieve better performance [Inoue et al., 2004]. For example, Suhara et al. [2003, 2005] developed and tested 3D isolation by
combining laminated rubber bearings as horizontal isolators in series with rolling seal type air springs as vertical isolators. The rolling seal type air spring is a steel/concrete cylinder lowered into an air cavity and attached with a rolling rubber seal. Kashiwazaki et al. developed and tested a hydraulic system consisting of load carrying hydraulic cylinders filled with nitrogen gas for vertical isolation, also connected in series with laminated rubber bearings [Kashiwazaki et al., 2003]. Kageyama et. al. [2003, 2004] proposed a 3D isolation system consisting of cable reinforced air springs. The 3D air spring was composed of an inner cylinder attached to the base and an outer cylinder attached to the structure separated by an air cavity bounded by a flexible rubber sheet.

The proposed systems have vertical isolation periods on the order of 1-2 seconds, and generally utilize dampers and rocking suppression devices to control both vertical and rocking displacements. Zhou et al [2016] examined the dynamic performance of several vertical and 3D isolation systems for potential application to modern nuclear facilities. An isolation system with vertical period $T_V = 0.33$ sec was found to be feasible for the nuclear power plant model examined, and could effectively reduce the vertical in-structure responses. Also, Zhou concluded that the rocking effect was obvious when $T_V$ increased to 1.0 sec, and vertical isolator displacements were at least as large as the horizontal displacements for $T_V = 2.3$ sec. Previously in Chapter (2), the fundamental dynamic response of structures with 3-dimensional 3D isolation systems using a simplified model of a rigid block resting on isolation bearings was explored. A parametric study was carried out to evaluate the effect of different site conditions, structure properties and 3D isolation parameters on the structure and isolator responses. The results showed that an acceptable range for the vertical isolation period is $T_V = 0.5 – 1.0$ sec, provided that the horizontal
isolation period $T_H$ is around 3-4 times greater than $T_V$ and 20\% equivalent viscous damping is provided in both directions.

In this research, a bilinear liquid-spring BLS - controllable magnetorheological fluid damper CMRD has been developed for vertical isolation that can also transmit the shear in the transverse direction. The purpose of the bilinear stiffness is to stiffen the device to arrest tension while simultaneously attenuating the vertical acceleration. The optimal parameters were found to be tension stiffness approximately four times the compression stiffness ($K_T \approx 4K_C$) as illustrated previously in Chapter (2). Recently, many researchers studied the capabilities of CMRDs and concluded that they have high potential for control of seismic-induced vibration [Dyke and Spencer, 1996 and Gordaninejad et al., 1999]. In such semi-active devices, the MR damping is activated by an input current [Carlson and Spencer, 1996 and Yi et al., 1998]. This controllable damping can be used to optimize energy dissipation according to earthquake intensity and limit the stroke of the BLS-CMRD to below its capacity.

Effective use of controllable damping helps the isolation system to accommodate a wider range of demands, and especially high intensity demands. Several semi-active control algorithms have been proposed, such as bang-bang, decentralized bang-bang and clipped optimal [Dyke and Spencer, 1997]. Leitmann [1994] applied Lyapunov's direct approach for the design of a semi-active controller. This algorithm is classified as a bang-bang controller, and is dependent on the sign of the measured control force and the states of the system. These states are estimated based on available measurements (e.g., device displacement, device forces and structural accelerations) using the Kalman filter. The
effectiveness of the algorithm depends on balanced measurements of the responses throughout the full structure [Leitmann, 1994]. McClamroch and Gavin [1995] also used Lyapunov's direct approach to develop the decentralized bang-bang control law. In this approach, the Lyapunov function was chosen to represent the total vibratory energy in the structure (kinetic plus potential energy), which requires only the floor velocities and applied forces to be measured. Dyke et al. [1996a, 1996b, 1996c and 1996d] proposed the clipped optimal control approach, and found it to be effective for use with MR dampers. Clipped optimal control uses a linear optimal controller that calculates a desired control force based on measured structural responses and the measured applied force. Through a force feedback loop, commands direct the MR damper to generate approximately the desired optimal control force. The bang-bang, decentralized bang-bang and clipped optimal control approaches were compared on a three story structure with MR dampers [Dyke and Spencer, 1997]. All three control algorithms varied in effectiveness for reducing different response parameters, and tradeoffs were observed. Compared to the other algorithms, bang-bang control achieved, on average, a balanced (but small) reduction of the peak displacement, interstory displacement and acceleration. However, the largest reduction in the peak acceleration was achieved with the decentralized bang-bang approach while the largest reductions in the peak roof and interstory displacements were achieved with the clipped optimal control.

In this paper, a new control strategy is proposed for the semi-active BLS-CMRD device when subjected to vertical ground shaking. In this strategy, the input current, which increases the controllable damping force, is set proportional to a vector valued function of the feedback displacement and velocity. The objective of this study is to optimize
parameters to achieve the best performance with the proposed control strategy. Also, two variations of the proposed control approach are evaluated against clipped optimal control strategy. This research was conducted alongside development and testing of a quarter scale BLS-CMRD device in the Large Scale Structures Laboratory at University of Nevada, Reno. Therefore, the design properties of the BLS-CMRD device, the numerical model developed to simulate its semi-active behavior, and the system model representing BLS-CMRDs acting in series with elastomeric bearings for 3D isolation are all presented.

3.2 Numerical Modeling

A simplified 2D rigid block is used in this study to model a hypothetical building structure. The discussion in this section is based on a full-scale prototype structure. This block is supported on isolation bearings at each base corner as shown in Figure 3-1. The isolation system is modeled as a combination of linear springs and dashpots to resist horizontal shaking in series with BLS-CMRDs to resist vertical shaking. The spring-dashpot is an adequate simplified representation of a traditional isolation device, such as an elastomeric bearing. Nonlinear springs are used to simulate BLS-CMRDs. The prototype block dimensions are height $h = 20$ m and width $b = 10$ m. This prototype block represents a hypothetical 5-story building with dimensions similar to the test building used in the aforementioned NEES/E-Defense project [Ryan et al., 2016]. This hypothetical building was assumed to be 20 m tall, $10 \times 10$ m and 2 bays $\times$ 2 bays in plan, and resting on the stacked isolation devices. The block has mass $m$ and moment of inertia $I_0$ lumped at the geometric center, which is also the center of mass CM. The model degrees-of-freedom DOFs are horizontal displacement $U_X$, vertical displacement $U_Z$, and rotation $\theta$ of the block
at its CM. The coupled equations of motion for the system subjected to horizontal and vertical ground accelerations $\ddot{U}_g(t)$ and $\ddot{U}_g(t)$ are:

$$M \dddot{U} + C \ddot{U} + K \dot{U} = -M \dddot{U}_g - L_C F_C$$  \hspace{1cm} \text{Eq. 3-1}$$

or

$$\begin{bmatrix}
m & 0 & 0 \\
0 & m & 0 \\
0 & 0 & I_0
\end{bmatrix}
\begin{bmatrix}
\dddot{U}_X(t) \\
\dddot{U}_Z(t) \\
\dddot{\theta}(t)
\end{bmatrix} +
\begin{bmatrix}
C_H & 0 & C_H \frac{h}{2} \\
0 & C_V & 0 \\
C_H \frac{h}{2} & 0 & C_H \frac{h^2}{4} + C_V \frac{b^2}{4}
\end{bmatrix}
\begin{bmatrix}
\dddot{U}_X(t) \\
\dddot{U}_Z(t) \\
\dddot{\theta}(t)
\end{bmatrix} +
\begin{bmatrix}
K_H & 0 & K_H \frac{h}{2} \\
0 & 0 & 0 \\
K_H \frac{h}{2} & 0 & K_H \frac{h^2}{4}
\end{bmatrix}
\begin{bmatrix}
\dddot{U}_X(t) \\
\dddot{U}_Z(t) \\
\dddot{\theta}(t)
\end{bmatrix}$$

$$= - \begin{bmatrix}
m & 0 & 0 \\
0 & m & 0 \\
0 & 0 & I_0
\end{bmatrix}
\begin{bmatrix}
\dddot{U}_{gx}(t) \\
\dddot{U}_{gz}(t) \\
0
\end{bmatrix} - \begin{bmatrix}
0 & 0 & 0 \\
1 & 0 & 1 \\
0 & 1 & 0
\end{bmatrix}
\begin{bmatrix}
K_{VR} U_{zR} + K_{VL} U_{zL} \\
(K_{VR} U_{zR} - K_{VL} U_{zL}) \frac{b}{2} \\
((F_{MR} + F_F)_R - (F_{MR} + F_F)_L) \frac{b}{2}
\end{bmatrix}$$  \hspace{1cm} \text{Eq. 3-2}$$

where $M$, $C$, and $K$ are mass, damping, and stiffness matrices of the system, $L_C$ is the control force location matrix, and $F_C$ is the control force vector that includes all nonlinear contributions to the equations of motion. The spring stiffness $K_H$ and damping constant $C_H$ represent the horizontal springs and dashpots, respectively, summed over all devices, and are defined consistent with assumed horizontal period $T_H = 3.0$ sec and viscous damping ratio $\xi_H = 20\%$. Likewise, $C_V$ is the vertical viscous damping constant used to compute viscous force $F_{ViscousV}$, summed over all BLS-CMRDs.

The control force vector, $F_C$, includes terms with the following nonlinear contributions: vertical bilinear spring force $F_{SpringV}$ (expressed in terms of left and right isolator vertical stiffness $K_{VL}$ and $K_{VR}$), the MR damping force $F_{MR}$ that depends on the
input current to the BLS-CMRD, and a friction force $F_F$ generated from seal friction on the device. These nonlinear force contributions are explained in detail in Section 4. Both vertical spring and MR forces are calculated at each isolator and then transformed to the model DOFs. $U_{Z_L}$ and $U_{Z_R}$ are left and right isolator vertical displacement, which are calculated as $U_Z - b/2 \theta$ and $U_Z + b/2 \theta$, respectively. The developed numerical model was evaluated via a state-space approach to solve the coupled differential equations using Simulink’s built-in solver.

Figure 3-1: Rigid block used in the analysis.

### 3.3 Target Spectra and Ground Motion Scaling

Target spectra were developed to represent the seismic hazard in the horizontal and vertical direction, which determine the design stroke for BLS-CMRD. A hypothetical site location and soil type have been defined to obtain these target spectra. The hypothetical site is located in greater Los Angeles area on soil class D. This site has Mapped Risk-Targeted Maximum Considered Earthquake MCE, 5 percent damped spectral acceleration
of $S_S = 2.11g$ at short periods and $S_1 = 0.74g$ at 1.0 sec period. The MCE$_R$ horizontal and vertical target spectral accelerations have been calculated for this site according to ASCE 7-16 [ASCE, 2016] and FEMA P-750 [NEHRP, 2009]. Equations to calculate these spectral accelerations are illustrated in Figure 3-2 for the horizontal direction and Figure 3-3 for the vertical direction. In these equations, $S_{MS} = F_a S_S$ and $S_{M1} = F_v S_1$ where $F_a$ and $F_v$ are site coefficients for short periods and 1.0 sec periods, respectively. For site class D and $S_S \geq 1.25$ g, $F_a = 1.0$, while $F_v = 1.5$ as $S_1 \geq 0.5$ g. According to Section 23 of FEMA-P750 [NEHRP, 2009], $C_V$ is a vertical coefficient that depends on $S_S$ and site class. From Figures 3-2 and 3-3, the peak vertical to horizontal spectral acceleration $V/H$ ratio is observed to be $0.8 C_V$. For site class D, $C_V$ varies from 0.7 for $S_S \leq 0.2$ g to 1.5 for $S_S \geq 2.0$ g; consequently, the $V/H$ ratio ranges from 0.56 to 1.2. Therefore, $V/H = 1.2$ for the selected site, which represents the largest intensity vertical hazard. The vertical spectral acceleration, adjusted for the effect of damping is converted to a spectral displacement at the design natural period, which corresponds to the design stroke of the BLS-CMRD.

Figure 3-2: MCE$_R$ spectrum for horizontal ground motion according to ASCE 7-16 [ASCE, 2016]

Figure 3-3: MCE$_R$ spectrum for vertical ground motion according to FEMA P-750 [NEHRP, 2009]
To study the effect of intense vertical shaking on the structural response, ground motions with relatively large vertical components were hand selected and scaled to represent the target spectra. These scaled ground motions are used in the numerical analysis conducted in this study to evaluate the proposed control approach. Initially, 30 recorded motions were selected from the PEER NGA database [Chiou et al., 2008] by identifying ground motion triplets that by inspection best matched the shape and relative intensity of the components of the target spectra. These motions were amplitude-scaled to minimize the sum of the squared error between the response spectrum of the ground motion component and the target spectrum in each direction. The error was minimized over a period range from 1.5 to 4.0 sec in both horizontal directions, and from 0 to 2.0 sec in the vertical direction. Both horizontal and vertical components were scaled by a single scale factor to preserve the relative component amplitudes of the original recorded motion, as referenced in the commentary of ASCE 7-16 [ASCE, 206].

These 30 motions were filtered to select a set of 12 motions to be used in the analysis. First, motions requiring a scale factor > 3.0 were removed to exclude ground motion distortion effects that may come from excessive scaling. Then, multiple attempts were made by trial and error to choose a smaller set of 12 motions with median spectra that matched the target spectra in two horizontal as well as vertical directions. Table 3-1 summarizes the selected ground motions and the calculated scale factors. The number of motions was limited because identifying motions that matched the target spectrum well in three directions using a single scale factor was found to be difficult. The medians of the twelve scaled ground motions are compared to MCER spectra for the horizontal x and vertical directions in Figure 3-4, as only one of the two horizontal components was used in
this study. Due to inconsistencies between relative component amplitudes of recorded and target spectra, the median spectrum of the scaled ground motions falls short of the target spectrum in the vertical direction.

Table 3-1: Selected ground motions and scale factors

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Tabas, Iran</td>
<td>1978</td>
<td>Tabas</td>
<td>1.22</td>
</tr>
<tr>
<td>2</td>
<td>San Salvador</td>
<td>1986</td>
<td>Geotech Investig Center</td>
<td>2.67</td>
</tr>
<tr>
<td>3</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>LGPC</td>
<td>1.13</td>
</tr>
<tr>
<td>4</td>
<td>Manjil, Iran</td>
<td>1990</td>
<td>Abbar</td>
<td>1.81</td>
</tr>
<tr>
<td>5</td>
<td>Northridge</td>
<td>1994</td>
<td>LA - Sepulveda VA Hospital</td>
<td>2.16</td>
</tr>
<tr>
<td>6</td>
<td>Northridge</td>
<td>1994</td>
<td>Beverly Hills - 14145 Mulhol</td>
<td>2.29</td>
</tr>
<tr>
<td>7</td>
<td>Northridge</td>
<td>1994</td>
<td>Jensen Filter Plant Generator Building</td>
<td>1.32</td>
</tr>
<tr>
<td>8</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU071</td>
<td>2.52</td>
</tr>
<tr>
<td>9</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU072</td>
<td>2.41</td>
</tr>
<tr>
<td>10</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU079</td>
<td>2.98</td>
</tr>
<tr>
<td>11</td>
<td>Chuetsu-oki</td>
<td>2007</td>
<td>Kashiwazaki NPP Unit 5: ground surface</td>
<td>1.65</td>
</tr>
<tr>
<td>12</td>
<td>L’Aquila, Italy</td>
<td>2009</td>
<td>L’Aquila - Parking</td>
<td>2.88</td>
</tr>
</tbody>
</table>

Figure 3-4: Median of twelve scaled ground motions compared to MCE\(_R\) spectra in (a) X-component and (b) Z-component
3.4 Design Properties of BLS-CMRD for Vertical Isolation

This section describes the design and nonlinear model of the BLS-CMRD. The model development was in conjunction with experimental work on a quarter scale BLS-CMRD that was fabricated and tested at UNR [Cesmeci et al., 2018]. The experimental program and results are summarized in Appendix (A). The device was subjected to cyclic loading with variable amplitude, frequencies and different input current to characterize device behavior. Also, displacement histories of three ground motions with different intensities were applied on BLS-CMRD at variable input current. The model described from here onward represents the fabricated device, and all analyses in this study were carried out for a one-quarter length scale model. Other basic scale factors based on appropriate similitude rules [Zohuri, 2015] are time scale factor = \( \frac{1}{2} \) and acceleration scale factor = 1:1. Applying these rules, the model block dimensions are \( h = 5 \text{ m} \) and \( b = 2.5 \text{ m} \) while, and the model horizontal spring stiffness corresponds to \( T_H = 1.5 \text{ sec} \). Table 3-2 summarizes the prototype and scaled model BLS-CMRD designed properties. The weight or dead load carried by a single BLS-CMRD was calculated based on the prototype block dimensions with devices distributed beneath the plan. The design stroke was computed based on selection of period, damping, and the developed MCE\(_R\) vertical target spectrum in the prototype domain, and converted to the model domain using the similitude relations.

As introduced previously, the BLS-CMRD resists ground motion through the additive effects of the spring force \( F_{spring,V} \), viscous damping force \( F_{viscous,V} \), friction force \( F_F \) and MR damping force \( F_{MR} \). To achieve this resistance, the proposed BLS-CMRD consists of a cylinder that has two chambers separated by a sealing system, a shaft with a piston, two caps to close the two chambers, and four external rods to fasten the caps against
the cylinder as shown in Figure 3-5. The top chamber is filled with MR fluid, while the bottom chamber is filled with pure silicone oil [Cesmeci, 2017].

Figure 3-5: Cut-out view of the BLS-CMRD [Cesmeci, 2017]

Table 3-2: Designed properties of prototype and model BLS-CMRD

<table>
<thead>
<tr>
<th>Property</th>
<th>Prototype</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period</td>
<td>0.5 sec</td>
<td>0.25 sec</td>
</tr>
<tr>
<td>Design Stroke</td>
<td>+/- 50 mm</td>
<td>+/- 12.5 mm</td>
</tr>
<tr>
<td>Stroke Limit (includes factor of safety and allowance for beyond MCE motions)</td>
<td>+/- 100 mm</td>
<td>+/- 25 mm</td>
</tr>
<tr>
<td>Weight for a Single BLS-CMRD</td>
<td>1484.8 kN</td>
<td>92.8 kN</td>
</tr>
<tr>
<td>Compression Stiffness $K_C$</td>
<td>24127.2 kN/m</td>
<td>6031.8 kN/m</td>
</tr>
<tr>
<td>Tension Stiffness $K_T$</td>
<td>102684 kN/m</td>
<td>25671 kN/m</td>
</tr>
</tbody>
</table>

Both spring and viscous forces are calculated as follows:
\( F_{\text{Spring,V}} = K_{V_{R/L}} U_{Z_{R/L}} \)  
Eq. 3-3

\( F_{\text{Viscous,V}} = C_{V_{R/L}} \dot{U}_{Z_{R/L}} = \left(1 + \frac{w h}{2A_p} \right) \frac{12 \mu L A_p^2}{w h^3} U_{Z_{R/L}} \)  
Eq. 3-4

where \( h \) is the height of the flow gap (m), \( A_p \) is the effective piston area (m²), \( \mu \) is the plastic viscosity of the MR fluid (kPa.sec), \( L \) is the effective axial pole length (m), and \( w \) is the piston width (m). Note that \( K_{V_{R/L}} \) takes on the value of \( K_T \) or \( K_C \) (Table 3-2) when the device is in compression (downward axial load) and tension (upward axial load), respectively. The friction between the seals and the shaft causes the friction force of approximately 5.5 kN (designed value) in the direction of motion. The MR force is activated by the input current \( I \) to the BLS-CMRD. The Bouc-Wen model [Charalampakis and Koumousis, 2009] is used to calculate MR and friction force, which are combined into a single equation, expressed as:

\( F_{\text{MR}} + F_F = \alpha \frac{F_y}{u_y} U + (1 - \alpha) F_y z \)  
Eq. 3-5

\( u_y z = \left[ A - \left( \beta \text{sgn} \left( \dot{U}_{Z_{R/L}} \right) + \gamma \right) |z|^n \right] \)  
Eq. 3-6

where \( A, \beta, \gamma \) and \( n \) are dimensionless quantities controlling the behavior of the model (\( A = 1, \beta = 0.5, \gamma = 0.5 \) and \( n = 1 \)). In the Bouc-Wen model, \( u_y = 1 \) mm is the yield displacement, \( \alpha = 0.0001 \) is the ratio of post-yield to pre-yield (elastic) stiffness, and \( z \) is dimensionless hysteretic parameter that obeys a single non-linear differential equation with zero initial condition. \( F_y \) is the yield force (kN) calculated as follows [Cesmeci, 2017]:

\( F_y = \left( 2.07 + \frac{12 Q \mu}{12 Q \mu + 0.4 w h^2 \tau_y} \right) \frac{\tau_y L A_p}{h} + 5.5 \)  
Eq. 3-7

\( \tau_y = 4.496 \tanh (1.108 I + 0.2893) \)  
Eq. 3-8
where \( Q \) is the flow rate through the annular gap (m\(^3\)/sec) and \( \tau_y \) is the dynamic yield stress (kPa) as calculated using Eq. 3-8. The BLS-CMRD mechanics are explained and illustrated, and the design equations are derived in details by Cesmeci, [2017].

Force displacement curves obtained from cyclic loading with amplitude +/-25 mm, frequency 4 Hz and current input \( I = 1 \) Amp are shown in Figure 3-6 (a). These values represent maximum allowed stroke, frequency corresponding to the vertical isolation period, and maximum applied current. The BLS-CMRD oscillates from the initial statically loaded state starting at the static displacement, \( U_{\text{Static}} = -15.47 \) mm (compression). The contributions of viscous damping forces and MR damping forces – the latter of which includes friction – are shown separately, and combined for the total effect. Viscous and MR damping forces depend on velocity, wherein the maximum velocity = 0.5 m/sec is achieved at the initial/static displacement. The force-displacement curves for both viscous and MR damping are observed to be unsymmetric about the x-axis as, the effective yield force is larger on the positive y-axis than the negative y-axis. This occurs because \( A_P \) for the rebound mode (piston moving toward the top chamber, Figure 3-5) is higher than that of the compression mode (piston moving toward the bottom chamber). This results in higher forces in the rebound mode for both viscous damping (Eq. 3-4) and MR damping (Eq. 3-7) [Cesmeci, 2017]. Maximum viscous damping forces are 31 and 72 kN, and maximum MR damping forces are 87 and 127 kN, in compression and rebound, respectively.

Equivalent damping ratios (\( \xi \)) due to viscous and MR components are computed as:

\[
\xi_{\text{Viscous}} = \frac{C}{C_{cr}} \quad \text{Eq. 3-9}
\]
\[
\xi_{\text{MR}+\text{Friction}} = \frac{1}{4\pi} \frac{E_D}{E_{\text{so}}}
\]

Eq. 3-10

where \(C_c\) = critical damping coefficient, \(E_D\) = energy dissipated in a cycle of harmonic vibration and \(E_{\text{so}}\) = maximum strain energy. According to these equations, BLS-CMRD has \(\xi_{\text{viscous}} = 20\%\) and \(\xi_{\text{MR}} = 40\%\). Because the equivalent damping ratios depend on stiffness, \(\xi_{\text{viscous}}\) and \(\xi_{\text{MR}}\) were calculated using the hysteresis loop obtained by applying the compression stiffness \(K_C\) over the whole cycle as shown in Figure 3-6 (b).

\[\begin{align*}
\text{a) Bilinear Stiffness} & \quad \text{b) Compression Stiffness} \\
\end{align*}\]

Figure 3-6: Force displacement curves obtained from cyclic loading with amplitude +/- 25 mm, frequency 4 Hz and current input 1 Amp.

### 3.5 Response of Passively Damped Structure

Passive-OFF and Passive-ON strategies are compared through response history analysis of the rigid block model subjected to the suite of 12 motions scaled to design level, to investigate the effect of full current-induced added damping on the overall structure behavior. During Passive-OFF no current is applied in the analysis, while during Passive-ON the maximum current of 1 Amp is applied continuously. Peak values of positive and negative vertical displacements (relative to the static displacement), vertical acceleration for the left BLS-CMRD, and overall drift ratio, are considered in this comparison. Peak
responses differ at the left and right devices because of the rotation $\theta$, however, the peak responses in the two devices were assumed to be similar when evaluated over many motions. The rigid block model drift ratio is computed as the ratio of relative horizontal displacement to the block height, which is shown to simplify to the negative of the block rotation angle $\theta$:

$$\text{Drift ratio} = \frac{(U_{X_{\text{Top}}}-U_{X_{\text{Bottom}}})}{h} = \frac{(U_X-\frac{h}{2}\theta)-(U_X+\frac{h}{2}\theta)}{h} = -\theta$$  \hspace{1cm} \text{Eq. 3-11}

In Eq. 3-11, $U_{X_{\text{Top}}}$ is the horizontal displacement at the top of the rigid block and $U_{X_{\text{Bottom}}}$ is the horizontal displacement at the base or isolator level. The rigid block drift ratio (rotation angle) has been shown in Chapter (2) to be a good estimate of total roof drift in a flexible structure. Also, since the BLS-CMRD vertical displacement depends on both $U_Z$ and $\theta$ (Eq. 3-2), the vertical displacement amplitude is naturally correlated with the drift ratio.

The cumulative distribution function (CDF) is calculated to represent the probability distribution of each response. CDF is the integral of the probability density function and is calculated (assuming a lognormal distribution) as follows:

$$P(\text{EDP} \leq x) = \frac{1}{\sigma \sqrt{2\pi}} \int_{0}^{x} \frac{e^{-(\ln(x)-\mu)^2/2\sigma^2}}{x} \, dx$$  \hspace{1cm} \text{Eq. 3-12}

where $P$ represents probability, EDP is an engineering demand parameter (response) of interest, $x$ is the value of the response, $\mu$ and $\sigma$ are the lognormally distributed mean and standard deviation computed from peak (absolute maximum) values of the 12 ground motions used in the analysis. CDFs for left BLS-CMRD peak positive and negative vertical displacements, peak vertical acceleration, as well as overall drift ratio are presented in
Figure 3-7. Peak relative positive and negative vertical displacements were calculated relative to $U_{Static} = -15.47$ mm, and peak negative displacement is shown as a positive (absolute) value. A relative positive vertical displacement $\leq 15.47$ mm means that no tension is sustained by the BLS-CMRD.

![Figure 3-7: CDF of peak responses at left BLS-CMRD, (a) vertical acceleration, (b) drift ratio, (c) relative negative vertical displacement, (d) relative positive vertical displacement](image)

The CDFs for peak BLS-CMRD displacements show that with no added damping (Passive-OFF), the negative and positive vertical displacement remain below the design stroke (Table 3-2) with approximately 70% and 80% probability, respectively. Applying maximum current (Passive-ON case) reduces the vertical displacement at the isolator and the overall drift ratio; but has the unwanted effect of increasing the BLS-CMRD vertical
acceleration substantially, such that it exceeds the vertical PGA. However, a key objective of isolation is to attenuate the acceleration below PGA. This demonstrates a need for developing an effective control strategy to apply current in response to varying ground shaking intensity, with the objective of limiting the device displacement demands when necessary. Therefore, a control strategy is proposed that aims to simultaneously limit the vertical displacement to the device stroke capacity and attenuate vertical acceleration below PGA.

3.6 Displacement/Velocity (Disp/Vel)-based Control

A control strategy is proposed that will adjust energy dissipation according to earthquake intensity and aims to limit the displacement or stroke of the BLS-CMRD below its capacity. In Disp/Vel-based Control, the input current to the BLS-CMRD is calculated based on the combined feedback displacement and velocity of the damper. The need to control displacement is clear as the displacement demand drives the design of the device. However, waiting until high displacement is detected to activate MR damping may be only partially effective. Since velocity is the derivative of displacement and the two functions are out of phase for sinusoidal motion, high velocity will generally precede a large displacement excursion. Therefore, a control strategy that responds to both displacement and velocity may be better-equipped to optimize energy dissipation. Two variations of the strategy are proposed.

Strategy I. Linear Current Variation: In this control strategy, the state of the damper is modeled as a vector valued function with components $U = \text{displacement}$ and $\dot{U} = \text{velocity}$, and the current is varied linearly with the instantaneous magnitude of the vector. The
current is activated when a threshold lower bound vector magnitude is reached, and maximum current is applied when a threshold upper bound vector magnitude is reached. These threshold magnitudes are modeled as ellipse surfaces with displacement as the semi-major axis and velocity as the semi-minor axis; $D_{\text{min}}$ and $V_{\text{min}}$ are the magnitudes of displacement and velocity for the inner ellipse or MR activation surface, while $D_{\text{max}}$ and $V_{\text{max}}$ are magnitudes of displacement and velocity for the outer ellipse, denoted the MR capping surface. These ellipses are centered at $U_{\text{Static}}$ and represented by the following equations:

\[
\left(\frac{U - U_{\text{Static}}}{D_{\text{min}}}\right)^2 + \left(\frac{\dot{U}}{V_{\text{min}}}\right)^2 = 1 \quad \text{(MR Activation Surface)} \quad \text{Eq. 3-13}
\]

\[
\left(\frac{U - U_{\text{Static}}}{D_{\text{max}}}\right)^2 + \left(\frac{\dot{U}}{V_{\text{max}}}\right)^2 = 1 \quad \text{(MR Capping Surface)} \quad \text{Eq. 3-14}
\]

In this context, the ellipse semi-minor to semi-major axis ratio is interpreted as a peak displacement to velocity ratio $D/V$. The instantaneous applied current as a function of the feedback displacement $U$ and velocity $\dot{U}$ is calculated by the following equations:

\[
\hat{u} = \left(\frac{U - U_{\text{Static}}}{D_{\text{min}}}\right)^2 + \left(\frac{\dot{U}}{V_{\text{min}}}\right)^2 \quad \text{Eq. 3-15}
\]

Case I: $\hat{u} < 1 \quad \rightarrow \quad I = 0$

Case II: $\hat{u} > R^2 \quad \rightarrow \quad I = 1 \text{ amp}$

Case III: $1 < \hat{u} < R^2 \quad \rightarrow \quad I = s x + I_o$

where $R$ is the outer to the inner ellipse ratio ($R = D_{\text{max}}/D_{\text{min}} = V_{\text{max}}/V_{\text{min}}$). For Case III, the current varies linearly with $x$ – the magnitude of the displacement and velocity vector – from the MR activation surface to the MR capping surface. The magnitude $x$, slope $s$ and intercept $I_o$ of the linear equation are given by:
\[ x = \sqrt{(U - U_{\text{Static}})^2 + (\dot{U} * D/V)^2} \quad \text{Eq. 3-16} \]

\[ s = \frac{1}{D_{\text{max}} - D_{\text{min}}}, \quad I_o = -\frac{D_{\text{min}}}{D_{\text{max}} - D_{\text{min}}} \quad \text{Eq. 3-17} \]

Strategy II. ON-OFF: The simplified ON-OFF strategy uses a single ON-OFF activation surface (ellipse characterized by \( D_O \) and \( V_O \)); the maximum current is turned on when the instantaneous vector magnitude is outside the ON-OFF activation surface and turned off when inside this surface. A schematic explanation of the proposed Linear Current Variation and ON-OFF strategies is presented in Figure 3-8.

![Figure 3-8: Proposed Linear Current Variation and ON-OFF theory.](image)

Most motions with scale factors as defined in Table 3-1 can be accommodated without activating the BLS-CMRD in semi-active mode, as illustrated in Figure 3-7. Thus, the following analyses have been conducted with the suite of motions scaled up by a factor of 3 (300% of design level) to evaluate the control strategy as added protection for very rare motions that exceed the design level. The histories of vertical displacement and current
at the left device are presented in Figure 3-9 for 300% of Manjil, Iran 1990 - Abbar (ground motion #4) for: 1) Passive-OFF (no current applied in the analysis); 2) Linear Current Variation with \( D_{\text{min}} = 12.5 \) mm, \( D_{\text{max}} = 22.5 \) mm and \( D/V = 0.07 \); and 3) ON-OFF control with \( D_0 = 17.5 \) mm and \( D/V = 0.07 \).

These histories show that the current is activated intermittently throughout the history according to the assigned thresholds in either control strategy. For Linear Current Variation, the current intensity varies after passing the MR activation surface, and the upper bound current of 1 Amp associated with reaching the MR capping surface is rarely observed. However, for ON-OFF control, the current fluctuates instantaneously between 0
and 1 Amp at the threshold ON-OFF activation surface, with no variation in between. With the additional damping, substantial reduction in the vertical displacement at corresponding instances of current activation is achieved in both cases relative to Passive-OFF.

### 3.7 Parameters Optimization for Linear Current Variation Control

Next, a sensitivity study to optimize the activation and capping surface intensities for the proposed Linear Current Variation strategy is presented. In this study, the displacement and velocity limits were systematically varied. Several responses were considered to evaluate the control strategy, including those previously defined (peak vertical displacement and acceleration of the left BLS-CMRD, overall drift ratio), and two additional parameters, which are operation time and energy ratio for the controller. The operation time T represents the cumulative time that the current is activated, while the energy ratio $E_R$ is calculated as follows:

$$E_R = \frac{E}{E_{\text{Passive-ON}}}$$  \hspace{1cm} \text{Eq. 3-18}

$$E = \int_0^T I^2 R \, dt$$  \hspace{1cm} \text{Eq. 3-19}

where $E$ = energy and $R$ = resistance. Eq. 3-19 was derived from energy $E = \int_0^T P \, dt$ where $P$ = power, $P = IV$ where $V$ = voltage, and $V = IR$. $T$ and $E_R$ reflect the ability of the proposed Linear Current Variation to control vertical isolator displacement without operating at full energy capacity. The maximum current is only activated when the combined displacement and velocity magnitude reaches the MR capping surface (determined by $D_{\text{max}}$ and $V_{\text{max}}$) as shown previously in Figure 3-8.
For the sensitivity study, the following parameter variations were considered. First, $D_{\text{min}}$ was varied from 10 to 20 mm with an increment of 2.5 mm, while $D_{\text{max}}$ was kept constant at 22.5 mm. Second, $D_{\text{max}}$ was varied from 12.5 to 22.5 mm with an increment of 2.5 mm, while $D_{\text{min}}$ was kept constant at 10 mm. The $D/V$ ratio was varied from 0.04 to 0.07 with an increment of 0.01 in both cases. Peak responses were computed for various combinations of $D/V$ and $D_{\text{min}}$ or $D_{\text{max}}$. Also, Passive-OFF and Passive-ON were considered for reference.

The median (over the 12 ground motions) of the peak absolute positive and negative vertical displacements, peak vertical acceleration, operation time $T$ and energy ratio $E_R$ for the left BLS-CMRD, as well as overall drift ratio, are presented as a function of $D_{\text{min}}$ and $D_{\text{max}}$ in Figures 3-10 and 3-11, respectively. Vertical displacement, drift ratio and vertical acceleration increase with increasing $D_{\text{min}}$ as shown in Figure 3-10 (a), (b) and (c). However, for increasing $D_{\text{max}}$, vertical displacement and drift ratio increase while vertical acceleration decreases as shown in Figure 3-11 (a), (b) and (c). The increase in the peak positive and negative vertical displacement for increasing $D_{\text{min}}$ is relatively small. As expected, peak absolute values of vertical displacement and drift ratio for Linear Current Variation are bounded by Passive-OFF and Passive-ON cases. However, Linear Current Variation helps reduce the vertical acceleration relative to both Passive-OFF and Passive-ON as desired for base isolation. Also, all studied cases mitigate the vertical acceleration below PGA, which is shown for reference in Figure 3-10 (c). The maximum permitted story drift ratio of 1.5% for design of isolated buildings [1] is shown for reference in Figure 3-10 (b) and 11 (b). When varying $D_{\text{max}}$, drift ratio ≤ 1.5% for all $D/V$ values, because of
the low value selected for $D_{\text{min}}$ (10 mm). However, when varying $D_{\text{min}}$, drift ratio $\leq 1.5\%$ only for $D_{\text{min}} \leq 13$ mm and $D/V = 0.06$ or 0.07.

Figure 3-10: Median of peak responses at left BLS-CMRD for $D_{\text{min}} = 10$-20 mm, $D_{\text{max}} = 22.5$ mm and $D/V$, (a) vertical displacement, (b) drift ratio, (c) vertical acceleration, (d) operation time, (e) energy ratio.
Figure 3-11: Median of peak responses at left BLS-CMRD for for $D_{\text{max}} = 12.5-22.5 \text{ mm}$, $D_{\text{min}} = 10 \text{ mm}$ and $D/V$, (a) vertical displacement, (b) drift ratio, (c) vertical acceleration, (d) operation time, (e) energy ratio.

Linear Current Variation leads to great reductions in operation time and consumed energy relative to Passive-ON as it is designed to scale the current relative to the instantaneous magnitude of response rather than apply the maximum current continuously. Operation time decreases as $D_{\text{min}}$ increases (Figure 3-10), but is almost unaffected by changes in $D_{\text{max}}$ (Figure 3-11), since $D_{\text{max}}$ only affects the amount of current applied. With regard to energy usage, increasing $D_{\text{min}}$ increases the magnitude of the activation surface, thus reducing the contribution of MR damping (reducing $E_R$) (Figure 3-10). On the other hand, increasing $D_{\text{max}}$ increases the magnitude of the capping surface that activates the full current, thus decreasing the contribution of MR damping (Figure 3-11). In general, $E_R$ is higher when $D_{\text{min}}$ is set to 10 mm and $D_{\text{max}}$ is varied, than when $D_{\text{max}}$ is set to 22.5 mm and
D_{\text{min}} is varied. However, the efficiency of Linear Current Variation is shown for all cases of D_{\text{min}} and D_{\text{max}} since ER < 10%.

The drift limit is the governing consideration for optimization of the proposed control strategy parameters. Optimized parameters are selected to be D_{\text{min}} = 12.5 \text{ mm}, D_{\text{max}} = 22.5 \text{ mm} and D/V = 0.07. The selected parameters keep the median drift ratio below the ASCE limit, limit the device operation time, and keep the energy ratio < 2.0 \% for the examined 300\% of design level intensity. However, it can be shown that this proposed control strategy gives very similar results to Passive-OFF case for design level motions. Likewise, optimized ON-OFF surface parameters are selected to be D_O = 17.5 \text{ mm} and D/V = 0.07, which represents an ON-OFF activation surface that is midway between (or an average of) the MR activation and MR capping surfaces according to optimization analysis.

3.8 Disp/Vel based Control Compared to a Benchmark Control Strategy

The clipped optimal control approach proposed by Dyke is a well-known and effective algorithm that has been used with MR dampers [Dyke et al., 1996 a, b, c and d]. The approach creates a force feedback loop that approximately produces a computed desired force in the control device:

\[ f_{C} = \mathcal{L}^{-1} \left\{-K_{C}(s)\mathcal{L}\left(\frac{y}{f}\right)\right\} \]

Eq. 3-20

where \( f_{C} \) is the desired optimal control force, \( \mathcal{L}\{\cdot\} \) is the Laplace transform, and \( K_{C}(s) \) is linear controller designed to provide \( f_{C} \) based on the measured responses \( y \) and measured force \( f \). The force \( f_{C} \) is generated by controlling the applied current \( I \) as follows:
$I = I_{\text{max}} H \{(f_c - f) f\}$

Eq. 3-21

where $I_{\text{max}}$ is the maximum applied current – 1 Amp in this study – and $H(.)$ is the Heaviside step function. The clipped optimal approach implements the simple strategy that if the desired force $f_c$ is greater than the measured force, the maximum current is turned on, otherwise the current is turned off.

The proposed Linear Current Variation and ON-OFF control strategy are compared to the clipped optimal approach, by analyzing the system subjected to the suite of 12 ground motions scaled to 300% of design intensity, with the various control strategies applied. The CDFs of the various responses are examined to understand not only average trends, but also trends in dispersion (i.e. design can be more reliable when the dispersion in predicted response is low). Also, Passive-OFF and Passive-ON were considered for reference. The CDFs for peak relative (to the static displacement) positive and negative vertical displacements, peak vertical acceleration, operation time $T$ and energy ratio $E_R$ for the left BLS-CMRD, as well as overall peak drift ratio, for all cases are presented in Figure 3-12. Average vertical accelerations for Passive-ON, Linear Current Variation and ON-OFF control are all lower than for Passive-OFF, while the average value for clipped optimal exceeds that for Passive-OFF. Importantly, the dispersion is higher for Passive-OFF confirming that the control strategies make the response more predictable. However, in all cases peak vertical acceleration is lower than PGA, which shows that acceleration is attenuated (Figure 3-12 (a)). The three control strategies (Linear Current Variation, ON-OFF strategy, and clipped optimal) all lead to peak drift ratio and relative negative and positive vertical displacements that are reduced relative to Passive-OFF but increased relative to Passive-ON as illustrated in Figure 3-12 (b), (c) and (d), respectively. However,
the CDFs for Linear Current Variation and ON-OFF strategy – all responses – are very close and have relatively lower response intensities than CDFs for the clipped optimal approach. Recall that relative positive vertical displacement ≤ |U_{Static}| = 15.47 mm means that no tension is sustained by the BLS-CMRD. The control strategies are generally not effective to prevent tension on average; however, the increased tension stiffness in the BLS-CMRD device (K_T ~ 4K_C) is effective to prevent very large tensile displacements from developing, especially with Linear Current Variation and ON-OFF control. This leads to peak relative positive displacements that on average are considerably less than peak relative negative displacements (Figure 3-12 (c) and (d)).
Figure 3-12: CDF of Peak responses at left BLS-CMRD, (a) vertical acceleration, (b) drift ratio, (c) relative negative vertical displacement, (d) relative positive vertical displacement, (e) operation time, (f) energy ratio.

Applying any control strategy leads to substantially reduced operation time compared to Passive-ON as presented in Figure 3-12 (e). For Linear Current Variation, ON-OFF strategy, and clipped optimal, T = 5, 1.7 and 2 sec, respectively, while T = 100 sec for Passive-ON. Linear Current Variation uses the least energy since it doesn’t have to apply maximum current all the time as shown in Figure 3-12 (f). To better illustrate the application of current in different control strategies, the time history of current is presented for 300% of Loma Prieta, 1989 – LGPC (ground motion #3) in Figure 3-13.
Figure 3-13: Current time history results for 300% of Loma Prieta, 1989 – LGPC

For this motion, energy ratio $E_R = 1.3$, 2.5 and 3.4 % for Linear Current Variation, ON-OFF strategy and clipped optimal approach, respectively, while $T$ is 1.55, 0.6 and 0.75 sec. The energy ratio is decreased while operating time is increased for Linear Current Variation relative to the other two strategies. This is because the current is activated at smaller response intensities, but its intensity is more limited. On the other hand, clipped optimal operating time is higher than ON-OFF, but it does not lead to improved response control.

3.9 Conclusions

A simplified 2D rigid block supported on isolation bearings at the block edges was used to model a hypothetical building structure with both horizontal and vertical isolation. The isolation system was modeled as a combination of elastomeric bearings to resist horizontal shaking in series with BLS-CM RDs to resist vertical shaking. Elastomeric
bearings were simulated with linear springs and dashpots, while BLS-CMRDs were simulated through a combination of nonlinear springs and viscous and hysteretic (semi-active) damping. A Disp/Vel-based Control strategy was proposed that adjusts the input current energy dissipation according to the instantaneous combined feedback displacement velocity of the damper device, and aims to limit the displacement or stroke of the BLS-CMRD below its capacity. Two variations of the control strategy were explored. For Linear Current Variation, the current is activated when a threshold lower bound magnitude of the vector combination of displacement and velocity is reached, and maximum current is applied when a threshold upper bound magnitude is reached. These threshold magnitudes are modeled as ellipse surfaces with displacement as the semi-major axis and velocity as the semi-minor axis; $D_{\text{min}}$ and $V_{\text{min}}$ are the magnitudes of displacement and velocity for the inner ellipse or MR activation surface, while $D_{\text{max}}$ and $V_{\text{max}}$ are magnitudes of displacement and velocity for the outer ellipse, denoted the MR capping surface. Alternatively, the simplified ON-OFF strategy uses a single ON-OFF activation surface that triggers the maximum current to turn on when the instantaneous vector magnitude moves outside the surface and turn off when it moves back inside this surface. An optimization study was conducted to select the activation and capping surface intensities for the Linear Current Variation strategy. The proposed Linear Current Variation and ON-OFF control strategies were evaluated with reference to the well-known clipped optimal approach, Passive-ON and Passive-OFF.
This study has led to the following conclusions:

- When subjected to design level shaking, Passive-ON reduced the BLS-CMRD vertical displacement and the overall drift ratio relative to Passive-OFF, but with a big penalty to acceleration, leading vertical acceleration demands to exceed the peak ground acceleration (PGA), which does not meet the objective of the isolation system. This indicated the need for a control strategy to moderate the level of energy dissipation.

- The proposed Displ/Vel-based Control was effective for ground motions that exceeded the design level, to moderate the level of energy dissipation, keep device vertical displacement within the design stroke limit, and attenuate vertical acceleration below PGA.

- The control activation and capping surface parameters were optimized considering the ASCE 7-16 drift limit for isolated buildings as the governing factor. Selected parameters were $D_{\text{min}} = 12.5 \text{ mm}$, $D_{\text{max}} = 22.5 \text{ mm}$ and $D/V = 0.07$, which limited the drift ratio to 1.5% even under application of motion intensities up to 300% design level. Furthermore, ON-OFF surface parameters were selected to be $D_{\text{ON}} = 17.5 \text{ mm}$ and $D/V = 0.07$ as an average surface between MR activation and capping surfaces.

- Nearly 30% of vertical acceleration values in case of Passive-ON, Linear Current Variation and ON-OFF strategy cases and 60% in clipped optimal case are higher than Passive-OFF case, however, it is still lower than PGA which enhances that acceleration attenuation is achieved.
• Both variations of Disp/Vel-based Control (Linear Current Variation and ON-OFF) reduced peak vertical acceleration, drift ratio, absolute negative and positive device stroke responses relative to the well-known clipped optimal control.

• Applying any control strategy substantially reduced the operation time compared to Passive-ON. For Linear Current Variation, ON-OFF strategy, and clipped optimal; T = 5, 1.7 and 2 sec, respectively, while T = 100 sec for Passive-ON.

• Disp/Vel-based Control greatly reduced the energy requirements relative to Passive-ON, as the energy ratio (fraction of energy used relative to Passive-ON) < 10% for all values of Dmin and Dmax under application of motion intensity up to 300% design level. As the energy ratio calculations depend on the current intensity, Linear Current Variation used the least energy since the maximum current is not always applied.

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3.11 References

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Chapter (4) – Performance of Flexible Frame Building with Horizontal and 3D Seismic Isolation when subjected to 3D Ground Shaking

Note. This chapter is a stand-alone paper submitted on Dec., 2018 for Earthquake Spectra.

Abstract

Seismic isolation is known to effectively mitigate effects of horizontal ground shaking in building structures. Recent research has suggested that vertical excitation causes high intensity slab vibrations that lead to nonstructural components damage. A detailed study of the structural response of a real multi-story frame building with 3D isolation is presented. The seismic response of hypothetical special concentric braced frame buildings with horizontal and 3D isolation are compared to evaluate the effectiveness of 3D isolation to mitigate the vertical ground shaking. Overall, 3D isolation with a relatively short isolation period (0.5 sec) is adequate to significantly mitigate the vertical acceleration in a flexible frame building, including the amplification at mid-slab relative to adjacent columns, without compromising the usual reductions in horizontal responses. Also, a simplified rigid block model is shown to estimate reasonably well the global structural responses, but underestimates story drifts and does not account for slab vibrations.

4.1 Introduction

Seismic isolation systems for buildings are utilized to minimize structural and non-structural damage during and after an earthquake event. Recent studies have suggested that even when horizontal shaking is mitigated by seismic isolation, vertical excitation has the potential to cause damage to buildings during earthquakes, especially the non-structural components and contents. For instance, Sato et al. [2011] and Furukawa et al. [2013]
conducted a full-scale shaking test of a four-story reinforced concrete medical building that was isolated with elastomeric bearings. The building was subjected to three-dimensional (3D) ground motions at Japan’s E-Defense shake table facility. The test showed that vertical accelerations were amplified in the columns relative to the shake table and at the centers of slabs relative to the columns; e.g., for the 1940 El Centro recorded ground motion, the peak vertical table acceleration (0.5 g) was amplified to approximately 4 g at the center of the roof slabs. Disruption and damage to building contents and equipment was significant; however, damage to nonstructural components such as ceilings, piping and sprinkler systems, and partition walls and sliding doors was not observed.

The building was tested with two different isolation systems – both of which were vertically stiff due to the presence of sliding bearings – and in the fixed-base configuration. An integrated ceiling, partition wall, and fire sprinkler piping (CPP) system was installed in the building to assess the performance of nonstructural components during 3D ground motion. In addition, the response of unattached contents was evaluated in two enclosed rooms. Vertical accelerations at column locations were amplified relative to the shake table by factors generally less than 2, while accelerations at mid-slab were amplified by average factors of approximately 3 on the second floor to 6 on the roof. Column acceleration amplification factors were low and insensitive to the height as columns were relatively rigid and could transfer the motion with little distortion. However, mid-slab vertical accelerations were dominated by slab vibration period that ranged from 0.08 to 0.14 sec in the test building [Ryan et al., 2016]. Field observations in real buildings validated the slab vibration frequencies observed in the test structure. The vibration period of composite floor systems in the field range from 0.08 to 0.25 sec, as measured through low-level forced
vibration with an eccentric mass shaker [Hicks, 2004] or determined by expert opinion [Murray et al., 1997 and Allen and Pernica, 1998].

The large vertical accelerations measured in the building led to CPP component damage. This damage was qualitatively evaluated by inspection of all available video footage, and observed damage states were compared to recorded peak horizontal floor and vertical slab accelerations parameters for every earthquake simulation. Damage was shown to be closely related to vertical accelerations recorded in the center of the floor slabs. Damage to CPP was classified as slight, moderate and extensive for vertical accelerations from 2-3g, 3-5g, and above 5g, respectively.

Despite these recent experiments, significant damage to nonstructural components in isolated buildings during past earthquakes has not been reported. However, vertical response generally has not been measured to determine whether vertical accelerations have been significant. In fact, most instrumented buildings do not include the right kind of sensors to measure the slab vibration [et al., 2016]. Bozorgnia et al. [1998] examined 12 buildings shaken in the 1994 Northridge earthquake (both base-isolated and conventional) that all included some instrumentation to measure vertical shaking. Three of these buildings contained vertical accelerometers both at the ground level and on the roof slab away from a column. The ratios of peak vertical accelerations recorded in the roof to those at the ground level were 3.75, 6.4, and 2.4. In addition, Lew and Hudson [1999] examined several instrumented buildings and observed that vertical accelerations transmitted from the ground to the building were not affected by the presence of an isolation system.
Researchers have begun to consider strategies to mitigate vertical acceleration amplification due to the vertical component of ground motion. Guzman and Ryan [2017] evaluated parameters that influence the vertical response of the floor system, including mass eccentricities, modeling assumptions of the isolators, and slab flexibility. A hypothetical three-story building with and without base isolation was investigated. Most floor system parameter variations led to slightly altered vertical modal frequencies, and these modified frequencies had an unpredictable influence on slab amplification factors. Thus, modifying the slab design is unlikely to be an effective mitigation strategy.

Researchers have proposed 3D isolation systems by either combining independent devices for horizontal and vertical isolation in series or developing 3D isolation devices to achieve better performance [Inoue et al., 2004]. Many researchers developed and tested different systems for 3D isolation [Suhara et al., 2003, Kageyama et al., 2003; Takahashi et al., 2008; Xu et al., 2012a and 2012b and Li et al., 2013]. The proposed systems have vertical isolation periods on the order of 1-2 seconds, and generally utilize dampers and rocking suppression devices to control both vertical and rocking displacements.

The authors hypothesize that effective 3D isolation can be achieved with lower vertical isolation periods than considered in these aforementioned studies, thus potentially eliminating the problems associated with rocking and large vertical displacement. The fundamental dynamic response of planar structures with horizontal and vertical isolation using a simplified 2D model of a rigid block resting on isolation bearings was explored previously (Chapter 2). A parametric study was carried out to evaluate the effect of different site conditions, structure properties and isolation parameters on the structure and isolator responses. The results showed that the horizontal ($T_H$) and vertical isolation period
(TV) should be well separated, and TH should be selected to be much longer than TV. The 2D rigid block analysis showed that vertical accelerations are greatly reduced with TV as low as 0.5 sec. Also, the rigid block model was shown to be a good simplified model to explore the fundamental dynamic behavior of a 3D isolated structure subjected to horizontal and vertical ground motion excitation. However, the rigid block model does not account for the effect of superstructure stiffness, and thus is less accurate for flexible superstructures. These findings are corroborated by Zhou et al. [2016], which examined the dynamic performance of several vertical and 3D isolation systems for potential application to modern nuclear facilities. An isolation system with vertical period TV = 0.33 sec was found to be feasible for the nuclear power plant model examined, and could effectively reduce the vertical in-structure responses. Also, Zhou concluded that the rocking effect was obvious when TV increased to 1.0 sec, and vertical isolator displacements were at least as large as the horizontal displacements for TV = 2.3 sec.

Building on the promising results from the rigid block idealization, the objective of this study is to evaluate the structural response and mitigation of vertical vibration in realistic flexible buildings with 3D isolation. Three hypothetical steel buildings with different aspect ratios that use special concentric braced-frame lateral systems have been designed for this purpose. The seismic response of the buildings with horizontal (Hz) and 3D isolation are compared. The effect of 3D isolation in mitigating the effect of vertical ground shaking is investigated, while qualifying the potential increase in horizontal response parameters (drift, acceleration) that should be considered in design. Also, responses determined by a 3D rigid block model are compared with the flexible building response to identify the limitations of the rigid block in predicting both global and local
structural responses. Finally, the influence of aspect ratio on the structural response of the building with 3D isolation is evaluated.

4.2 **Target Spectra and Ground Motion Scaling**

A suite of 12 ground motions was developed for numerical analyses to evaluate 3D isolation for buildings with flexible floor systems. The same suite has been used for the previous mentioned exploratory studies on 3D isolation in Chapter 2 and 3. The motions were selected and scaled to represent target strong horizontal and vertical shaking in a near fault region. Target spectra were developed for a hypothetical location in greater Los Angeles area on site class D soil. This site has Mapped Risk-Targeted Maximum Considered Earthquake MCER, 5% damped spectral acceleration of $S_S = 2.11g$ at short periods and $S_I = 0.74g$ at 1.0 sec period. The authors have used ground motions associated with these site parameters for previous studies on seismic isolation [Sayani et al., 2010; Erduran et al. 2011 and Cutfield et al., 2016]. The MCE horizontal spectral accelerations were calculated according to ASCE 7-16 [ASCE, 2016]. Little guidance is available on the selection and scaling of vertical ground motions; however, FEMA-P750 (Section 23) [NEHRP, 2009] published guidance to develop a target spectrum in the vertical direction from the corresponding horizontal spectrum. The intensity of vertical shaking is determined by the coefficient $C_V$, which depends on $S_S$ and site class. In the short period range, vertical spectral acceleration is $0.8 \cdot C_V S_S$, so the peak vertical to horizontal spectral acceleration (V/H) ratio is observed to be $0.8 \cdot C_V$. For site class D, $C_V$ varies from 0.7 for $S_S \leq 0.2$ g to 1.5 for $S_S \geq 2.0$ g; consequently, the V/H ratio ranges from 0.56 to 1.2. V/H $= 1.2$ for the selected site, which represents the largest intensity vertical hazard. The target spectra are illustrated previously in Chapter (2) and (3).
With the intent that vertical shaking be realistically represented, ground motions with relatively large vertical components were hand selected and scaled to represent the target spectra. Initially, 30 recorded motions were selected from the PEER NGA database [Chiou et al., 2008] by identifying ground motion triplets that by inspection best matched the shape and relative intensity of the components of the target spectra. These motions were amplitude-scaled to minimize the sum of the squared error between the response spectrum of the ground motion component and the target spectrum in each direction. The error was minimized over periods ranging from 1.5 to 4.0 sec in both horizontal directions, and from 0 to 2.0 sec in the vertical direction, which are considered to be relevant operating period ranges for buildings with Hz or 3D isolation systems. Both horizontal and vertical components were scaled by a single scale factor to preserve the relative component amplitudes of the hand selected motions. These 30 motions were then filtered to select a set of 12 motions to be used in the analysis. First, motions requiring a scale factor > 3.0 were removed to exclude ground motion distortion effects that may come from excessive scaling. Then, multiple attempts were made by trial and error to choose a smaller set of 12 motions with median spectra that matched the target spectra in two horizontal as well as vertical directions.

Selected ground motions and the calculated scale factors were summarized previously in Chapter (2) and (3). For each component (X, Y, Z), acceleration spectra for individual motions along with the median over the 12 motions are plotted against the target MCE\(_R\) horizontal and vertical spectra in Figure 4-1. Due to inconsistencies between relative component amplitudes of recorded and target spectra, the median spectrum of the scaled ground motions falls short of the target spectrum in the vertical direction. However, several
of the scaled ground motions reach very high intensity spectral accelerations in the short period range in both horizontal and vertical directions when scaled to match the long period region of the spectrum.

![Acceleration response spectra for 12 scaled ground motions, median spectrum, and MCE\(_R\) target spectrum](image)

**Figure 4-1:** Acceleration response spectra for 12 scaled ground motions, median spectrum, and MCE\(_R\) target spectrum (a) X–component, (b) Y–component and (c) Z–component

### 4.3 Design and Modeling of Case Study Buildings

#### 4.3.1 Design Procedure

Hypothetical steel special concentric braced-frame buildings were designed for this study to benchmark the performance of flexible buildings with Hz and 3D isolation systems subjected to 3D ground excitations. The buildings were designed for office occupancy. Three buildings with different structure height/width (h/b), or aspect ratios, were studied. All buildings are 6-story tall, multi-bay plane frame buildings, with story height = 5.0 m and bay width = 7.5 m. Different aspect ratios were achieved by varying the number of
bays in the three buildings. Bld. I – 6x6 with 6 bays by 6 bays is classified as wide, with low aspect ratio h/b = 0.67. Bld. III – 2x2, representing a slender building, with h/b = 2.0, was achieved by decreasing to 2 bays by 2 bays, while Bld. II – 3x3 had intermediate slenderness with 3 bays by 3 bays and h/b = 1.34.

Design loads were calculated according to ASCE 7-16 [ASCE, 2016]. The assumed self-weight of framing and composite floor section including corrugated sheets and concrete topping was 2.15 kN/m². Superimposed dead loads were 1.35 and 1.05 kN/m² while assigned live loads were 3.8 and 1.0 kN/m² for building floors and roof, respectively. An additional uniform line load = 3.5 kN/m was applied to edge girders to represent exterior wall self-weight. Seismic loads were computed using the equivalent lateral force procedure [ASCE, 2016], and the buildings were designed for the larger of the minimum lateral forces for a fixed-base building (force reduction factor R = 6) and a base-isolated building (with effective period = 4.0 sec, damping = 25%, and force reduction factor R_I = 2). Minimum lateral forces for the fixed-base building controlled the design. As such, drift limits were evaluated for the building in the fixed base configuration. The design calculations are presented in greater detail in Appendix (B).

3D frame models were developed and analyzed for these buildings using SAP2000 software. Frame sections were chosen and evaluated iteratively for Bld. I - 6x6, and the same sections were used for the other two buildings for design efficiency. Table 4-1 summarizes the designed beam, column and brace sections, which were common for all three studied buildings. Beams and columns were designed to carry gravity loads, while braces were designed to resist seismic loads. Braced bays were limited to the building
exterior walls. Bracing was applied to two bays on each exterior wall in Bld. I – 6x6, and only one bay in Bld. II – 3x3 and Bld. III – 2x2 because of the significantly reduced plan area. An extruded view of each model is presented in Figure 4-2.

Table 4-1: Selected steel sections for case study buildings

<table>
<thead>
<tr>
<th>Floor</th>
<th>Main Beam Section</th>
<th>Secondary Beam Section</th>
<th>Gravity Column Section</th>
<th>Braced Bay Column Section</th>
<th>Bracing Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof (6th)</td>
<td>W360x72</td>
<td>W360x51</td>
<td>W310x67</td>
<td>Roof (6th)</td>
<td>W360x72</td>
</tr>
<tr>
<td>5th</td>
<td>W410x100</td>
<td>W360x72</td>
<td>W310x67</td>
<td>W360x262</td>
<td>HSS152.4x152.4x12.7</td>
</tr>
<tr>
<td>3rd - 4th</td>
<td>W410x100</td>
<td>W360x72</td>
<td>W310x143</td>
<td>W360x262</td>
<td>HSS203.2x203.2x12.7</td>
</tr>
<tr>
<td>1st - 2nd</td>
<td>W410x100</td>
<td>W360x72</td>
<td>W310x158</td>
<td>W360x262</td>
<td>HSS254x254x12.7</td>
</tr>
<tr>
<td>Base</td>
<td>W610x140</td>
<td>W460x82</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 4-2: Extruded view of 3d frame models exterior framing (a) Bld. I – 6x6, (b) Bld. II – 3x3 and (c) Bld. III – 2x2
Each building design was evaluated by checking each element Demand/Capacity (D/C) ratio, which represents the element factor of safety. The calculated D/C ratios include load and resistance factors according to AISC 360-16 [AISC, 2016]. The maximum D/C ratio permitted during design was 0.95, corresponding to minimum factor of safety = 1.05. Table 4-2 summarizes models characteristics, including number of bays, number of braced bays, h/b, and percentage of elements with D/C ratio > 0.5. The percentage of elements with D/C ratio > 0.5 (factor of safety ≤ 2.0) was 70%, 50% and 30% in Bld. I – 6x6 and Bld. II – 3x3, and Bld. III – 2x2, respectively (Table 4-2). The designs were considered to be economical if factor of safety ≥ 2.0 for no more than 50% of the model elements; thus, Bld. III – 2x2 is over-designed but still compliant. Also, story drift ratios due to applied lateral loads – with deflection amplification factor $C_d = 5.0$ to account for inelastic demands – were evaluated against the 2% drift limit per ASCE 7-16 [ASCE, 2016], as summarized in Table 4-3. The building designs are also compliant for the 1.5% story drift limit for isolated buildings.

Table 4-2: Design check of the frame models in the fixed-base configuration

<table>
<thead>
<tr>
<th>Building</th>
<th>No. of Bays</th>
<th>No. of braced bays each direction</th>
<th>h/b</th>
<th>Percentage of Frame Elements with D/C Ratio from 0.5 to 0.95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bld. I – 6x6</td>
<td>6x6</td>
<td>4</td>
<td>0.67</td>
<td>70%</td>
</tr>
<tr>
<td>Bld. II – 3x3</td>
<td>3x3</td>
<td>2</td>
<td>1.34</td>
<td>50%</td>
</tr>
<tr>
<td>Bld. III – 2x2</td>
<td>2x2</td>
<td>2</td>
<td>2.0</td>
<td>30%</td>
</tr>
</tbody>
</table>
Table 4-3: Building story drift ratio demands in fixed-base configuration

<table>
<thead>
<tr>
<th>Story</th>
<th>Model Drift Ratio (%)</th>
<th>Bld. I – 6x6</th>
<th>Bld. II – 3x3</th>
<th>Bld. III – 2x2</th>
</tr>
</thead>
<tbody>
<tr>
<td>6th</td>
<td>0.6</td>
<td>0.3</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>5th</td>
<td>0.7</td>
<td>0.4</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>4th</td>
<td>0.9</td>
<td>0.5</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>3rd</td>
<td>1.0</td>
<td>0.5</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>2nd</td>
<td>0.9</td>
<td>0.4</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>0.5</td>
<td>0.3</td>
<td>0.1</td>
<td></td>
</tr>
</tbody>
</table>

4.3.2 Modeling Assumptions

Computational frame models were developed for each building in the fixed-base, Hz isolated, and 3D isolated configuration. The basic superstructure frame model (common to all configurations) used linear steel frame elements for beams, columns and braces and shell elements for slabs. Shell elements with thickness = 0.1 m were used to model the composite floor section including corrugated sheets and concrete topping. These shell elements were divided into a small mesh with 0.5 m x 0.5 m element size to distribute the mass uniformly over the slab. The shell elements were connected to the top of the beam sections so that the beam-slab composite stiffness was simulated correctly. All beam, column, and brace connections were assigned to be perfectly pinned.

An eigenvalue analysis of the fixed-base models using Ritz vectors was performed to quantify the dynamic behavior. The fundamental periods of the three models are summarized in Table 4-4 for translation in X, Y and Z directions and torsion. The building horizontal period decreases (i.e. relative stiffness increases) with increasing slenderness ratio as shown in Table 4-4. This is contrary to the expectation that for a given height,
flexibility (as measured by the natural period) would increase with building slenderness. The decrease in period with increasing slenderness is an artifact of the particular design assumptions used here; i.e. lateral resistance reduced only by factor of 2 (1 braced bay vs. 2 braced bays per exterior side) for Bld. II – 3x3 and Bld. III – 2x2, which have 1/4 and 1/9, respectively, of the original plan area of Bld. I – 6x6. However, the laterally stiff building designs are suitable for a meaningful evaluation of Hz versus 3D isolation. Since the vertical vibration characteristics are controlled by slab vibration, all buildings have about the same vertical period \( \approx 0.16 \) sec, which is within the range observed in the literature [Murray et al., 1997; Allen and Pernica, 1998 and Hicks, 2004].

<table>
<thead>
<tr>
<th>Bld.</th>
<th>Translation Period (sec)</th>
<th>Torsional Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>I – 6x6</td>
<td>0.93</td>
<td>0.93</td>
</tr>
<tr>
<td>II – 3x3</td>
<td>0.71</td>
<td>0.71</td>
</tr>
<tr>
<td>III – 2x2</td>
<td>0.52</td>
<td>0.52</td>
</tr>
</tbody>
</table>

In the building with Hz isolation, isolator response was represented by bilinear force-deformation in the horizontal direction, such as provided by elastomeric or rubber bearings. The horizontal force-deformation parameters were selected to achieve effective horizontal period, \( T_{eff,H,R} = 4 \) sec and effective damping ratio, \( \xi_{eff,H,R} = 25\% \) at the maximum displacement, \( D_{Max,H,R} = 0.7 \) m. The yield displacement was also selected as \( D_{Y,R} = 0.01 \) m. The bearing response was assumed to be linear in the vertical direction, with stiffness and damping calibrated to achieve vertical period \( T_{V,R} = 0.05 \) sec and damping ratio \( \xi_{V,R} = 5\% \). The desired response was represented in SAP 2000 by nonlinear rubber isolator link.
elements; these elements account for bidirectional coupling in the two horizontal directions as well as linear vertical properties.

Each isolator in the 3D-isolated building was modeled by the same rubber isolator (elastomeric bearing) combined in series with a multi-directional spring-dashpot to represent the vertical isolation device. The vertical isolation could be provided by a vertically oriented liquid spring – damper (LS) that transmits the lateral shear, which has been investigated by the authors [Cesmeci et al., 2018]. The vertical LS had stiffness computed based on vertical period $T_{\text{eff,V,LS}} = 0.5$ sec and viscous damping $\xi_{V,LS} = 20\%$. The horizontal response across the LS was represented by linear spring/dashpot calibrated to period $T_{H,LS} = 0.05$ sec and damping ratio $\xi_{H,LS} = 5\%$. Each vertical isolator was modeled by elastic link elements to represent the spring/dashpot properties in each direction. Isolation system properties were distributed uniformly to the link elements assigned at each column. For all simulations, modal fast nonlinear analysis (FNA) was used to simulate nonlinear modal history analysis, and a damping ratio of 5% was assigned to each mode.

4.4 Rigid Block Model

4.4.1 Modeling Assumptions

For this study, a 3D rigid block model was developed for cross validation of the global response predictions (e.g., isolator displacement, roof acceleration and roof drift ratio) of the flexible frame model, and to identity the limitations of the rigid block modeling approach. Previously, the authors developed a simplified 2D rigid block model to understand the fundamental dynamics of combined horizontal and vertical isolation in Chapter (2). This rigid block was validated using a 5-story single bay plane frame model
developed and analyzed using SAP2000. The rigid block model was shown to be a good simplified model to understand the fundamental dynamic behavior of a 2D isolated plane frame structure subjected to horizontal and vertical ground motion excitation. The rotation \( \theta \) in rigid block model was an effective “first order” prediction of the roof drift. However, the rigid block model doesn’t account for variation in superstructure stiffness, and increase in the frame flexibility slightly increased the error between the rigid block and frame model. The influence of the superstructure flexibility was more significant for shorter horizontal isolation periods.

3D analysis introduces additional complexity to the structure behavior, and leads to additional observed responses such as torsional response and slab vibration. To extend the previous 2D rigid block simulation, the response of a 3D rigid block model was compared to the response of Bld. III – 2x2; both had aspect ratios \( h/b = 2.0 \). The 3D rigid block was modeled and analyzed using SAP2000 to simulate the six fundamental degrees of freedom: displacements \( U_X \), \( U_Y \) (horizontal) and \( U_Z \) (vertical), and rotations about each axis center of mass (CM), including \( R_X \) and \( R_Y \) (rocking), and \( R_Z \) (torsion). The CM joint was constrained to the supporting joints at the bearings to enforce rigid block behavior. The block has mass \( m \) and moment of inertia \( I_\theta \) lumped at the CM, assumed to be the block geometric center. The lumped mass was assigned to be equivalent to the total mass of Bld. III – 2x2 flexible model. The isolators for the rigid block model (both Hz and 3D isolation) were modeled using the same number, distribution, element types and nonlinear isolation properties as for Bld. III – 2x2. The developed 3D rigid block represented only the geometry of the building, but ignored flexibility of beams, columns, and floor slabs. By
comparison, the rigid block studied previously considered only 2D motion (horizontal and vertical) and linear isolators placed only at the block edges.

4.4.2 Evaluation of Rigid Block Modeling Approach

A single ground motion – 1994 Northridge @ LA Sepulveda (#5) – scaled to match the target spectra was applied to both the rigid block and Bld. III models to compare their responses. Response histories of X-direction roof drift ratio (in percent), vertical displacement at the corner bearing, and roof vertical acceleration at corner column and mid-slab are compared for the rigid block and multi-story frame models in Figure 4-3 for the assumptions of Hz and 3D isolation. For Bld. III – 2x2, the studied mid-slab is located in the middle of one of the four corner slabs and the same point was assigned in the rigid block model. X-direction roof drift was selected as the stronger component of shaking was applied in the X-direction. For the rigid block model, the roof drift ratio was computed as the ratio of relative horizontal X-direction displacement to the block height, which simplifies to the rocking rotation component $\theta_Y$:

$$\text{Drift ratio} = \frac{(U_{X\text{Top}} - U_{X\text{Bottom}})}{h} = \frac{(U_X - \frac{h}{2} \theta_Y) - (U_X + \frac{h}{2} \theta_Y)}{h} = -\theta_Y \quad \text{Eq. 4-1}$$

In Eq. 4-1, $U_{X\text{Top}}$ and $U_{X\text{Bottom}}$ are the horizontal displacements at the top and bottom of the rigid block, respectively, both taken at the plan center of the block. For Bld. III – 2x2 model, the drift ratio was computed as total relative superstructure displacement from base to roof over total height, evaluated at the building centerline.

Compared to Bld. III – 2x2, the rigid block model – by inspection – predicts with reasonable accuracy the responses of the 3D isolated building except for the roof vertical acceleration at mid-slab. The rigid block under predicts the mid-slab acceleration at the
roof for both Hz and 3D isolation, as the roof acceleration is influenced significantly by slab vibration, which is not captured by the rigid block model. The discrepancy between the rigid block model and Bld. III – 2x2 is greater for Hz isolation, since the rigid block model only represents the relatively short period associated with the vertical stiffness of the isolators. For 3D isolation, on the other hand, the dominant vertical vibration period is determined by the vertical isolators, which the rigid block can feasibly replicate. The rigid block model somewhat underestimates the drift ratio for both Hz and 3D isolation. In summary, the simplified rigid block model estimates reasonably well global structural responses such as roof drift ratio, isolator vertical displacement, and roof column vertical acceleration; but the rigid block model does not account for slab vibrations.

Figure 4-3: Time history results for rigid block model and Bld. III – 2x2 with Hz isolation and 3D isolation for (a) Corner bearing vertical displacement, (b) Drift ratio, (c) Column vertical acceleration at the roof and (d) Mid-slab vertical acceleration at the roof.

Error measures were calculated for the applied ground motion to better quantify the error in the rigid block model relative to Bld. III – 2x2 model. The normalized root mean square deviation (NRMSD), is the RMSD normalized by max peak-to-peak response of Bld. III – 2x2:
RMSD = \sqrt{\frac{1}{N} \sum_{i=1}^{N} (r_{i,B} - r_{i,M})^2} \quad \text{Eq. 4-2}

\text{NRMSD} = \frac{\text{RMSD}}{r_{max,M} - r_{min,M}} \quad \text{Eq. 4-3}

where\ r = \text{response quantity of interest (drift, vertical displacement, acceleration)},\ i = i^{th}\ \text{step},\ N = \text{number of steps},\ B\ \text{refers to rigid block and M refers to the Bld. III – 2x2 multi-story frame model. NRMSD is a measure of average relative error in the response of the rigid block relative to Bld. III – 2x2 over the history. The peak error, or relative error in the peak response of the rigid block relative to Bld. III – 2x2, was computed as:}

\text{Peak Error} = \frac{\max_i |r_{i,B}| - \max_i |r_{i,M}|}{\max_i |r_{i,M}|} \quad \text{Eq. 4-4}

Table 4-5 summarizes the NRMSD and peak error obtained for the same responses evaluated earlier (Figure 4-3). A negative peak error indicates that the rigid block model underestimates the peak response relative to the Bld. III – 2x2 model. The error analysis (Table 4-5) confirms that the 3D rigid block analysis is more accurate for 3D isolation than Hz isolation, for all compared responses except drift ratio. In the prior 2D rigid block analysis done in Chapter (2), the peak error never exceeded 25\% and NRMSD never exceeded 6\% for any response quantity when simulating 3D isolation (Tv \geq 0.5 \text{ sec}). The peak error and NRMSD are found to be higher for a 3D rigid block compared to a 2D rigid block due to the application of bidirectional horizontal shaking and the assumed nonlinear isolation properties, which was confirmed by additional analyses not presented here. The 3D rigid block model overestimates both vertical displacement at the bearing and roof vertical acceleration at column, but not excessively for a 3D isolated building (peak error \approx 20\% in column vertical acceleration). The substantial underestimation of roof drift ratio
(peak error ≈ 50% for 3D isolation) is due to the bidirectional coupling between horizontal translation and rocking, which increases the overall rocking effect. The error in the roof vertical acceleration at mid-slab is large (peak underestimated by about 80%) and is not much affected by type of isolation system. In summary, the 3D rigid block can give good prediction of bearing vertical displacement and roof vertical acceleration at columns, but notably underestimates the drift ratio and especially roof vertical acceleration at mid-slab. Evaluation of a full flexible building model is necessary to determine if a specific application of 3D isolation will effectively mitigate the vertical response.

<table>
<thead>
<tr>
<th>Structure Response</th>
<th>NRMSD (%)</th>
<th>Peak Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hz</td>
<td>3D</td>
</tr>
<tr>
<td>Corner Bearing Vertical Displacement</td>
<td>13.1</td>
<td>10.6</td>
</tr>
<tr>
<td>Drift ratio</td>
<td>9.3</td>
<td>9.7</td>
</tr>
<tr>
<td>Roof Vertical Acceleration at Column</td>
<td>13.0</td>
<td>6.2</td>
</tr>
<tr>
<td>Roof Vertical Acceleration at Mid-Slab</td>
<td>6.0</td>
<td>5.6</td>
</tr>
</tbody>
</table>

### 4.5 Response of Flexible Frame Building with 3D Isolation

The three designed and developed models for hypothetical buildings were analyzed with fixed base, Hz isolation and 3D isolation. The suite of twelve ground motions scaled to match the target spectra was applied to the computational frame models. The following responses were evaluated: story drift ratios and horizontal floor accelerations over the structure height, and select vertical accelerations at the roof level. Responses were evaluated at four locations in plan for each building, as indicated in Figure 4-4. For Bld. I – 6x6 and Bld. II – 3x3, the locations include mid-slab point (P), two adjacent column
points C1 and C2 and corner point C3. Bld. III – 2x2 was limited to 3 locations as the corner point C3 coincides with C2. Vertical accelerations were computed at the roof level at these locations, while story drifts and horizontal accelerations at all floors were computed based on the corner location. Horizontal responses were evaluated in the X-direction, which was subjected to the stronger component of shaking. The response of the fixed-base building was evaluated for reference. However, the ground motion scaling assumptions led to excessive short period horizontal spectral accelerations for some motions (Figure 4-1), and the nonlinear response of the braced frame lateral system was not simulated. For these reasons, the simulated horizontal responses of the fixed-base building are not presented, as they would be misleading. The simulated vertical accelerations of the fixed-base building are considered an acceptable reference against which to evaluate vertical response in the Hz and 3D isolated buildings.

![Figure 4-4](image.png)

Figure 4-4: Locations for evaluating accelerations at the roof for (a) Bld. I – 6x6, (b) Bld. II – 3x3 and (c) Bld. III – 6x6

4.5.1 Effect of Horizontal and 3D Seismic Isolation

First, the 1994 Northridge@ LA Sepulveda motion scaled to the target spectra was applied to Bld. I – 6x6 to investigate Hz vs 3D isolation. Histories of X-direction horizontal
isolator displacement and roof acceleration, as well as drift ratio (in percent) at column C3 for Hz and 3D isolation are presented in Figure 4-5. 3D isolation causes small increase in isolator displacement and roof acceleration relative to Hz isolation, while the drift ratio is approximately doubled.

![Figure 4-5: Response history results for Bld. I at column C3 for fixed base, Hz isolation and 3D isolation in X-direction for ground motion (#5) (a) Bearing horizontal displacement, (b) Roof horizontal acceleration and (c) Roof drift ratio](image)

The histories of roof vertical displacement and acceleration at column C1 for fixed base, Hz isolated and 3D isolated buildings, due to the same motion (LA Sepulveda) are presented in Figure 4-6. The peak vertical acceleration is 1.5, 2.0 and 0.7 g for fixed-base, Hz isolation and 3D isolation, respectively, compared to the vertical peak ground acceleration (PGAz) of 0.7 g. Vertical acceleration was amplified at the roof for both fixed-base and Hz isolation. The vertical acceleration increased with Hz isolation compared to fixed-base, which was likely due to the assumed vertical flexibility of the isolation bearings. 3D isolation limited the roof vertical acceleration at the column location to the PGAz, while the 3D isolation system would be designed to accommodate the increased vertical displacement.
Next, the horizontal responses of Bld. I – 6x6 were investigated and compared for Hz and 3D isolation. Peak values of X-direction story drift ratio and horizontal acceleration at C3 over the structure height due to the applied twelve ground motions and the median over the motions are presented in Figure 4-7. Both peak story drift and horizontal acceleration increase for 3D isolation relative to Hz isolation due to the rocking effect. The potential increase in the drift ratio is more significant; median story drift ratio approximately doubles with 3D isolation (Figure 4-7 (b)). However, the drifts are still small relative to a fixed-base condition and thus 3D isolation as implemented still allows for a significant mitigation of horizontal response.
Spectral analysis with was carried out for select accelerations alongside a comparative modal analysis of the Bld. I – 6x6 models with Hz and 3D isolation to better quantify dynamic response characteristics. The modal participation factors for effective modes are summarized in Table 4-6 for models with Hz and 3D isolation. The 5% damped horizontal spectral accelerations at roof column C3 for Hz and 3D isolation for the individual ground motions and the median are presented in Figure 4-8. Both Hz and 3D isolation led to similar horizontal spectral acceleration with nearly aligned peaks, which provides further evidence that adding vertical isolation does not alter the dynamic characteristics that support the attenuation of horizontal acceleration. Peaks associated with first isolation mode cannot be identified in the spectral response, however some rises appear at 3.5-5 sec for some ground motions. Peak horizontal spectral acceleration occurs at lower periods that align with the structural modes as shown in Table 4-6. These horizontal spectral acceleration peaks occur relatively close to 1\textsuperscript{st} and 2\textsuperscript{nd} structural modes in the X-direction, which are at 0.64 sec (Mode 5) and 0.24 sec (Mode 8) for Hz isolation.
(line a and b), and 0.85 sec (Mode 5) and 0.25 sec (Mode 17) for 3D isolation (line c and d). The period of the 1\textsuperscript{st} structural mode depends somewhat on the effective (activation) period of the isolation system, and thus varies from motion to motion.

Figure 4-8: Roof horizontal spectral acceleration in X-direction for column C3 in Bld. I for Hz and 3D isolation

Table 4-6: Modal Participation factors for Bld. I with Hz isolation

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period (sec)</th>
<th>Translation</th>
<th>Mode</th>
<th>Period (sec)</th>
<th>Translation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hz Isolation</td>
<td></td>
<td></td>
<td>3D Isolation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
<td>Z</td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>1</td>
<td>4.10</td>
<td>1.00</td>
<td>0</td>
<td>1</td>
<td>4.08</td>
</tr>
<tr>
<td>2</td>
<td>4.10</td>
<td>1.00</td>
<td>0</td>
<td>2</td>
<td>4.03</td>
</tr>
<tr>
<td>4</td>
<td>0.64</td>
<td>9.7E-4</td>
<td>0</td>
<td>4</td>
<td>0.87</td>
</tr>
<tr>
<td>5</td>
<td>0.64</td>
<td>9.6E-4</td>
<td>0</td>
<td>5</td>
<td>0.84</td>
</tr>
<tr>
<td>7</td>
<td>0.24</td>
<td>1.7E-5</td>
<td>0</td>
<td>6</td>
<td>0.53</td>
</tr>
<tr>
<td>8</td>
<td>0.24</td>
<td>1.7E-5</td>
<td>0</td>
<td>9</td>
<td>0.46</td>
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<tr>
<td>9</td>
<td>0.18</td>
<td>0</td>
<td>0.47</td>
<td>11</td>
<td>0.36</td>
</tr>
<tr>
<td>10</td>
<td>0.18</td>
<td>0</td>
<td>7.4E-6</td>
<td>12</td>
<td>0.31</td>
</tr>
<tr>
<td>12</td>
<td>0.18</td>
<td>0</td>
<td>0.056</td>
<td>16</td>
<td>0.25</td>
</tr>
<tr>
<td>19</td>
<td>0.15</td>
<td>0</td>
<td>0.013</td>
<td>17</td>
<td>0.25</td>
</tr>
<tr>
<td>24</td>
<td>0.13</td>
<td>0</td>
<td>0.09</td>
<td>25</td>
<td>0.16</td>
</tr>
</tbody>
</table>
Horizontal roof acceleration was evaluated at a single point because the floor moves rigidly in plan. However, vertical acceleration and displacement vary significantly over the floor plan due to slab vibration, as the slab bending resistance is relatively low compared to the column vertical stiffness. Vertical spectral acceleration for points P (at mid-slab), and C1 and C2 (directly over columns) for Hz and 3D isolation due to individual ground motions and the median are presented in Figure 4-9. For Hz isolation, the spectral acceleration peak at the column occurs at 0.18 sec and aligns with the dominant vertical mode (Mode 9 from Table 4-6). However, mid-slab point (P) peaks over the period range from 0.13-0.18 sec, which suggests that a number of local vertical modes are contributing (also identified in Table 4-6). The range of these modes is shown by vertical line in Figure 4-9 (a). This is consistent with prior observation [Ryan et al., 2016] that multiple slab vibration modes influence the response. As a result, peak vertical acceleration is amplified mid-slab relative to the columns by approximately 250%, which is consistent with prior experimental observation.

A fundamental contribution of the present work is to verify that 3D isolation with the relatively short isolation period (0.5 sec) is adequate to significantly mitigate the vertical acceleration in a flexible frame building, including the amplification at mid-slab relative to adjacent columns. For 3D isolation, vertical spectral acceleration peaks are distributed over period range from 0.16 to 0.5 sec (Figure 4-9 (b)). These peaks occur relatively close the fundamental modes associated with overall vertical movement of the isolators, some of which have substantial rocking. These modes are at 0.53 sec (Mode 6), 0.46-0.31 sec (Mode 9, 11 and 12), and 0.16 sec (Mode 25) for 3D isolation as indicated by vertical lines in Figure 4-9 (b). This period lengthening substantially reduces the overall intensity of
vibration and the amplification of vertical acceleration from column to mid-slab. Peak column and slab accelerations are 2.0 and 5.8 g for Hz isolation (amplification factor ≈ 3.0), and only 1.0 and 1.5 g for 3D isolation (amplification factor ≈ 1.5). As a result, the peak vertical acceleration in the slabs is reduced to below 2g, which should be sufficient to prevent nonstructural component damage according to Ryan et al. [2016].

Figure 4-9: Roof vertical spectral acceleration for column C1, C2 and mid-slab P in Bld. I – 6x6 (a) Hz isolation and (b) 3D isolation

4.5.2 Effect of Structure Aspect Ratio

Recall that buildings with different aspect ratio have been considered in this study to investigate the effect of building slenderness on the effectiveness of 3D isolation. The analysis presented thus far has demonstrated that 3D isolation is effective for the most squat Bld. I – 6x6 with lowest aspect ratio h/b = 0.67, but rocking could still be an impediment for 3D isolation in a more slender building. Recall that the design concept applied in this study was to design Bld. I – 6x6 for code compliance, and apply the same sections for the other two buildings with reduced bays: Bld. II – 3x3 (h/b = 1.33) and Bld. II – 2x2 (h/b = 2.0), which actually led to increased stiffness (reduced fundamental period) with increasing aspect ratio (Table 4-4).
The response trends of the three varying aspect ratio buildings with 3D isolation are compared. The medians over the ground motions of the peak values of X-direction story drift ratio and horizontal floor acceleration over the structure height (sampled at the corner column C3 in Bld. I – 6x6 and Bld. II – 3x3 and at C2 in Bld. III – 2x2) are presented in Figure 4-10. Horizontal acceleration is compared to the median PGA for reference. Peak horizontal floor acceleration is found to be independent of the structure aspect ratio, while for all buildings 3D isolation suppresses the horizontal acceleration well below the PGA (Figure 4-10 (a)). In spite of the increasing aspect ratio, the story drift ratio decreases for both Bld. II – 3x3 and Bld. III – 2x2 relative to Bld. I – 6x6, which is due to the decrease in structure period. The drift is well below the ASCE limit of 1.5% for isolated structures. This suggests that regardless of aspect ratio, stiff superstructures that are good candidates for conventional isolation are also suitable for 3D isolation. However, without further investigation this observation is restricted to the relatively stiff 6-story buildings considered in the study, and may not be applicable to the class of buildings with substantially increased height, which is naturally accompanied by significant period lengthening.
Figure 4-10: Peak response profile over building height in X-direction for Bld. I – 6x6, II – 3x3 and III – 2x2 with 3D isolation (a) Horizontal acceleration and (b) Drift ratio

For further insight, spectral analysis was carried out for the X-direction roof horizontal acceleration at the corner column, as and for vertical acceleration at the roof level mid-slab (point P), and column C1 and C2 locations, and the median over the twelve ground motions for the different buildings is presented in Figure 4-11. The floor horizontal spectral acceleration is again essentially independent of aspect ratio as shown in Figure 4-11 (a). Two spectral peaks can be identified for all buildings: one at the period associated with 1st structural mode 5 (Table 4-6), which is almost constant for different aspect ratios, and another at period associated with 2nd structural mode 17, which differs with aspect ratio according to the change in the fixed-base natural period.

Vertical spectral acceleration peaks are distributed over the period range from 0.14 to 0.5 sec with only minor variations for different aspect ratios as shown in Figure 4-11 (b). In general, the intensity of slab vibration is essentially insensitive to aspect ratio. Therefore, 3D isolation of a slender building is not problematic as long as it is designed with sufficient period shift in the horizontal direction. The design approach used in this
study resulted in increased stiffness (reduced period) as the slenderness ratio increased, which consequently led to improve response for buildings with increased slenderness ratio.

![Graphs showing spectral acceleration](image)

Figure 4-11: Roof spectral acceleration in Bld. I – 6x6, II – 3x3 and III – 2x2 for 3D isolation (a) Horizontal in X-direction at corner column and (b) Vertical for column C1, C2 and mid-slab P

### 4.6 Conclusion

Three hypothetical steel special concentric braced-frame buildings were designed for this study to benchmark the performance of flexible frame buildings with horizontal and 3D isolation systems subjected to 3D ground excitations. Different structure height/width (h/b), or aspect ratios, were achieved by varying the number of bays in the three buildings: Bld. I – 6x6 (h/b = 0.67), Bld. II – 3x3 (h/b = 1.34), and Bld. III – 2x2 (h/b = 2.0). The hypothetical building models were analyzed with fixed base, Hz isolation and 3D isolation to a suite of twelve ground motions scaled to match the target spectra horizontal and vertical spectra. The story drift ratios and horizontal acceleration profiles over the structure height as well as roof level horizontal and vertical acceleration histories and spectra were investigated to evaluate the response of flexible frame buildings with 3D isolation. Also, a
A 3D rigid block with \( h/b = 2.0 \) was analyzed and compared to the response of Bld. III – 2x2. This study has led to the following conclusions:

- A simplified 3D rigid block model estimates reasonably well the global structure responses such as isolator vertical displacement and roof column vertical acceleration, but does not account for slab vibrations and underestimates roof drift ratio.

- On average, vertical ground shaking is amplified by factor of 2.0 in columns relative to the ground and by an additional 3.0 from the columns to the slabs in a fixed-base building. These amplification factors are even slightly increased in a Hz isolated building. However, with the 3D isolation system investigated here, these amplification factors are reduced to 1.0 and 1.5, respectively. As a result, 3D isolation reduces peak vertical accelerations in columns and slabs to below 2g, a level believed to keep nonstructural component safe.

- 3D isolation causes story drift to increase (up to double) relative to Hz isolation, but it does not compromise the mitigation of the horizontal shaking.

- 3D isolation is effective to shift the energy of vertical shaking over a wider period range (0.16 to 0.5 sec). However, the Hz isolated building sees an intense spectral peak at the slab vibration period of 0.18 sec, where the input energy of the ground motion is very high.

- Story drifts were observed to decrease as building aspect ratio increased, while horizontal floor accelerations and sampled roof accelerations (column and mid-slab) were insensitive to aspect ratio. The trend for story drift ratio was an artifact of the
design approach that resulted in increased building stiffness (reduced period) with increasing slenderness ratio.

- 3D isolation can be effectively applied to stiff superstructures that are well-suited for conventional isolation, regardless of aspect ratio. This conclusion cannot be extended to significantly taller buildings without further investigation.

4.7 Acknowledgments

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4.8 References

American Society of Civil Engineers (ASCE) [2016] "ASCE Standard – ASCE/SEI 7-16: Minimum design loads for buildings and other structures," American Society of Civil Engineers (ASCE), Reston, VA.

American Institute of Steel Construction (AISC) [2016] “AISC 360-16: Specifications for Structural Steel Buildings”, American Institute of Steel Construction (ASIC), Chicago, IL.


Chapter (5) – Summary and Conclusions

5.1 Summary

Seismic isolation is an effective technique used to mitigate effects of ground shaking and achieve higher seismic performance in building structures. Recent research has suggested that vertical excitation has significant effects on the structural response during an earthquake. In this dissertation, the dynamic response and control of buildings with 3-dimensional (3D) isolation systems is explored. First, the fundamental dynamic response of a simplified model consisting of a rigid block resting on isolation bearings at the block edges, and subjected to horizontal and vertical ground motion input was investigated. The overall structural response of the 3D isolated block was evaluated through drift ratio, vertical and horizontal displacement and acceleration amplification factors. Furthermore, a multi-story frame model was developed to validate the numerical simplified rigid block model. A parametric study was carried out to evaluate the effect of different site conditions, structure properties and 3D isolation parameters on the structural and isolator responses.

Next, a concept was proposed for 3D isolation that combines elastomeric bearings to resist horizontal ground shaking in series with bilinear liquid spring (BLS) - controllable magnetorheological fluid dampers (CMRD) to resist vertical shaking. The simplified rigid block model was extended to predict the response of the building isolated with BLS-CMRD devices under earthquake loading. BLS-CMRDs were simulated through a combination of nonlinear springs, viscous and hysteretic (semi-active) damping. A new Disp/Vel-based Control strategy was proposed that adjusts the input current according to the instantaneous vector combination of feedback displacement and velocity of the damper. Two variations
of the control strategy were explored. For Linear Current Variation, the current is activated when a threshold lower bound magnitude of the vector combination of displacement and velocity is reached, and maximum current is applied when a threshold upper bound magnitude is reached. These threshold magnitudes are modeled as ellipse surfaces with displacement as the semi-major axis and velocity as the semi-minor axis; \( D_{\text{min}} \) and \( V_{\text{min}} \) are the magnitudes of displacement and velocity for the inner ellipse or MR activation surface, while \( D_{\text{max}} \) and \( V_{\text{max}} \) are magnitudes of displacement and velocity for the outer ellipse, denoted the MR capping surface. Alternatively, the simplified ON-OFF strategy uses a single ON-OFF activation surface that triggers the maximum current to turn on when the instantaneous vector magnitude moves outside the surface and turn off when it moves back inside this surface. An optimization study was conducted to select the activation and capping surface intensities for the Linear Current Variation strategy. The proposed Linear Current Variation and ON-OFF control strategies were evaluated with reference to the well-known clipped optimal approach, Passive-ON and Passive-OFF.

Finally, hypothetical steel special concentric braced-frame buildings were designed for this study to benchmark the performance of flexible buildings with horizontal and 3D isolation systems subjected to 3D ground excitations. Three buildings with different structure height/width (h/b), or aspect ratios, were studied. Different aspect ratios were achieved by varying the number of bays in the three buildings: Bld. I – 6x6 (h/b = 0.67), Bld. II – 3x3 (h/b = 1.34), and Bld. III – 2x2 (h/b = 2.0). The hypothetical building models were analyzed with fixed base, Hz isolation and 3D isolation to a suite of twelve ground motions scaled to match the target spectra horizontal and vertical spectra. The story drift ratios and horizontal accelerations over the structure height as well as the roof horizontal
and vertical spectral acceleration were investigated to evaluate the response of flexible frame building with 3D isolation. Also, a 3D rigid block with h/b = 2.0 was analyzed and compared to the response of Bld. III – 2x2.

5.2 Conclusions

This research study has led to the following main conclusions:

- A simplified 3D rigid block model estimates reasonably well the global structure responses such as roof drift ratio, isolator vertical displacement and roof column vertical acceleration, but does not account for slab vibrations.

- According to 2D rigid block model parametric study, the acceptable range of 3D isolation periods is concluded to be $T_V = 0.5$ sec and $T_H \geq 1.5$ sec or $T_V = 1.0$ sec and $T_H \geq 4.0$ sec and $\zeta_H$ and $\zeta_V \geq 20\%$. $T_H$ and $T_V$ should not be closely coupled, and $T_H$ should be selected to be much longer than $T_V$.

- When evaluated using a flexible frame model with more realistic details, on average, vertical ground shaking is amplified by factor of 2.0 in columns relative to the ground and by an additional 3.0 from the columns to the slabs in a fixed-base building. These amplification factors are even slightly increased in a Hz isolated building. However, with the 3D isolation system investigated here, these amplification factors are reduced to 1.0 and 1.5, respectively. As a result, 3D isolation reduces peak vertical accelerations in columns and slabs to below 2g, a level believed to keep nonstructural component safe.

- 3D isolation causes story drift to increase (up to double) relative to Hz isolation, but it does not compromise the mitigation of the horizontal shaking.
• Story drifts were observed to decrease as building aspect ratio increased, while horizontal floor accelerations and sampled roof accelerations (column and mid-slab) were insensitive to aspect ratio. The trend for story drift ratio was an artifact of the design approach that resulted in increased building stiffness (reduced period) with increasing slenderness ratio.

• 3D isolation can be effectively applied to stiff superstructures that are well-suited for conventional isolation, regardless of aspect ratio. This conclusion cannot be extended to significantly taller buildings without further investigation.

• When evaluating the response of a 3D isolation system with BLS-CMRD for a vertical isolator, for design level shaking, Passive-ON reduced the vertical displacement and the overall drift ratio relative to Passive-OFF, but increased vertical acceleration demands to exceed the peak ground acceleration (PGA), which does not meet the objective of the isolation system. This shows the need for a good control strategy.

• The proposed Displ/Vel-based Control was effective for ground motions that exceeded the design level, to moderate the level of energy dissipation, keep device vertical displacement within the design stroke limit, and attenuate vertical acceleration below PGA.

• Both variations of Disp/Vel-based Control (Linear Current Variation and ON-OFF) reduced peak vertical acceleration, drift ratio, absolute negative and positive device stroke responses relative to the well-known clipped optimal control.

• Applying any control strategy substantially reduced the operation time compared to Passive-ON.
Appendix (A) - Experimental Work

As part of this research, a one-quarter scale BLS-CMRD was built and tested in the Large Scale Structure Laboratory at University of Nevada, Reno. The BLS-CMRD was designed according to the specifications in Table 3-2, which were determined based on the dynamic study to optimize fundamental parameters for 3D isolation. Device behavior was experimentally simulated to validate the Bouc-Wen model used in the computational simulation of the system, and to show that a device could be designed to provide the bilinear stiffness and controllable damping targeted in design. The test setup is illustrated in Figure A-1 [Cesmeci, 2017]. The device was connected to a hydraulic actuator for dynamic loading in the axial direction. For some cases, a constant shear loading was applied by a 5 ton hydraulic pulling ram attached overhead to a stiff frame. Complete details of the design, test setup, and results are presented in Cesmeci [2017].

The experimental program consisted of two different types of loading: controlled cyclic loading for device characterization, and earthquake simulations. Cyclic loading was applied with variable amplitude, frequencies and different input current as presented in Table A-1. This cyclic loading was used to characterize the device hysteretic response as a function of the aforementioned parameter variables. The results of these cyclic characterization tests are presented comprehensively in Cesmeci [2017], but select results are summarized here for completeness. Earthquake simulations were conducted by imposing a displacement history (or stroke history) to the device that had been predicted by simulation of the system response to an input ground motion through a Simulink model.
Table A-2 summarizes the ground excitations, intensities ranging from 100-300% of the intensity scaled to meet the target spectra, and applied current assumed in both Simulink and during the test (passive-OFF or passive-ON). The applied displacement histories of these earthquakes were originally obtained using the developed Simulink model with estimated/design parameters. Due to laboratory constraints, these simulations were conducted prior to calibrating the model to adjust for differences between predicted and observed device hysteresis response. The predicted stroke history was used as input to the device, and the generated resisting force was recorded and compared to the model.

Table A-1: Cyclic loading test matrix [Cesmeci, 2017]

<table>
<thead>
<tr>
<th>Property</th>
<th>Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amplitude (mm)</td>
<td>12.7</td>
</tr>
<tr>
<td></td>
<td>25.4</td>
</tr>
<tr>
<td>Frequency (Hz)</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
</tr>
<tr>
<td>Current, I (Amp)</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
</tbody>
</table>

Figure A-1: BLS-CMRD test setup [Cesmeci, 2017]
Table A-2: Ground excitation test matrix

<table>
<thead>
<tr>
<th>EQ</th>
<th>Earthquake Name</th>
<th>Station Name</th>
<th>Passive-OFF I = 0.5 Amp</th>
<th>Passive-OFF I = 0.5 Amp</th>
<th>Passive-OFF I = 1.0 Amp</th>
<th>Passive-OFF I = 1.0 Amp</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Northridge</td>
<td>LA - Sepulveda VA Hospital</td>
<td>100% 150% 150%</td>
<td>200% 200% 250%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Loma Prieta</td>
<td>LGPC</td>
<td>100% 150% 150%</td>
<td>200% 200% 250%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Chi-Chi, Taiwan</td>
<td>TCU079</td>
<td>100% 150% 150%</td>
<td>250% 250% 300%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Characterization of the device done using the cyclic loading shows that BLS-CMRD has modified $K_C = 7668.5$ kN/m, and $K_T = 48417.5$ kN/m. The stiffnesses obtained from experimental results were higher than the designed values, which were 6031.8 kN/m and 25671 kN/m for compression and tension, respectively. The increase in the $K_C$ was approximately 30%; however, the $K_T$ nearly doubled. Also, the ratio between $K_T/K_C$ increased to 6, while the design ratio was 4. The discrepancy between the design and realized tension stiffness was concluded to be a result of entrapped air in the MR fluid [Cesmeci, 2017]. As the shaft moves further into the top chamber, it first compresses tiny air bubbles and then squeezes the MR fluid. Therefore, the realized fluid volume is less than the design volume. Regarding the compression stiffness, the bulk modulus for the silicone oil filling the bottom chamber was assumed to be 1.13 GPa from the literature, while the actual bulk modulus was found to be 1.40 GPa [Cesmeci, 2017].

Experimental force-displacement hysteresis loops for the device under cyclic loading are compared with simulated hysteresis response using both a pre-test model (design parameters) and a modified model (model parameters adjusted to match the experimentally
observed. These comparisons are presented for cyclic loading with amplitude 12.7 mm, in Figure A-2 for frequency = 2 Hz and no input current, and Figure A-3 for frequency = 1 Hz and 0.5 Amp input current. The experimental results show cut-outs that cannot be captured by the developed model, this cut-out region is due to the entrapped air in the MR fluid [Cesmeci, 2017].

The experimental and modified model results for the applied EQ1 with different intensities and current conditions are shown in Figure A-4. Thus, the modified model predicts the device response when subjected to the same displacement history as applied to the device; however, it does not represent the whole system response under the applied earthquake. The cut-out region in the experimental response becomes more apparent as the applied current increases, whereas the modified model exhibits the desired full hysteresis loops. Otherwise, the experimental and modified model results are well-correlated in the compression zone. The flat slope region in the center of the experimental force vs. displacement curve, most visible in Figure A-4 (c) and (d) is due to the air in both the
bottom and top chambers that results during any load reversal [Cesmeci, 2017]. The flat slope region, in particular, delays engagement of the tension stiffness compared to its response in the modified model. In summary, the experimental results validate that the device can theoretically provide the stiffness and damping characteristics that were assumed in the simulation studies to evaluate effectiveness of 3D isolation. Potential design improvements have been identified that would align the physical device response more closely with the ideal model response [Cesmeci, 2017].

![Comparison between the experimental and modified model results for the applied ground motions with different intensities and current conditions.](image)

**Figure A-4**: Comparison between the experimental and modified model results for the applied ground motions with different intensities and current conditions.
A.1 References

Appendix (B) - Building Design

Hypothetical steel special concentric braced-frame buildings were designed for office occupancy. Three buildings with different structure height/width (h/b), or aspect ratios, were studied. All buildings are 6-story tall, multi-bay plane frame buildings, with story height = 5.0 m and bay width = 7.5 m. Different aspect ratios were achieved by varying the number of bays in the three buildings. Bld. I – 6x6 with 6 bays by 6 bays is classified as wide, with low aspect ratio h/b = 0.67. Bld. III – 2x2, representing a slender building, with h/b = 2.0, was achieved by decreasing to 2 bays by 2 bays, while Bld. II – 3x3 had intermediate slenderness with 3 bays by 3 bays and h/b = 1.34.

Design loads were calculated according to ASCE 7-16 [ASCE, 2016]. Superimposed dead loads were 1.35 and 1.05 kN/m² while assigned live loads were 3.8 and 1.0 kN/m² for building floors and roof, respectively. An additional uniform line load = 3.5 kN/m was applied to edge girders to represent exterior wall self-weight. A hypothetical site located in Los Angeles area on soil class (D) with Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R), 5% damped spectral acceleration of S_S = 2.11g at short periods and S_I = 0.74g at 1.0 sec period. Seismic loads were computed using the equivalent lateral force procedure [ASCE, 2016]. The weight of composite floor section including corrugated sheets and concrete topping used in lateral force calculations were 2.15 kN/m².

B.1 Fundamental Period

- Bld. I – 6x6: Fundamental period calculated based on section 12.8.2 [ASCE, 2016].

\[ T_a = C_t \times (h_a)^x = 0.0488 \times (30)^{0.75} = 0.625 \text{ sec} \]
T = C_u * T_a = 1.4 * 0.625 = 0.875 sec

where T_a is approximate fundamental period (sec), C_t and x are coefficients determined from Table 12.8-2 [ASCE, 2016] according to structure type. For special concentric braced-frame, C_t = 0.0488 and x = 0.75. Also, T is the fundamental period (sec) and C_u is coefficient for upper limit determined from Table 12.8-1 [ASCE, 2016] according to design spectral response acceleration at 1.0 sec. for studied site parameters, C_u = 1.4.

- Bld. II – 3x3 and III – 2x2: Fundamental period determined from modal analysis of developed models using same sections of Bld. I – 6x6 with modifying plan dimensions. T = 0.71 and 0.52 sec for Bld. II – 3x3 and III – 2x2, respectively as presented in Chapter (4)

### B.2 Equivalent Static Force Calculation for Fixed Base Buildings

Seismic base shear is calculated based on section 12.8.1 and horizontal distribution of forces are calculated based on section 12.8.4 [ASCE, 2016].

#### B.2.1 Seismic Response Coefficient Cs

\[
C_s = \frac{S_{DS}}{R/I} \leq \frac{S_{D1}}{T(R/I)}, \quad C_s \geq \frac{0.5 S_1}{R/I} \quad (S_1 \geq 0.6g)
\]

where \( S_{DS}, S_{D1} \) is design spectral response acceleration parameter in the short period and 1.0 sec, R = the response modification factor in Table 12.2-1 according to structure type and I = the importance factor determined in accordance with section 11.5.1. For special concentric braced-frame, R = 6 and I assumed = 1.0.

- Bld. I – 6x6
\[
\frac{S_{DS}}{R/I} = \frac{1.40667}{6/1} = 0.234 \quad \frac{S_{D1}}{T(R/I)} = \frac{0.74}{0.875\left(\frac{6}{1}\right)} = 0.141
\]

\[
0.5 \frac{S_1}{R/I} = \frac{0.5 \times 0.74}{6/1} = 0.062 \quad \Rightarrow \quad C_S = 0.141
\]

- Bld. II – 3x3

\[
\frac{S_{DS}}{R/I} = \frac{1.40667}{6/1} = 0.234 \quad \frac{S_{D1}}{0.7(R/I)} = \frac{0.74}{0.71\left(\frac{6}{1}\right)} = 0.175
\]

\[
0.5 \frac{S_1}{R/I} = \frac{0.5 \times 0.74}{6/1} = 0.062 \quad \Rightarrow \quad C_S = 0.175
\]

- Bld. III – 2x2

\[
\frac{S_{DS}}{R/I} = \frac{1.40667}{6/1} = 0.234 \quad \frac{S_{D1}}{0.52(R/I)} = \frac{0.74}{0.52\left(\frac{6}{1}\right)} = 0.247
\]

\[
0.5 \frac{S_1}{R/I} = \frac{0.5 \times 0.74}{6/1} = 0.062 \quad \Rightarrow \quad C_S = 0.234
\]

**B.2.2 Seismic Base Shear**

- Bld. I – 6x6

\[
W_s = (6\times3.5 + 3.2)\times45\times45 + 6\times3.5\times45\times4 = 52785 \text{ kN}
\]

\[
V = C_S \times W_s = 0.141 \times 52785 = 7442.7 \text{ kN}
\]

- Bld. II – 3x3

\[
W_s = (6\times3.5 + 3.2)\times22.5\times22.5 + 6\times3.5\times22.5\times4 = 14141.25 \text{ kN}
\]

\[
V = C_S \times W_s = 0.175 \times 14141.25 = 2474.7 \text{ kN}
\]
- Bld. III – 2x2

\[ W_s = (6 \times 3.5 + 3.2) \times 15 \times 15 + 6 \times 3.5 \times 15 \times 4 = 6705 \text{ kN} \]

\[ V = C_s \times W_s = 0.234 \times 6705 = 1569.0 \text{ kN} \]

**B.2.3 Horizontal Distribution of Seismic Forces**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Lateral Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bld. I</td>
</tr>
<tr>
<td>6th Floor</td>
<td>1765.7</td>
</tr>
<tr>
<td>5th Floor</td>
<td>2031.3</td>
</tr>
<tr>
<td>4th Floor</td>
<td>1557.6</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>1106.1</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>682.7</td>
</tr>
<tr>
<td>1st Floor</td>
<td>299.2</td>
</tr>
<tr>
<td>Total</td>
<td>7442.7</td>
</tr>
</tbody>
</table>

**B.3 Equivalent Static Force Calculation for Isolated Base Bld. I**

In the building with Hz isolation, isolator response is represented by bilinear force-deformation in the horizontal direction, such as provided by elastomeric or rubber bearings. The horizontal force-deformation parameters are selected to achieve effective horizontal period, \( T_{\text{eff,H,R}} = 4 \) sec and effective damping ratio, \( \xi_{\text{eff,H,R}} = 25\% \) at the maximum displacement, \( D_{\text{Max,H,R}} = 0.7 \) m. The yield displacement is also selected as \( D_{Y,R} = 0.01 \) m.

Force calculations and distribution are based on section 17.5 [ASCE, 2016].

\( W_{\text{Floor}} = 9261 \text{ kN} \). \( W_{\text{roof}} = 6480 \text{ kN} \) and \( W_{\text{base}} = 4630.5 \text{ kN} \)

Total weight, \( W = 9261 \times 5 + 6480 + 4630.5 = 57415.5 \text{ kN} \)

Total weight without base weight, \( W_s = 52875 \text{ kN} \)
\[
D_M = \frac{g_{SM1} T_M}{4 \pi^2 B_M} = \frac{9.81 \times 1.11 \times 4}{4 \pi^2 \times 1.6} = 0.69 \text{ m} \approx 0.7 \text{ m}
\]

\[
T_M = 2 \pi \sqrt{\frac{W}{k_M g}} \rightarrow 4 = 2 \pi \sqrt{\frac{57415.5}{k_M \times 9.81}} \rightarrow k_M = 14441.1 \text{ kN/m}
\]

To use code equations, \( T_M = T_{\text{eff,H.R}} \), \( \beta_M = \xi_{\text{eff,H.R}} \), \( D_M = D_{\text{Max,H.R}} = \) and \( K_M \) is effective stiffness.

\[
\beta_M = \frac{2 Q_d (D_M - D_Y)}{\pi F_{\text{max}} \times D_M} \rightarrow 0.25 = \frac{2 Q_d (D_M - D_Y)}{\pi D_M^2 \times k_M} \rightarrow 0.25 = \frac{2 Q_d (0.7 - 0.01)}{\pi 0.7^2 \times 14441.1}
\]

\[
\rightarrow Q_d = 4027.2 \text{ kN}
\]

\[
F_{\text{max}} = Q_d + D_M \times k_d = D_M \times k_M \rightarrow 0.7 \times 14441.1 = 4027.2 + 0.7 \times k_d \rightarrow k_d = 8688 \text{ kN/m}
\]

\[
Q_{d,\text{max}} = 1.5 Q_d = 1.5 \times 4027.2 = 6040.8 \text{ kN}
\]

\[
k_{d,\text{max}} = 1.3 k_d = 1.3 \times 8688 = 11294.4 \text{ kN/m}
\]

where \( \beta_M \) is equivalent viscous damping assuming a bilinear force-deformation loop. \( Q_d \) is Characteristic Strength and \( k_d \) is Post-Yield Stiffness. Isolator properties, \( T_M, k_M, \beta_M, D \) and \( F \) are calculated iteratively.
Total unreduced seismic base shear, $V_{st} = V_b \left( \frac{W_s}{W} \right)^{1-2.5\beta_M}$

$$= 11607 \left( \frac{52875}{57415.5} \right)^{0.175} = 11440.9 \text{ kN}$$

$$R_I = \frac{3}{8} \times R \leq 2$$

Total seismic design base shear, $V_s = \frac{V_{st}}{R_I} = \frac{11440.9}{2} = 5720.45 \text{ kN}$

where $V_b$ is $F_{max}$, $R_I$ is factor based on the type of seismic force resisting system used for the structure above the base level, and shall be three-eighths of the value of $R$ given in Table 12.2-1, with a maximum value not greater than 2.0 and a minimum value not less than 1.0.
\[ F_1 = \frac{V_b - V_{st}}{R_1} = \frac{11607 - 11440.9}{2} = 83.05 \text{ kN} \]

\[ F_X = C_{VX} V_S \text{ where } C_{VX} \text{ is distribution factor} \]

<table>
<thead>
<tr>
<th>Floor</th>
<th>Lateral Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6\text{th} Floor</td>
<td>2777.67</td>
</tr>
<tr>
<td>5\text{th} Floor</td>
<td>1893.60</td>
</tr>
<tr>
<td>4\text{th} Floor</td>
<td>765.30</td>
</tr>
<tr>
<td>3\text{rd} Floor</td>
<td>238.00</td>
</tr>
<tr>
<td>2\text{nd} Floor</td>
<td>45.88</td>
</tr>
</tbody>
</table>

**B.4 Isolation Modeling and Design for Isolated Base Buildings**

**B.4.1 Horizontal Isolation**

Horizontal and vertical isolators were simulated as stacked link supports.

The horizontal bearing was modeled in SAP 2000 as nonlinear rubber isolator.

The calculated parameters are for one link support and equation used to calculate link support properties are as follows:

\[ D_M = \frac{g S_{M1} T_M}{4 \pi^2 b_M}, T_M = 2 \pi \frac{W}{\sqrt{k_M g}} \rightarrow \text{get } k_M \]

\[ \beta_M = \frac{2 Q_d (D_M - D_y)}{\pi F_{\text{max}} \times D_M} = \frac{2 Q_d (D_M - D_y)}{\pi D_M^2 \times k_M} \rightarrow \text{get } Q_d \]

\[ F_{\text{max}} = Q_d + D_y \times k_d + (D_M - D_y) \times k_d = D_M \times k_M \rightarrow \text{get } k_d \]

\[ F_y = Q_d + k_d \times D_y, k_i = F_y / D_y \text{ and } \alpha = k_d / k_i \]
Vertical isolation was modeled in SAP 2000 as two parallel link supports (Multi-linear Elastic for spring force and linear link for viscous force). The calculated parameters are for one link support

Effective Period ($T_V$) = 0.5 sec

Maximum displacement = 0.1 m

B.5 References

American Society of Civil Engineers (ASCE) [2016] "ASCE Standard – ASCE/SEI 7-16: Minimum design loads for buildings and other structures," American Society of Civil Engineers (ASCE), Reston, VA.