SEISMIC PERFORMANCE AND SENSITIVITY OF FLOOR ISOLATION SYSTEMS IN STEEL PLATE SHEAR WALL STRUCTURES

A Thesis in
Civil Engineering
by
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In the US, there is growing interest in utilizing the floor isolation systems to protect targeted valuable content or equipment, e.g., computer servers during earthquake. The concept of paring the steel plate shear wall frame (SPSW) with floor isolation system (FIS) is that SPSW might effectively limit drift, protect drift sensitive components, whereas the floor isolation system reduces acceleration demands on targeted acceleration sensitive components. This study investigates the performance and sensitivity of floor isolation systems in steel plate shear wall frames under design basis (DBE) and maximum considered earthquake (MCE). The floor isolation systems were first designed and modeled for the level 1, 2 and level 3, 5 and 8 for 3-story and 9-story steel plate shear walls in OpenSees, respectively; followed with a performance study using nonlinear response history analysis to investigate the performance of floor isolation systems in steel plate shear walls. Finally a sensitivity study was performed to investigate the effect that variations in the web plate strength and stiffness have on the performance of the FISs in SPSWs.

The results show that: (i) the floor isolation system is effective in limiting acceleration demand on equipment in SPSW frame under DBE; and (ii) isolator displacement demands are large and need to be reduced; (iii) the floor isolation system performs better on the lower level in the steel plate shear wall frames. Moreover, from the sensitivity study, the results show that: (iv) the equipment absolute acceleration ratio is insensitive to the variations in the structure and do not need to be considered; (v) the maximum displacement response of the isolators is sensitive to the variations in strength and stiffness and need to be considered when designing the capacity of isolators; (vi) the floor absolute acceleration is sensitive to variations in web plate strength and stiffness which suggests that significant damage to rigidly attached equipment in SPSWs might be expected under DBE and MCE events.
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CHAPTER 1

Introduction

1.1 General

Base isolation is a method that has been used to protect structures from the damaging effects of earthquake ground shaking in the U.S. for nearly three decades (Naeim and Kelly 1999). Base isolation is achieved by introducing horizontally flexible elements called “isolators” typically located at the interface of the superstructure and substructure as illustrated in Fig. 1a. The horizontally flexible isolators increase the period of vibration of the structure thereby reducing the absolute acceleration demands imposed on the structure and contents. The high cost and complexity associated with isolating an entire structure coupled with the knowledge that the majority of the value of the building is in the contents and nonstructural components (Taghavi and Miranda 2003), from a cost breakdown of the value of building components shown in Fig. 2, has raised interest in isolating targeted pieces of equipment (e.g. computer servers, medical equipment) or valuable content (e.g. precious artifacts).

Equipment isolation involves seismically protecting individual pieces of acceleration sensitive equipment (or content) located at a particular level within a traditional fixed-base multistory structure from an earthquake hazard as illustrated in Fig 1b. Isolating groups of acceleration sensitive components or contents, such as computer servers, is referred to as floor isolation. The floor isolation system consists of a raised floor system supported by the isolation system on top of the primary structure's floor system.

![Image of base and floor isolation](image)

a. Base isolation  

b. Floor isolation

Fig. 1.1 Illustration of base and floor isolation
Both Equipment isolation systems (EIS) and floor isolation systems (FIS) involve decoupling acceleration sensitive component located at a particular level within the multi-story structure from the supporting primary structural system using horizontally flexible elements that can be similar to those used for base isolation.

Fig. 1.2 Cost breakdown of office buildings, hotels and hospitals
(After Taghavi and Miranda 2003)

However, the one key difference between traditional base isolation (Fig. 1a) and floor isolation (Fig. 1b) is the seismic input to the system. For a base isolated structure, the seismic input to the system is the ground motion (ignoring soil-structure interaction) that arrives at the base of the structure, whereas the seismic input to the floor isolation system is the response of the primary structural system at the location of the FIS system due to the ground level motion where both the ground motion and the response of the structure to a particular ground motion are uncertain.

For design of the primary structural system and the FIS, uncertainty associated with the ground motion is implicitly considered through the use of design spectrum or explicitly considered by performing a number of response history analyses using ensembles of recorded earthquake ground motions. Currently, uncertainties associated with the response of the primary structural system due to variability in material
and geometric properties are not considered for design of floor isolation systems and it is not clear whether these uncertainties should be considered.

1.2 Objectives

The objectives of this study are to: (i) investigate the seismic performance of the floor isolation systems in steel plate shear walls; and (ii) determine the sensitivity of the equipment and isolation system response to variations in the material and geometric parameters of the primary structural system. For the purpose of this study, floor isolation systems will be designed to protect computer servers located at various levels in three (SPSW3M) and nine (SPSW9M) story steel plate shear wall (SPSW) frames (Berman 2011). A steel plate shear wall is chosen for the lateral force resisting system among all the commonly used systems (e.g., moment frames, braced frames, among others) because the combination of SPSW for limiting drift and floor isolation systems for limiting acceleration could simultaneously protect drift and targeted acceleration sensitive components.

1.3 Scope

The thesis will first design and investigate the performance of floor isolation systems in steel plate shear walls under design and maximum considered earthquakes; then quantify the sensitivity of the response of the isolated equipment to variations including SPSW web plate thickness and plate yield strength.

The floor isolation systems (FISs) were first designed for level 1, 2 and level 3, 5 and 8 for 3-story and 9-story steel plate shear wall frames (SPSWs), respectively. SPSWs and FISs were then modeled in OpenSees (McKenna et al. 2006) to assess the performance of the FIS system in steel plate shear walls under design and maximum considered earthquake ground shaking using earthquake ground motion records obtained from the SAC (SEAOC, ATC and CUREE) ground motion database (SAC 1997). Following the performance assessment, a sensitivity study will be performed to investigate the sensitivity of the response of the acceleration sensitive equipment and floor isolation system to variation in the material and geometric properties of the primary structural system, specifically, the SPSW web plate yield strength and web plate thickness.

1.4 Thesis Organization

The remainder of the thesis is organized as follows. Chapter 2 presents background information and relevant literature on steel plate shear wall and isolated equipments; Chapter 3 presents the prototype models of SPSW and the design of the floor isolation systems. In addition, Chapter 3 presents the numerical model in Opensees (McKenna et al. 2006) used for the study. Chapter 4 presents the performance and sensitivity of FISs at different levels within the 3-story and 9-story steel plate shear wall
frames under design and maximum considered earthquakes. Chapter 5 presents a short summary and the conclusions. A list of references is provided after Chapter 5. Appendix A presents the period calculation for the 3- and 9-story steel plate shear wall frames. Appendix B presents the design of floor isolation systems. Appendix C presents the results from sensitivity study.
CHAPTER 2

Background and Literature Review

2.1 General

For several decades engineers have recognized that certain structures, e.g., hospitals, data centers among others, house equipment that is vital to the operation of these facilities. Damage to such equipment during an earthquake is likely to lead to disruption of service or business and potentially substantial social and/or economic losses. In the US, there is growing interest in floor isolation systems as a cost effective means of protecting targeted equipment. It can be easily moved and installed on different level in the structures.

The concept of pairing the steel plate shear wall (SPSW) with Floor Isolation System (FIS) is that the SPSW can effectively limit drift, protect drift sensitive components, while the floor isolation system reduces acceleration demands on acceleration sensitive components. However, there is still a lack of overall understanding of the seismic performance of floor isolation system when implemented in the upper levels of multi-story structures.

This chapter is organized into sections that provide background information and literature on: (i) equipment and floor isolation; (ii) steel plate shear walls.

2.2 Equipment and Floor Isolation

Systems to seismically protect individual pieces of equipment composed of a complex arrangement of springs and dampers (Demetriades et al. 1992, Kosar et al. 1993, Makris and Constantinou 1992) have been shown to improve the seismic response of equipment through experimental testing. However these systems have not been widely adopted likely due to the complexity of the system itself and implementation. Seismically isolating individual pieces of equipment have been considered (Lambrou and Constantinou 1994) however the typical low weight of the equipment makes it difficult to achieve the desired period using traditional hardware for the base isolation of buildings. Ball-in-cone systems consisting of a spherical stainless steel ball that sits in a spherical concave dish where the weight of the equipment is carried by bearing between the ball and concave dish have also been investigated (Kasalanati et al. 1997) for the seismic protection of individual pieces of equipment and are commercially available (Kemeny 1997). However the large stress concentration at the contact point between the spherical ball and concave dish that can create a groove of the displacement history in the dish following earthquake excitation that will impede free motion under subsequent earthquake events.
Isolating entire floor systems supporting groups of equipment, i.e., floor isolation, increases the weight so that a period long enough to effectively protect the equipment can be achieved. Lambrou and Constantinou (1994) demonstrated by experimental and analytical simulations that substantial reductions in the response of a computer cabinet could be achieved by isolating a computer floor with friction pendulum (Zayas et al. 1987) bearings with and without adding viscous fluid dampers. For the earthquake simulation testing the isolated floor system was mounted directly on the earthquake simulator platform and simulated floor motions used as the seismic excitation. A bi-directional spring unit has been investigated (Cui et al. 2010) for isolated floor systems. An advantage of the bi-directional spring unit over traditional friction pendulum bearings is that the vertical and horizontal force transfer mechanisms are uncoupled so a wide range of isolated periods can be achieved by tuning the horizontal springs. The studies summarized above have focused primarily on the performance of the floor or equipment isolation systems alone. There have been studies investigating the performance and reliability of passive and semi-active equipment isolation systems located on the upper levels of multi-story structures (Alhan and Gavin 2005; Gavin and Zaicenco 2007). However, these studies assumed the primary structural system response to be linear, which is unlikely under moderate to severe earthquake ground shaking.

2.3 Steel Plate Shear Wall Frame

This study investigates the performance and sensitivity of floor isolation systems in multistory structures with steel plate shear walls (SPSWs) as the lateral force resisting system through numerical simulations. Steel plate shear walls have been used as a lateral force resisting system in buildings to provide wind or seismic resistance (Sabelli and Bruneau 2007). Typical SPSWs consist of a thin, unstiffened, web plate connected to a boundary frame composed of horizontal boundary frame elements (HBEs or beams) and vertical boundary frame elements (VBEs or columns) through fishplates (Driver et al. 1997; Sabelli and Bruneau 2007). Under in-plane lateral loading, the infill plate buckles in shear, developing diagonal tension field action that resists lateral forces (Timler and Kulak 1983; Driver et al. 1997; Berman and Bruneau 2005). A photograph of the steel plate shear wall (SPSW) and its hysteresis response are presented in Fig. 2.1.
(a) Photograph of the SPSW tested by Vian and Bruneau (2005)

(b) Hysteretic response of SPSW (Vian and Bruneau 2005)
Fig. 2.1 Experimental test of SPSW under cyclic load and hysteretic characterization
CHAPTER 3

Modeling of Floor Isolation Systems

3.1 General

This chapter provides details of the prototype acceleration sensitive equipment, the numerical models used to analyze the isolated equipment in steel plate shear wall frames (SPSW), and the design of the floor isolation systems. Additional details on the design of floor isolation systems are provided in Appendix B.

3.2 Prototype Systems

To facilitate the design of the floor isolation system the acceleration sensitive equipment is assumed to be computer servers that for example, could be used for telecommunications or data storage. In lieu of fragility data a manufacturer’s (IBM 2010) recommended maximum real-time instantaneous acceleration of 0.3 g was used as the limiting value of absolute acceleration applied to the equipment for the design of the floor isolation system. A standardized computer floor layout of 50 ft (15 m) by 30 ft (9 m) that gives maximum efficiency of electrical power (Rasmussen and Torell 2007) was assumed for the prototype computer floor. It was assumed the floor is isolated on twelve friction pendulum (Zayas et al. 1987) type bearings. The 50 ft by 30 ft layout accommodates 60 cabinets with dimensions of 38" wide, 24" deep and 81" height that can hold 120 server blades resulting in a weight of 1300 lbs per cabinet. The total weight of the isolated computer floor was taken as the weight of the 60 cabinets computer equal to 78 kips.

Two steel plate shear wall frames designed by Berman (2011) for modified version of the SAC buildings were adopted for this study. One frame is three stories in height (SPSW 3M) and the other nine stories (SPSW 9M). Details of the design can be found in Berman (2011). Steel plate shear walls consist of a thin, unstiffened, web plate connected to a boundary frame composed of horizontal boundary frame elements (HBEs or beams) and vertical boundary frame elements (VBEs or columns) through fishplates (Driver et al.1997; Sabelli and Bruneau 2007). Fig. 3.1 shows a typical web plate to boundary frame connection detail adopted from Sabelli and Bruneau (2007). The web plate is welded to a thicker "fish plate" that is welded to the flange of the HBE or VBE. Under in-plane lateral loading, the web plate buckles in shear, developing diagonal tension field action that resists lateral forces (Timler and Kulak 1983; Driver et al. 1997; Berman and Bruneau 2005). Fig. 3.2a presents a schematic of the SPSW3M with typical nomenclature.
The geometric dimensions and the first mode period of the SPSW3M and SPSW9M frames are presented in Table 3.1. The first mode periods reported in Table 3.1 are obtained from modal analysis of the frames performed in OpenSees (McKenna et al. 2006) as $T_{\text{analysis}}$, and by an approximate method $T_{\text{approx}}$, considering shear-flexural interaction (details provided in Appendix A). These buildings are assumed to be located on site class D soil with mapped maximum considered earthquake (MCE) spectral response parameters of $S_{MS}$ 1.61g and $S_{M1}$ 1.19g, for 0.2s and 1.0s periods, respectively (Berman 2011). Resulting
plate thicknesses, member sizes and story floor masses for the 3M and 9M frames are presented in Tables 3.2 and 3.3.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Number of Stories</th>
<th>Number of SPSW frames</th>
<th>Bay width (ft)</th>
<th>Story Height (ft)</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$T_{\text{analysis}}^a$</td>
</tr>
<tr>
<td>3M</td>
<td>3</td>
<td>4</td>
<td>15</td>
<td>13</td>
<td>0.39</td>
</tr>
<tr>
<td>9M</td>
<td>9</td>
<td>6</td>
<td>15</td>
<td>13</td>
<td>1.14</td>
</tr>
</tbody>
</table>

$a$ first mode period from modal analysis of strip models in Opensees.

$b$ first mode period calculated using approximation method (see Appendix A).

<table>
<thead>
<tr>
<th>Story</th>
<th>Story Mass (kips·sec²/in)</th>
<th>Infill Plate Thickness (in)</th>
<th>HBE Section$^a$</th>
<th>VBE Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.38</td>
<td>0.125</td>
<td>W24x131</td>
<td>W14x398</td>
</tr>
<tr>
<td>2</td>
<td>1.38</td>
<td>0.125</td>
<td>W21x131</td>
<td>W14x257</td>
</tr>
<tr>
<td>3</td>
<td>1.38</td>
<td>0.063</td>
<td>W21x131</td>
<td>W14x257</td>
</tr>
</tbody>
</table>

$^a$ HBE sizes are beams above infill plate. HBE 0 is a W24x178

<table>
<thead>
<tr>
<th>Story</th>
<th>Story Mass (kips·sec²/in)</th>
<th>Infill Plate Thickness (in)</th>
<th>HBE Section$^a$</th>
<th>VBE Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.96</td>
<td>0.25</td>
<td>W18x86</td>
<td>W14x730</td>
</tr>
<tr>
<td>2</td>
<td>0.94</td>
<td>0.25</td>
<td>W18x86</td>
<td>W14X730</td>
</tr>
<tr>
<td>3</td>
<td>0.94</td>
<td>0.25</td>
<td>W24x146</td>
<td>W14x500</td>
</tr>
<tr>
<td>4</td>
<td>0.94</td>
<td>0.187</td>
<td>W18x86</td>
<td>W14x500</td>
</tr>
<tr>
<td>5</td>
<td>0.94</td>
<td>0.187</td>
<td>W18x86</td>
<td>W14x500</td>
</tr>
<tr>
<td>6</td>
<td>0.94</td>
<td>0.187</td>
<td>W24x146</td>
<td>W14x500</td>
</tr>
<tr>
<td>7</td>
<td>0.94</td>
<td>0.125</td>
<td>W16x57</td>
<td>W14x370</td>
</tr>
<tr>
<td>8</td>
<td>0.94</td>
<td>0.125</td>
<td>W24x146</td>
<td>W14x370</td>
</tr>
<tr>
<td>9</td>
<td>1.01</td>
<td>0.063</td>
<td>W18x106</td>
<td>W14x370</td>
</tr>
</tbody>
</table>

$^a$ HBE sizes are beams above infill plate. HBE 0 is a W24x178
3.3 Numerical Models

Models of SPSW and floor isolation systems were developed and analyzed in OpenSees (McKenna et al. 2006). The SPSW is modeled using the strip model approach that has been shown to provide reasonable predictions of the ultimate capacity (Driver et al. 1997) and cyclic response (Berman 2011) of steel plate shear walls. Fig. 3.2b presents a schematic of the strip model. The boundary frame (HBE and VBE) elements are modeled as nonlinear beam-column elements with fiber sections. Each HBE and VBE element is represented using seven nonlinear beam-column elements connected to the nodes. Each element has five integration points along the element length to account for the possibility of yielding at locations other than HBEs and VBEs ends. Fig. 3.3 shows a fiber section (W flange) for the 1st story column of the 3-story steel plate shear wall frame. The flange is discretized into 2 fibers along the height and 17 fibers along the width. The web is discretized into 2 fibers along the width and 19 fibers along the height. Each fiber is assumed material with bilinear stress-strain relationship to model the steel material.

The bilinear stress-strain relationship is represented using Giuffre-Menegotto-Pinto model shown in Fig. 3.4. The rounded shape of the Giuffre-Menegotto-Pinto model is found to aid convergence of the numerical analyses (Berman 2011). The boundary frame members are assumed to be ASTM A572 Grade 50 steel with minimum nominal yield strength $f_y$ of 50 ksi. The elastic modulus ($E$) is specified to be 29,000 ksi with a strain-hardening ratio of 0.02 (ratio between post-yield tangent and initial elastic tangent: $E_p / E$).

Fig. 3.3 A typical fiber section for 1st story column in SPSW3M
Since the VBEs are designed to have thick flanges and webs, it is reasonable to assume the panel zone deformation is negligible. The panel zone is modeled using an elastic beam with rigid offsets from the intersection of column and beam centerlines to the outlines of the column and beam flanges with large elastic modulus ($E$) of 290,000 ksi, area of 1,100 in$^2$ and moment of inertia of 70,000 in$^4$. Each horizontal rigid offset is 8.8 in.. Each vertical rigid offset is 12.5 in..

The web plate is represented using two sets of parallel pin-ended, tension-only, elements (truss element with Hysteretic Material Model in OpenSees). Each tension strip is connected to nodes on the other HBE and VBE. One set of strips is oriented at an angle, $\alpha$ with respect to the vertical, which represents the angle of the tension field that develops after the web plate buckles in shear. The strip orientation angle, $\alpha$ is established (Timler and Kulak 1983) using:

$$\tan^4 \alpha = \frac{1 + \frac{tL}{2A_c}}{1 + t\left(\frac{1}{A_b} + \frac{h^3}{360I_cL}\right)}$$

(3.1)

where $t$ is the web plate thickness; $L$ is the web plate width (see Fig. 3.2b); $h$ is the web plate height; $A_b$ is the HBE cross-sectional area; $A_c$ is the VBE (column) cross-sectional area; and $I_c$ is the VBE moment of inertia. The second set of strip is oriented at $(2\pi - \alpha)$ with respect to the vertical to provide resistance under reversed loading. All HBE-to-VBE connections are moment resisting. One half of the total floor mass is lumped at each of the nodes connecting the HBE-to-VBE elements and the frames are pinned at the base (see Fig. 3.2b).
The truss elements with high tension strength and low compression strength can allow the tension field action developed at a small lateral deformation, which would capture the shear buckling behavior of the web plate. The force-displacement relationship for the truss elements is shown in Fig. 3.5. The web plate is assumed A36 plate material. Therefore, a trilinear curve is presented with the initial elastic modulus of 29,000 ksi, yield strength $F_y$ of 46.8 ksi and tensile strength $F_u$ of 64.6 ksi.

![Fig. 3.5 Strip element cyclic axial load behavior](image)

The use of strip model for response history analysis is based on the Driver et al. (1998) that evaluates the strip model's capacity to predict the SPSW specimen behavior under quasi-static cyclic loading. The result from Driver et al. (1998) show that the strip model can adequately represent the global force-displacement behavior of the SPSW specimen with a somewhat more pinched behavior concluding that the model could provide reasonable but conservative results when applied in nonlinear response history analysis. Furthermore, the representation of the web plate as strips is justified (Vian and Bruneau 2005) on single-story, single-bay SPSWs under quasi-static cyclic loading. From the results of that experimental tests, the initial buckling are observed at a displacement corresponding to 0.2% intersotry drift.

Fig. 3.6 presents a schematic illustrating the floor isolation system and acceleration sensitive equipment as modeled in the SPSW frame. The floor isolation system consists of a rigid beam (raised floor) connecting two zero-length elements representing the isolators that are connected to the HBE-to-VBE connection nodes. The isolators are shown in Fig. 3.6 with length for illustration. The raised floor is assumed to be rigid with large elastic modulus ($E$) of 290,000 ksi, area of 1,100 in$^2$ and moment of
inertia of 70,000 in\(^4\) in lieu of further information. Also in the prototype model the isolators are distributed underneath the raised floor.

![Numerical model of the floor isolation system within SPSW frame](image)

**Fig. 3.6** Numerical model of the floor isolation system within SPSW frame

The isolators for the FIS were assumed to be a friction pendulum type bearing. Fig. 3.7 presents a schematic of a friction pendulum bearing and its components, an articulated friction slider with bearing material typically polytetrafluoroethylene (PTFE) and a spherical concave dish with stainless steel overlayment.

![FPS isolator section](image)

**Fig. 3.7** FPS isolator section (taken from Zayas et al. 1987)

A rate-independent plasticity model with a bilinear force-displacement relationship as shown in Fig. 3.8 is used to model the transverse force-displacement behavior of the isolators that has been shown to be a reasonable model for friction pendulum bearings (Mosqueda et al. 2004).
The bilinear relationship is characterized by $Q_d$, the zero-displacement force intercept; $K_d$, the second-slope stiffness; $K_{\text{eff}}$, the effective stiffness or secant stiffness; $d_y$, the yield displacement; and $d_{\text{max}}$, the maximum isolator displacement. The vertical force-displacement behavior of the isolators is assumed to be linear with a stiffness of $1\times10^{10}$ kips/in that is a reasonable assumption for friction pendulum bearings. For friction pendulum bearings:

$$Q_d = \mu W$$  \hspace{1cm} (3.2)

where $\mu$ is the coefficient of friction and $W$ is the weight acting on the isolators. Additionally, the second-slope stiffness ($K_d$) is:

$$K_d = \frac{W}{R}$$  \hspace{1cm} (3.3)

where $R$ is the radius of the concave dish (see Fig. 3.7). The yield displacement, $d_y$, is assumed to be $0.02$ in that is a typical value and independent of the geometry of the slider (Sheller and Constantinou 1999). The parallel nature of isolators in an isolation system is utilized to simplify the model whereby the twelve isolators are modeled using 2 isolator elements each with properties equal to six times those of the individual isolators.

The 60 computer cabinets, i.e., acceleration sensitive equipment, are collectively modeled as a single lumped mass-stiffness, single-degree-of-freedom system (SDOF), atop the floor isolation system as illustrated in Fig. 3.6. The height of the SDOF system is specified to be 40 in approximately half the total height of the computer cabinet. The stiffness of the SDOF system ($k_c$) is calculated to be $65.38$ kips/in.
from the results of white noise testing performed on a computer cabinet (Lambrou and Constantinou 1994) that determined the first lateral mode frequency of 40 Hz. The mass of the SDOF system (m_r) was specified to be 0.2 k⋅s^2/in based on the total weight of 78 kips for the computer cabinets.

3.4 Floor Isolation System Design

3.4.1 General Procedure

There is currently no codified procedure for designing floor isolation system. However, the generally accepted procedure suggested by a bearing manufacturer (DIS 2010) is summarized here:

1. Selection of design basis earthquake ground motions (DBE) to perform unidirectional nonlinear response history analysis for buildings. Vertical excitation is not considered in this study.
2. Development of floor acceleration spectral demands at the location in the building at which the floor isolation system will be installed.
3. Design and detailing of the floor isolation system to meet the required performance objectives, i.e., acceleration limit.
4. Detailed design for manufacture and installation purpose.

3.4.2 Earthquake Ground Motions and Floor Spectra

This study utilized target spectral values and ground motions created for the SAC (SEAOC, ATC and CUREE) steel project (SAC 1997) for the Los Angeles area and an earthquake hazard corresponding to 2% in 50 years probability of exceedence with the target short (0.2 sec) and 1.0 second spectral values of 1.61g and 1.19g, respectively, for site D soil. Details of the ground motions are listed in Table 3.4.

The SPSW frames designed by Berman (2011) were assumed to be located in Los Angeles region and designed for S_DS and S_D1 value of 1.07g and 0.79g based on modified maximum spectral value of 1.61g and 1.19g for 0.2 sec and 1.0 sec, respectively.

For this study, the target spectral values of 1.61g and 1.19g and associated ground motions were assumed to represent the modified maximum (MCE) spectral values taken as S_MSS = 1.61g and S_M1 = 1.19g.

The design spectral acceleration parameters S_DS and S_D1 were determined as 1.07g and 0.79g per ASCE 7 (ASCE 2010):
\[ S_{DS} = \frac{2}{3} S_{MS} \]  

\[ S_{D1} = \frac{2}{3} S_{M1} \]  

Therefore, the design basis earthquake ground motions (DBE) were obtained by amplitude scaling the ground motions (Table 3.4) by a 2/3 factor. Fig. 3.9 presents the average of twenty DBE spectra with the design spectrum for SPSW frames. By comparing the average to the design spectrum, DBE ground motions satisfy ASCE 7 (ASCE 2010) requirement for nonlinear response history analysis that the average of ground motion spectra do not fall below the design spectrum for the period range of 0.2\(T\) to 1.5\(T\) where \(T\) is the 1st mode period of the structure.
Table 3.4 Details of Los Angeles ground motions having a probability of exceedence of 2% in 50 years

<table>
<thead>
<tr>
<th>SAC Name</th>
<th>Record</th>
<th>Duration (sec)</th>
<th>PGA (in/sec²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LA21</td>
<td>1995 Kobe</td>
<td>59.98</td>
<td>495.3</td>
</tr>
<tr>
<td>LA22</td>
<td>1995 Kobe</td>
<td>59.98</td>
<td>355.4</td>
</tr>
<tr>
<td>LA23</td>
<td>1989 Loma Prieta</td>
<td>24.99</td>
<td>161.4</td>
</tr>
<tr>
<td>LA24</td>
<td>1989 Loma Prieta</td>
<td>24.99</td>
<td>182.6</td>
</tr>
<tr>
<td>LA25</td>
<td>1994 Northridge</td>
<td>14.945</td>
<td>335.3</td>
</tr>
<tr>
<td>LA26</td>
<td>1994 Northridge</td>
<td>14.945</td>
<td>364.3</td>
</tr>
<tr>
<td>LA27</td>
<td>1994 Northridge</td>
<td>59.98</td>
<td>357.8</td>
</tr>
<tr>
<td>LA28</td>
<td>1994 Northridge</td>
<td>59.98</td>
<td>513.4</td>
</tr>
<tr>
<td>LA29</td>
<td>1974 Tabas</td>
<td>49.98</td>
<td>312.4</td>
</tr>
<tr>
<td>LA30</td>
<td>1974 Tabas</td>
<td>49.98</td>
<td>382.9</td>
</tr>
<tr>
<td>LA31</td>
<td>Elysian Park (simulated)</td>
<td>29.99</td>
<td>500.5</td>
</tr>
<tr>
<td>LA32</td>
<td>Elysian Park (simulated)</td>
<td>29.99</td>
<td>458.1</td>
</tr>
<tr>
<td>LA33</td>
<td>Elysian Park (simulated)</td>
<td>29.99</td>
<td>302.1</td>
</tr>
<tr>
<td>LA34</td>
<td>Elysian Park (simulated)</td>
<td>29.99</td>
<td>262.8</td>
</tr>
<tr>
<td>LA35</td>
<td>Elysian Park (simulated)</td>
<td>29.99</td>
<td>383.1</td>
</tr>
<tr>
<td>LA36</td>
<td>Elysian Park (simulated)</td>
<td>29.99</td>
<td>424.9</td>
</tr>
<tr>
<td>LA37</td>
<td>Palos Verdes (simulated)</td>
<td>59.98</td>
<td>274.7</td>
</tr>
<tr>
<td>LA38</td>
<td>Palos Verdes (simulated)</td>
<td>59.98</td>
<td>299.7</td>
</tr>
<tr>
<td>LA39</td>
<td>Palos Verdes (simulated)</td>
<td>59.98</td>
<td>193.1</td>
</tr>
<tr>
<td>LA40</td>
<td>Palos Verdes (simulated)</td>
<td>59.98</td>
<td>241.4</td>
</tr>
</tbody>
</table>
3.4.3 FEMA 450 Equation for FIS Design

FEMA 450 (2003) presents Eq. 3.6 for calculating the design displacement for isolation systems:

\[
D_D = \left( \frac{g}{4\pi^2} \right) \frac{S_{D1}T}{B_D}
\]  

(3.6)

where \( g \) is the acceleration due to gravity; \( S_{D1} \) is the 1-second design spectral acceleration parameter; \( T \) is the effective period of the isolation system at the design displacement, and \( B_D \) is the coefficient to account for damping other than 5% of the critical damping. Eq. 3.6 is derived based on the following relationship between spectral displacement, \( S_D(T) \), and the spectral acceleration \( S_A(T) \):

\[
S_D(T) = \frac{1}{\omega^2} S_A(T) \cdot g
\]

(3.7)

where \( \omega \) is the circular frequency equal to \( 2\pi / T \). Substituting \( 2\pi / T \) into Eq. 3.7 gives:

\[
S_D(T) = \frac{T^2}{4\pi^2} S_A(T) \cdot g
\]

(3.8)

For \( T \) greater than 1.0 sec, Eq. 3.6 assumes that the spectral acceleration \( S_A(T > 1.0) \) is:

\[
S_A(T) = \frac{S_{D1}}{T}
\]

(3.9)
where $S_{D1}$ is the 1-second spectral acceleration. Substitute Eq. 3.9 into Eq. 3.8 gives:

$$S_{D}(T) = \frac{T^2}{4\pi^2} \times \frac{S_{D1}}{T} = \left(\frac{g}{4\pi^2}\right)S_{D1}T = 10S_{D1}T$$  \hspace{1cm} (3.10)

However, an assumption of 5% of critical damping is included in the $S_{D1}$ value. For damping other than 5%, a damping coefficient ($B_D$) is introduced: that is a function of the effective damping as shown in Table 3.5.

$$d = S_{D1} = 10\frac{S_{D1}}{B_D}$$ \hspace{1cm} (3.11)

<table>
<thead>
<tr>
<th>Damping (Percentage of Critical)</th>
<th>\leq 2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_D$</td>
<td>0.8</td>
<td>1.0</td>
<td>1.2</td>
<td>1.5</td>
<td>1.7</td>
<td>1.9</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Fig. 3.10 plots the floor spectrum generated using floor acceleration history at level 1 obtained from the response history analysis of the SPSW3M frame. Also plotted is a $S_{D1}/T$ curve as assumed in the FEMA 450 equation for the period greater than 1 sec. However, as shown in Fig. 3.10, the $S_{D1}/T$ assumption overestimates the spectral demand by comparison to the floor response spectrum. Therefore, rather than using Eq. 3.8 that relies on the $S_{D1}/T$ assumption, a more general equation was used.

![Floor response spectrum for level 1 in SPSW3M](image)
3.4.4 General Equation for FIS Displacement Design

Since the FEMA 450 equation assumes $S_d(T > 1 \text{ sec}) = S_{01}/T$ that might not be a reasonable assumption for floor spectrum, a general equation using spectral acceleration ordinates from the floor spectrum was adopted for the design. The isolator displacement was calculated using Eq. 3.8 modified with the damping coefficient $B_D$ as:

$$
d = 10 \frac{S_d(T) \cdot T^2}{B_D}
$$

where $S_d(T)$ is determined directly from the 5% damped floor response spectra at each value of $T$. The design procedure is otherwise identical to the procedure put forth in FEMA 450 (FEMA 2003) and AASHTO Guide Specification (AASHTO 1999).

Section 3.4.5 presents an example of the general design procedure for a FIS on the level 1 of SPSW3M. Appendix B presents a summary of the design of the FIS located on other floors of the SPSW3M and SPSW9M frames.

3.4.5 Design Example of FIS for level 1 in SPSW3M using General Equation

This section illustrates the iterative procedure required to design isolation systems characterization by a bilinear relationship as shown in Fig. 3.8. The bilinear relationship (see Fig. 3.8) requires the effective properties ($K_{\text{eff}}$, $T_{\text{eff}}$, and $\beta_{\text{eff}}$) of the isolation system to be used in an iterative process to determine the design displacement and effective period. From the floor spectrum (Fig. 3.10) and the 0.3g acceleration limit, the design target effective period is 3.18 sec. A second-slope period $T_d = 4.0 \text{ sec}$ was assumed for a friction pendulum bearing that gives $K_d = 0.042 \text{ kips/in}$ for the total weight of 39 kips. Additionally, a coefficient of friction of 0.07 ($\mu$) was assumed (Constantinou et al. 1999). $\mu = 0.07$ is a reasonable coefficient of friction for friction pendulum bearings.

To initiate the process, the characteristic strength is calculated as:

$$
Q_d = \mu W = 0.07 \times 6.5 \text{ kips} = 0.455 \text{ kips}
$$

A FIS displacement of 15 in is assumed and the effective stiffness of a FIS isolator is calculated as:
\[
K_{\text{eff}} = \frac{Q_d}{d_0} + K_d = \frac{0.455 \text{ kips}}{15 \text{ in}} + 0.042 \text{ kips/in} = 0.072 \text{ kips/in}
\] (3.14)

From which \(T_{\text{eff}}\) is calculated as:

\[
T_{\text{eff}} = 2\pi \sqrt{\frac{W}{K_{\text{eff}} \cdot g}} = 2\pi \sqrt{\frac{6.5 \text{ kips}}{0.072 \text{ kips/in} \times 386.4 \text{ in/sec}^2}} = 3.04 \text{ sec}
\] (3.15)

The effective damping ratio is calculated as:

\[
\beta_{\text{eff}} = \frac{2Q_d}{\pi K_{\text{eff}} \cdot d_0} = \frac{2 \times 0.455 \text{ kips}}{\pi \times 0.072 \text{ kips/in} \times 15 \text{ in}} = 0.268
\] (3.16)

By interpolation from Table 3.5, the damping coefficient \(B_D\) is determined to be 1.64. The updated displacement is calculated using Eq. 3.12 as:

\[
d_i = \frac{10S_d (T_{\text{eff}}^2 \cdot T_{\text{eff}}^2)}{B_D} = \frac{10 \times 0.325 \times (3.04)^2}{1.64} = 18.3 \text{ in}
\] (3.17)

where \(S_d(3.04) = 0.325 \text{ g}\) is the spectrum ordinate at the period of 3.04 sec obtained from Fig. 3.10. The relative error in the estimated displacement is:

\[
\text{Relative Error} = \frac{|d_i - d|}{d_i} \times 100\% = \frac{|15 \text{ in} - 18.3 \text{ in}|}{18.3 \text{ in}} \times 100\% = 18\%
\] (3.18)

The error is unacceptably large so the updated displacement of \(d_i = 18.3 \text{ in}\) is used for the second iteration. Again the effective stiffness is calculated:

\[
K_{\text{eff}} = \frac{Q_d}{d_0} + K_d = \frac{0.455 \text{ kips}}{18.3 \text{ in}} + 0.042 \text{ kips/in} = 0.067 \text{ kips/in}
\] (3.19)

The effective period \((T_{\text{eff}})\) of an isolator is:

\[
T_{\text{eff}} = 2\pi \sqrt{\frac{W}{K_{\text{eff}} \cdot g}} = 2\pi \sqrt{\frac{6.5 \text{ kips}}{0.067 \text{ kips/in} \times 386.4 \text{ in/sec}^2}} = 3.15 \text{ sec}
\] (3.20)

The effective damping ratio \(\beta_{\text{eff}}\) is calculated as:
The damping coefficient \( B_d \) is determined by interpolation in Table 3.5 as:

\[
B_d = 1.57
\]  

(3.22)

The updated displacement is:

\[
d_2 = \frac{10S_A(T_{eff})^2}{B_d} = \frac{10 \times 0.304 \times (3.15)^2}{1.57} = 19.18 \text{ in}
\]

(3.23)

The relative error is calculated as:

\[
\text{Relative Error} = \frac{|d_2 - d_1|}{d_2} \times 100\% = \frac{|19.18 \text{ in} - 18.3 \text{ in}|}{19.18 \text{ in}} \times 100\% = 4.59\%
\]

(3.24)

One more iteration was performed so that the relative error for the displacement estimate is 0.6% resulting in a design displacement of 19.3 in. After three iterations the design displacement of \( d = 19.3 \text{ in} \) results in an effective period of \( T_{eff} = 3.18 \text{ sec} \) for an isolation system with \( \mu = 0.07 \) and \( T_d = 4.0 \text{ sec} \).

A Summary of FIS design parameters for the isolation systems at the level 1 and 2 of 3 story SPSW frame (SPSW3M) is presented in Table 3.6. Table 3.7 presents a summary of the design of FIS at level 3, 5 and 8 of the 9 story SPSW frame (SPSW9M). Details pertaining to the other FIS designs are provided in Appendix B.

Table 3.6 Summary of FIS design for SPSW3M

<table>
<thead>
<tr>
<th>Level</th>
<th>Friction Coefficient ( \mu )</th>
<th>Second-Slope Period ( T_d ) (sec)</th>
<th>Effective Period ( T_{eff} ) (sec)</th>
<th>Design Displacement ( d ) (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.07</td>
<td>4.00</td>
<td>3.2</td>
<td>19</td>
</tr>
<tr>
<td>2</td>
<td>0.07</td>
<td>4.00</td>
<td>3.2</td>
<td>21</td>
</tr>
</tbody>
</table>
Table 3.7 Summary of FIS design for SPSW9M

<table>
<thead>
<tr>
<th>Level</th>
<th>Friction Coefficient $\mu$</th>
<th>Second-Slope Period $T_d$ (sec)</th>
<th>Effective Period $T_p$ (sec)</th>
<th>Design Displacement $d$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.053</td>
<td>4.00</td>
<td>3.5</td>
<td>27</td>
</tr>
<tr>
<td>5</td>
<td>0.053</td>
<td>4.00</td>
<td>3.8</td>
<td>42</td>
</tr>
<tr>
<td>8</td>
<td>0.027</td>
<td>4.20</td>
<td>4.2</td>
<td>51</td>
</tr>
</tbody>
</table>
CHAPTER 4

Performance and Sensitivity Study

4.1 General

This chapter presents the performance and sensitivity studies of floor isolation systems (FISs) within multistory steel plate shear wall frames (SPSW) subjected to design and maximum considered earthquake ground motions. Four metrics have been employed to characterize the performance of this FIS-SPSW system: (i) the SPSW peak story drift ratio; (ii) the SPSW peak floor absolute acceleration; (iii) the FIS peak isolator displacement; and (iv) the peak FIS equipment absolute acceleration. The sensitivity study was conducted to investigate sensitivity of FIS performance to variations in the strength, or strength and stiffness of the primary (SPSW) system parameters. Variations considered for two parameters of the SPSW: (i) the web plate yield strength, $F_y$, which affects overall strength of the system; and (ii) the web plate thickness, $t$, that affects both the overall stiffness and strength of the system.

4.2 Performance Study

4.2.1 General

The performance of the FIS-SPSW is evaluated using unidirectional nonlinear response-history analysis in OpenSees (McKenna et al. 2006). Vertical excitation is not considered in this study. The earthquake ground motions listed in Table 3.4 were utilized for the performance and sensitivity study. Two earthquake hazards were considered: (i) a design basis earthquake (DBE) whereby the ground motions were amplitude scaled by 2/3 as discussed in Chapter 3 and (ii) maximum considered earthquake (MCE) where the ground motions were left unscaled.

Floor Isolation systems were designed (see Chapter 3) and modeled at two locations within 3-story SPSW (3M) frame and three locations within a 9-story SPSW (9M) frame. Table 4.1 presents a summary of the FIS-SPSW ten combinations analyzed for the performance and sensitivity studies.
Table 4.1 FIS-SPSW analysis combinations

<table>
<thead>
<tr>
<th>Frame</th>
<th>FIS Level</th>
<th>Earthquake hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3M</td>
<td>1</td>
<td>DBE and MCE</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>DBE and MCE</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>DBE and MCE</td>
</tr>
<tr>
<td>9M</td>
<td>5</td>
<td>DBE and MCE</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>DBE and MCE</td>
</tr>
</tbody>
</table>

Fig. 4.1 presents a schematic of the SPSW3M model showing the notation used to identify the acceleration and displacement response at a particular level. For example, L0 indicates the base and ground level, L1 the first floor level, and so on. The equipment level is denoted as LE. Response quantities of interest were the absolute accelerations of each floor by adding the relative acceleration from OpenSees to the ground acceleration. Also of interest for the SPSW frame was the interstory drift ratio that was obtained directly from OpenSees, but can also be determined from the relative displacement between two adjacent floor levels divided by the story height. The response of the floor isolation system was assessed using the absolute acceleration of the equipment that was determined by adding the relative acceleration from the OpenSees to the ground motion. Also of interest was the relative displacement across the isolator element.

Fig. 4.2 presents a sample hysteretic (force-displacement) response of the FIS system on the second floor of SPSW3M from response history analysis using La21 scaled by 2/3. The peak displacement of the FIS is 12 in. Peak relative isolator displacement were determined from the response history analysis and used to summarize performances.

Fig. 4.3 presents sample absolute acceleration histories from L0, L1, L2, L3 and LE of SPSW3M from La21 scaled by a 2/3 factor. From Fig. 4.3 it can be seen that the FIS significantly reduced absolute acceleration demand of the equipment and that under La21 scaled by 2/3, the absolute acceleration does not exceed the 0.3g limit. Peak absolute acceleration values determined from the absolute acceleration response histories are use to summarize performance.
Fig. 4.1 Schematic of SPSW3M showing level notation

Figure 4.2 Hysteretic behavior for the isolators on level 1 for DBE La21
Fig. 4.3 Performance of SPSW3M and FIS on level 1 for DBE La21
4.2.2 FIS in SPSW3M

Fig. 4.4 presents the peak absolute acceleration and peak drift ratio response of the SPSW3M from response history analysis using 20 motions at DBE and MCE levels. From Fig. 4.4a the SPSW drift ratio does not exceed the 2% limit specified in ASCE 7 (ASCE 2010) shown by a dashed vertical line. From Fig. 4.4b amplification of the absolute floor acceleration is observed up the height of the SPSW frame with peak acceleration of approximate 1.5g. From Fig. 4.4c, significant drift ratios are observed in the SPSW with the first story exceeding the drift ratio limit for 11 out of 20 ground motions; the second story exceeding 10 out of 20 and the third floor 9 out of 20.

Fig. 4.5 presents the peak isolator displacement (Fig. 4.5a and 4.5c) and peak equipment absolute acceleration (Fig. 4.5b and 4.5d) for FIS at level 1 and 2. From Fig. 4.5b, the absolute equipment acceleration does not exceed the limit of 0.3g shown by a vertical dashed line for any of the ground motions at the DBE level. However, the peak isolator displacement demands are significant with values exceed 30 in as shown in Fig. 4.5a. From Fig. 4.5d, under the MCE scenario the equipment absolute acceleration demands exceed the 0.3g limit at level 1 and 2, though the average values of 0.23g and 0.27g, respectively, are below the limit. Under the MCE scenario. The average of equipment absolute acceleration is shown using the sign "+" from Fig. 4.5. Isolation displacement demands are quite large reaching 60 in for one ground motion with average values of 51 in and 58 in, for the 2nd and 3rd floor respectively.

![Fig. 4.4 SPSW3M frame response under DBE and MCE: (a) DBE story drift ratio (b) DBE story absolute acceleration (c) MCE story drift ratio (d) MCE story absolute acceleration](image-url)
4.2.3 FIS in SPSW9M

Fig. 4.6 presents the peak absolute acceleration and peak drift ratio response of SPSW9M from response history analysis using 20 ground motions at DBE and MCE levels. From Fig. 4.6a, the drift ratio demands exceed the 2% limit specified in ASCE 7 (ASCE 2010) though the average is below the limit which meets the requirement by ASCE 7 (ASCE 2010). From Fig. 4.6b, significant amplifications of the floor absolute acceleration are observed up the height of the SPSW frame with the peak value of 4g. From Fig. 4.6c, under MCE scenario, the peak drift ratio demands of the SPSW9M frame are large with the average of peak drift ratio at each story exceeding the 2% limit. From Fig. 4.6d, large absolute acceleration responses are observed that for a few ground motions approach 5g.

Fig. 4.7 presents the peak isolation displacement (Fig. 4.7a and 4.7c) and peak equipment absolute acceleration (Fig. 4.7b and 4.7d) for FIS systems at level 3, 5 and 8 in SPSW9M. The average values of peak equipment absolute acceleration are shown as diamond in Fig. 4.7b and 4.7d. From Fig. 4.7b, though the average of the peak equipment absolute acceleration does not exceed the 0.3g limit shown as the dashed line, exceedence under some ground motions are observed. While the average values of equipment absolute acceleration are 0.19g, 0.24g and 0.24g, the number of exceedence are 2 out of 20, 6 out of 20 and 5 out 20 ground motions for the equipments on 3rd, 5th and 8th floor, respectively. From Fig. 4.7a, the isolator displacement demands are 52 in, 73 in, and 80 in on 3rd, 5th and 8th floor, respectively. From
Fig. 4.7d, under the MCE scenario, the average value of absolute equipment acceleration demand is of 0.36g for 5th and 8th floor exceeding the 0.3g limit. The limit is exceeded 10 out of 20 times for the FIS on the 5th floor and 8 out of 20 times for the FIS on the 8th floor. While the average of the equipment absolute acceleration on the 3rd floor is still below the limit of 0.3g the limit is exceeded for 4 out of 20 ground motions. From Fig. 4.7c, the peak isolator displacement demands reached 100 in.

Fig. 4.6 SPSW9M frame response under DBE and MCE: (a) DBE story drift ratio (b) DBE story absolute acceleration (c) MCE story drift ratio (d) MCE story absolute acceleration
Fig. 4.7 FIS response within SPSW9M: (a) DBE isolator displacement (b) DBE equipment absolute acceleration (c) MCE isolator displacement (d) MCE equipment absolute acceleration
4.2.4 Summary of Performance Study

The main primary results from the performance study of floor isolations systems (FISs) in steel plate shear wall frames (SPSWs) for all scenarios are summarized as follow:

1. The floor isolation system is effective in limiting the acceleration demand in 3-story SPSW under design basis earthquake (DBE).
2. Average equipment absolute acceleration does not exceed the 0.3g limit for FISs in 3-story SPSW under maximum considered earthquake (MCE) though the limit is exceeded for some individual ground motions.
3. The isolator displacement demand in both DBE and MCE is large with a maximum under MCE approaching 60 in.
4. Average equipment absolute acceleration exceeds the 0.3g limit for FISs in the 9-story SPSW under DBE scenario.
5. Average equipment absolute acceleration exceeds the 0.3g limit for FISs in the 9-story SPSW under MCE scenario.
6. The isolator displacement demand in both DBE and MCE is large with a maximum under MCE approaching 100 in.
4.3 Sensitivity Study

A sensitivity study was performed to identify how variations in the SPSW strength and stiffness affect the performance of the FIS and the acceleration sensitive equipment. In this study variations were considered for two parameters of the primary structure: web plate yield strength $F_y$ (affecting the structural strength) and web plate thickness $t$ (affecting both the structural strength and stiffness). The material yield strength, $F_y$, was chosen because the values specified by ASTM (American Society For Testing & Materials) are minimum required values. However the actual yield strength can be significantly large. For example, from a study by Bartlett (2003) from testing 131 coupons of ASTM A992 steel, the average and maximum yield strength were 55 ksi and 64 ksi respectively, while the minimum specified by ASTM (ASTM 2009) is 50 ksi. Furthermore, modifying $F_y$ only affects the strength of the SPSW system.

The web plate thickness was also chosen since variability can be expected because of the mill tolerance permitted by ASTM (ASTM 2010) on sheet ASTM A568 and plate material. Additionally, modifying the web plate thickness ($t$) affects both the strength and stiffness of the system for a given yield strength. Table 4.2 presents the yield ($F_y$) and tensile ($F_t$) strength considered for the sensitivity study. Also tabulated in Table 4.2 is the web plate yield strength ratio ($F_{yi} / F_{yn}$) that is the varied value ($F_{yi}$) divided by the nominal value ($F_{yn}$) that ranged from 0.8 to 2.1 Table 4.3 presents nominal ($t_n$) and varied ($t_i$) values of the web plate thickness for each of the three web plates used for SPSW3M. Table 4.2 also presents the web plate thickness ratios ($t_i / t_n$) that varies from 0.52 to 1.51. Table 4.4 provides varied ($t_i$) and nominal ($t_n$) values for each web plate thickness ($t_i - t_n$). Also presented are the varied-to nominal ratio ($t_i / t_n$) that ranged from 0.52 to 1.51.
Table 4.2 Strength values used for sensitivity study for SPSW3M and SPSW9M

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4.3.1 Results of Sensitivity Study

This section presents summary results from the sensitivity study of the SPSW3M and SPSW9M with floor isolations system (FIS) installed at various level. Detailed results from individual analysis for each of the frame and FIS locations are presented in Appendix C. The procedure for the sensitivity study is as follows:

1. Perform twenty response history analyses for SPSW frame and FIS using nominal values at DBE level.
2. Nominal peak response values for isolator displacement, equipment absolute acceleration and floor absolute acceleration are obtained for the SPSW-FIS response from OpenSees.
3. Perform twenty analyses at DBE level with the $i^{th}$ value of the varied parameter (either $F_y$ or $t$).
4. Peak response result were obtained and the following response ratios were calculated:

$$\text{Isolator. Disp. Ratio} = \frac{\text{Peak Disp. for } i^{th} \text{ Parameter}}{\text{Peak Disp. for Nominal Value}}$$

$$\text{Equipment Abs. Acc. Ratio} = \frac{\text{Peak Eq. Abs. Acc. for } i^{th} \text{ Parameter}}{\text{Peak Eq. Abs. Acc. for Nominal Value}}$$

$$\text{Floor Abs. Acc. Ratio} = \frac{\text{Peak Floor Abs. Acc. for } i^{th} \text{ Parameter}}{\text{Peak Floor Abs. Acc. for Nominal Value}}$$

5. Repeat step 3 and 4 for all value of varied parameters listed in Table 4.1-4.3.
6. Repeat 1-5 for MCE level ground motions.

Fig. 4.8 presents a sample sensitivity response from the SPSW3M with the FIS on Floor 2 to variations in the web plate strength at the DBE level. The strength range is considered from 0.8 to 2.1 times the nominal yield and tensile strength. Fig. 4.8a presents the sensitivity of the isolator displacement to variation in the web plate strength. Fig. 4.8b presents the sensitivity of the equipment absolute acceleration to the web plate yield and tensile strength variation. Fig. 4.8c shows the peak response from the individual analysis (shown by a dot), the median response (shown by thick solid line) and the envelop
of the response (thin solid line). While there is some change in isolator displacement with variation in web plate strength, the median response appears insensitive to variation in yield strength $F_y$. Fig. 4.8.b shows the equipment absolute acceleration response to variations in yield strength $F_y$. Again the absolute acceleration demands on the equipment appear to be insensitive to $F_y$. Fig. 4.8.c presents absolute acceleration demand of the SPSW floor beneath the FIS system. From Fig. 4.8.c the floor absolute acceleration demands act relatively sensitive to $F_y$ with a 1.5 increase in floor acceleration when $F_y$ is doubled. The sensitivity of the primary floor system demand is sensitive to variations in $F_y$ and $t$ can be attributed to the increase in overall system strength and stiffness.
Fig. 4.8 Sensitivity study of response of SPSW3M with FIS on level 1 to variations in the web plate strength $F_y$ under DBE
Due to the large number of simulations, a more compact graphical presentation of the results was developed that shows only the median, maximum and minimum results for each case. Detailed results for floor isolation systems on level 1, 2 and level 3, 5, and 8 for 3-story and 9-story steel plate shear walls with variations in web plate strength $F_y$ and plate thickness $t$ are presented in Appendix C.

Fig. 4.9 presents median, max and min value for sensitivity analysis of SPSW3M to variations in the web plate yield strength for the level 1 under DBE (3M/1/D), level 1 under MCE (3M/1/M), level 2 under DBE (3M/2/D) and level 2 under MCE (3M/2/M). The median values are shown by a symbol with a line whereas the max. and min. values are shown by a corresponding smaller symbol. From Fig. 4.9a and 4.9b, the median isolator displacement and median equipment absolute acceleration are insensitive to the variation in $F_y$. However, the max/min isolator displacement values are somewhat sensitive under the MCE event with isolator displacements of 1.6 times nominal for $F_{yi}/F_{yn} = 1.6$. From Fig. 4.9c, the median and max/min values for floor absolute acceleration show sensitivity to variation in the web plate yield strength. The median response is 1.4 times the nominal value for $F_{yi}/F_{yn} = 2.1$. The maximum values for level 1 are somewhat more sensitive than level 2 under the MCE scenario.

Fig. 4.10 presents the median, max. and min. value for sensitivity analysis of SPSW3M to variations in the web plate thickness $t$ for all cases. Fig. 4.11 and 4.12 present the median, max. and min. value for sensitivity study of SPSW9M to variations in the web plate yield strength $F_y$ and thickness $t$. From Fig. 4.10a, the median isolator displacement ratio appears to be insensitive to variations in the web plate thickness. There is some sensitivity of peak values of isolator displacement at level 1 and level 2 at MCE level with the increase of 1.5 times the nominal value. From Fig. 4.10b, the equipment absolute acceleration is insensitive to the web plate thickness variations. From Fig. 4.10c, the max/min floor absolute acceleration for level 1 is more sensitive than level 2 at both DBE and MCE level with the increase of 2 times the nominal value for $t_{yi}/t_{yn} = 1.48$.

From Fig. 4.11a, the median and the max/min isolator displacements are insensitive to the web plate yield strength $F_y$. From 4.11b, the median equipment acceleration is insensitive to the web plate yield strength $F_y$, whereas the peak acceleration for the equipment on the level 3 under MCE scenario was most sensitive to the web plate strength variation with the increase of 1.5 times the nominal value for $F_{yi}/F_{yn} > 1.26$. From Fig. 4.11c, the max/min values for floor absolute acceleration are sensitive to the web plate yield strength at DBE and MCE level for level 3, 5 and 8 with increase of 4.5 times the nominal value for $F_{yi}/F_{yn} = 1.75$. 
For variations in the web plate thickness ($t_i$) in the SPSW9M frame the median isolator displacement and equipment accelerations shown in Fig. 4.12a and 4.12b are again insensitive to the variations however the maximum and minimum values show some sensitivity. The median and max/min SPSW floor absolute acceleration appear to be sensitive to variations in the web plate thickness as expected.

4.3.2 Summary of Sensitivity Study

The main primary results from the sensitivity study are summarized as follows:

1. Equipment absolute acceleration ratio is insensitive to the variations in web plate yield strength $F_y$ and plate thickness $t$ of the steel plate shear walls.

2. The median values of the isolator displacement ratio are insensitive to $F_y$ and $t$, although maximum values are somewhat sensitive.

3. Floor absolute acceleration ratio is sensitive to $F_y$ and $t$ as expected because both $F_y$ and $t$ affect overall system strength therefore the performance of the system.
Fig. 4.9 Median, max and min sensitivity of response of SPSW3M to variations in the web plate yield strength $F_y$ for all cases
Fig. 4.10 Median, max and min sensitivity of response of SPSW3M to variations in the web plate thickness $t$ for all cases
Fig. 4.11 Median, max and min sensitivity of response of SPSW9M to variations in the web plate yield strength $F_y$ for all cases
Fig. 4.12 Median, max and min sensitivity of response of SPSW9M to variations in the web plate yield strength $t$ for all cases
CHAPTER 5

Summary and Conclusions

5.1 Summary

In the US, there is a growing interest in utilizing floor isolation systems to protect nonstructural components during earthquakes. However, most of the research focused on the mechanical behavior of the floor isolation system itself and not on the performance within the building. This study investigated the performance of the floor isolation systems within multistory steel plate shear wall (SPSW) frames under the design basis (DBE) and maximum considered (MCE) earthquake event. The concept of pairing the SPSW with FIS is that SPSW might effectively limit drift, protecting drift sensitive components, whereas the floor isolation system reduce acceleration demands on targeted acceleration sensitive components.

In this study, the floor isolation systems (FISs) were first designed for the level 1 and 2 in a 3-story steel plate shear wall frame (SPSW3M) and for the level 3, 5 and 8 in a 9-story steel plate shear wall frame (SPSW9M). The 3- and 9-story SPSWs were developed by Berman (Berman 2011) and represent the usual type of low and mid-height buildings.

Floor isolation systems (FISs) were then modeled in steel plate shear walls using Opensees (McKenna et al. 2006) and nonlinear time history analyses were utilized to evaluate the performance by identifying the peak response of the FISs and SPSW frames for design basis (DBE) and maximum considered (MCE) ground motion event. The twenty MCE ground motions were taken from SAC (SEAOC, ATC and CUREE) steel project (SAC 1997) for the Los Angeles area with the probability of exceedence of 2% in 50 years and assumed to represent the MCE hazard. The twenty DBE ground motions were obtained by scaling the MCE motions by 2/3.

A sensitivity study was then performed to investigate the effect that variations in the web plate strength $F_y$ and thickness $t$ effect have on the performance of the FISs in SPSWs.

5.2 Conclusions

The conclusion based on the performance and sensitivity of numerical models of FISs in SPSWs and do not reflect FIS performance in the other framing systems. Based on the results, the following conclusions are drawn:
1. The floor isolation system (FIS) is effective in limiting acceleration demand on equipment in SPSW frame under DBE and median demands under MCE. Though performance is best on the lower levels.

2. The isolator displacement demands are large enough that it would be difficult to accommodate using existing hardware.

3. The normalized strength ratio \( \frac{Q_s}{W} \) required for the level 5 of the 9-story SPSW were lower \( (\mu = 0.03) \) than is possible with typical unlubricated PFTE (polytetrafluoroethylene)-stainless steel bearing material used in friction pendulum bearing.

4. The median response of the FIS is insensitive to the variations of the structure strength and stiffness, though the performance limit could be exceeded for individual ground motions.

5. The median isolator displacement demand was insensitive to variations in strength and stiffness but the maximum were not so that maximum should be considered when designing the displacement capacity of the device.

6. The SPSW floor absolute acceleration was sensitive to variations in the web plate parameters as would be expected. This result suggests that significant damage to rigidly attached equipment in SPSWs might be expected under DBE or MCE events.

5.2 Recommendations

Following conclusion 2, damping and control strategies should be investigated to reduce isolator displacement demands.
REFERENCE


APPENDIX A

Period Calculation for Steel Plate Shear Wall Frames

A.1 General
This appendix presents a method to estimate the first three modal periods of steel plate shear walls accounting for shear and shear flexural interaction. Also the first three periods of 3-story SPSW (SPSW3M) and 9-story SPSW (SPSW9M) are calculated as verification for the results from the Opensees (McKenna et al. 2006).

The modal frequencies of SPSWs are evaluated using Dunkerley's equation (Thomson And Dahleh 1998):

\[
\frac{1}{\omega^2} \approx \frac{1}{\omega_F^2} + \frac{1}{\omega_S^2}
\]

where \( \omega_F \) and \( \omega_S \) are the circular frequencies due to flexural and shear-flexure action, respectively.

A.2 Shear Mode Frequency
The shear modal frequency (\( \omega_s \)) of the SPSW is estimated from an eigenvalue analysis of idealizing the SPSW as a lumped mass and stiffness system. The individual story stiffness is composed of contributions from the frame and the infill panels. The frame stiffness is calculated using:

\[
k_{cf} = 2\left(\frac{12EI}{h^3}\right)
\]

which assumes the beams are rigid. The infill panel story stiffness is estimated using the following equation from Thorburn and Kulak (1983) for a fully developed tension field:

\[
k_{pi} = \frac{E \cdot W_{panel} \cdot t}{H_{panel}} \cdot \sin^2 \alpha \cdot \cos^2 \alpha
\]

where \( \alpha \) is the inclination angle from vertical of the tension field in a SPSW infill plate (Thorburn and Kulak 1983) calculated as:

\[
\alpha = \arctan\left(\sqrt{\frac{1 + \frac{tL}{2A_t}}{1 + th\left(\frac{1}{A_b} + \frac{h^3}{360I_tL}\right)}}\right)
\]
where \( A_c \) and \( A_b \) is the cross-sectional area of the column and beam, respectively, at each level; \( W_{\text{panel}} \) is the infill panel width; \( H_{\text{panel}} \) is the panel height and \( t \) is the panel thickness; \( L \) is the bay width; \( I_c \) is the moment of inertia of the vertical boundary elements and \( h \) is the story height.

The total story stiffness is approximate as the sum of the frame (i.e. columns) and infill panel stiffness as:

\[
k_i = k_{ci} + k_{pi}
\]

\( (A.5) \)

where \( k_{ci} \) is the column stiffness and the \( k_{pi} \) is the panel stiffness at each level, respectively.

The total stiffness matrix \( K \) becomes:

\[
K = \begin{bmatrix}
k_1 + k_2 & -k_2 & 0 & 0 & \cdots & 0 \\
-k_2 & k_2 + k_3 & -k_3 & 0 & \cdots & 0 \\
0 & -k_3 & k_3 + k_4 & -k_4 & \cdots & 0 \\
0 & \ddots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & 0 & -k_{n-1} & k_{n-1} + k_n & -k_n \\
0 & \cdots & 0 & -k_n & k_n & \end{bmatrix}
\]

\( (A.6) \)

and \( M \) is the lumped mass matrix, for a \( n \)-story building is:

\[
M = \begin{bmatrix}
m_1 & 0 & \cdots & 0 & 0 \\
0 & m_2 & 0 & \cdots & 0 \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
0 & \cdots & 0 & m_{n-1} & 0 \\
0 & 0 & \cdots & 0 & m_n \\
\end{bmatrix}
\]

\( (A.7) \)

With the stiffness and mass matrices, the shear modal frequencies are obtained from an eigenvalue analysis as:

\[
\begin{bmatrix}K - \lambda M\end{bmatrix}\Phi = 0
\]

\( (A.8) \)

where \( \Phi \) is the deflected shape and does not vary with time. The eigenvalue \( \lambda \) can be obtained from the determinate of Eq.8.

\[
\det\left[K - \lambda M\right] = 0
\]

\( (A.9) \)

The shear modal frequency (\( \omega_i \)) are obtained from the eigenvalue analysis using:
\[ \lambda = \omega_s^2 \]  

(A.10)

From which the shear modal period \( T_s \) is calculated as follows:

\[ T_s = \frac{2\pi}{\omega_s} \]  

(A.11)

### A.3 Flexural Mode Frequency

The SPSW is approximated as a cantilever with uniformly distributed mass and stiffness to estimate the flexural frequency. The frequencies of the first three mode of a cantilever (Clough and Penzien 1975) are:

\[ \omega_{F1} = 1.875^2 \sqrt{\frac{E I}{\bar{m} H^4}} \]  

(A.12a)

\[ \omega_{F2} = 4.694^2 \sqrt{\frac{E I}{\bar{m} H^4}} \]  

(A.12b)

\[ \omega_{F3} = 7.855^2 \sqrt{\frac{E I}{\bar{m} H^4}} \]  

(A.12c)

where \( \omega_{F1}, \omega_{F2}, \text{ and } \omega_{F3} \) are first, second and third vibration modes, \( \bar{m} \) is the mass per unit length, and \( H \) is the length of the cantilever in these equations and here taken as the total height of the SPSW.

For the flexural frequency calculation it is assumed that the columns act as the "flanges" and the infill panel the "web". The moment of inertia, ignoring the infill panel, is calculated as:

\[ I = 2I_c + \frac{A_i L^2}{2} \]  

(A.13)

where \( I_c \) and \( A_i \) are the moment of inertia and section area of the columns, \( L \) is the bay width of the frame.

Because the column sections reduce up the height of the frame, and Eq. A.12 assumes a prismatic member, the average column moment of inertia \( I_c \) is used. The distributed mass \( \bar{m} \) is calculated as the sum of the mass at each level divided by the total height of the frame.

The circular frequency of the SPSW accounting for shear and flexural is then estimated using Eq. A.1 with \( \omega_s \) and \( \omega_{Fj} \) for a particular mode.
A.4 SPSW3M Period Calculation

This section illustrates the frequency estimation methodology for a 3 story SPSW (SPSW3M) (Berman 2010). The section properties for the SPSW3M frame are presented in Table A.1.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column Section (VBE)</th>
<th>Beam Section (HBE)</th>
<th>Web Plate (in)</th>
<th>H (ft)</th>
<th>L (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14x398</td>
<td>W27x178</td>
<td>0.125</td>
<td>13</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>W14x233</td>
<td>W24x131</td>
<td>0.125</td>
<td>13</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>W14x233</td>
<td>W24x131</td>
<td>0.0623</td>
<td>13</td>
<td>15</td>
</tr>
</tbody>
</table>

Using Eq. A.2, A.3 and A.4, the frame, panel and story stiffness of the SPSW3M were calculated. The result of the calculation is summarized in Table A.2.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column Stiffness $k_{ci}$ (kip/in)</th>
<th>Panel Stiffness $k_{pi}$ (kip/in)</th>
<th>Story Stiffness $k_{i}$ (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1099.98</td>
<td>1039.09</td>
<td>2139</td>
</tr>
<tr>
<td>2</td>
<td>623.32</td>
<td>1040.89</td>
<td>1664</td>
</tr>
<tr>
<td>3</td>
<td>623.32</td>
<td>520.40</td>
<td>1044</td>
</tr>
</tbody>
</table>

The floor seismic mass is 1.38 kips-sec^2/in so the mass matrix becomes:

$$
M = \begin{bmatrix}
1.38 & 0 & 0 \\
0 & 1.38 & 0 \\
0 & 0 & 1.38
\end{bmatrix}
$$

(A.14)

From the SPSW stiffness (Table A.2) and Eq. A.9, the stiffness matrix is:

$$
K = \begin{bmatrix}
3803 & -1664 & 0 \\
-1664 & 2808 & -1144 \\
0 & -1144 & 1144
\end{bmatrix}
$$

(A.15)

From Eq. A.6, A.7 and A.10, the shear mode frequency is calculated as:

$$
\omega_s = \begin{bmatrix}
16.0 \\
40.3 \\
61.2
\end{bmatrix}
rad/sec
$$

(A.16)

Using Eq. A.11, the first three shear mode periods are:
The flexural mode frequencies are estimated by calculating the average moment of inertia \( I_{\text{average}} \) using:

\[
I_1 = 2 \times \left( 6000 + 117 \times \left( \frac{180}{2} \right)^2 \right) = 1907400 \text{ in}^4
\] (A.18a)

\[
I_2 = 2 \times \left( 3400 + 75.6 \times \left( \frac{180}{2} \right)^2 \right) = 1231520 \text{ in}^4
\] (A.18b)

\[
I_3 = 2 \times \left( 3400 + 75.6 \times \left( \frac{180}{2} \right)^2 \right) = 1231520 \text{ in}^4
\] (A.18c)

\[
I_{\text{average}} = 1456813 \text{ in}^4
\] (A.19)

Using Eq. A.12, the first three flexural modes are:

\[
\omega_{F1} = (1.875)^2 \sqrt{\frac{29000 \times 1456813}{8.85 \times 10^{-3} \times 468^4}} = 35.07 \text{ rad/sec}
\] (A.20a)

\[
\omega_{F2} = (4.694)^2 \sqrt{\frac{29000 \times 1456813}{8.85 \times 10^{-3} \times 468^4}} = 219.8 \text{ rad/sec}
\] (A.20b)

\[
\omega_{F3} = (7.855)^2 \sqrt{\frac{29000 \times 1456813}{8.85 \times 10^{-3} \times 468^4}} = 615.5 \text{ rad/sec}
\] (A.20c)

where the mass per unit length is calculated as:

\[
\bar{m} = \left( 1.38 + 1.38 + 1.38 \right) / 468 = 8.85 \times 10^{-3} \text{ kips-sec}^2 / \text{in}^2
\] (A.21)

Using Dunkerley's equation (Eq.A.1), the shear frequency and flexural frequency are combined to estimate the first three mode frequencies of the SPSW as:

\[
\frac{1}{\omega_i^2} = \frac{1}{16.0^2} + \frac{1}{35.07^2}
\] (A.22a)
\[ \frac{1}{\omega^2} = \frac{1}{40.3^2} + \frac{1}{219.8^2} \]  
(A.22b)

\[ \frac{1}{\omega^2} = \frac{1}{61.2^2} + \frac{1}{615.5^2} \]  
(A.22c)

\[ \omega = \begin{bmatrix} 14.6 \\ 39.6 \\ 60.9 \end{bmatrix} \text{ rad/sec} \]  
(A.23)

The first three modal periods are calculated using Eq. A.11:

\[ T = \begin{bmatrix} 0.43 \\ 0.16 \\ 0.10 \end{bmatrix} \text{ sec} \]  
(A.24)

The estimated periods for the first three modes are summarized in Table A.3 accounting for shear (Eq. A.8) and shear-flexural (Eq. A.1). These values are also compared to periods estimated from a finite element model of the 3 story SPSW using OpenSees (McKenna and Fenves 2006).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Opensees Period (sec)</th>
<th>Shear-Flexural Period (sec)</th>
<th>Shear Frequency (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.39</td>
<td>0.43</td>
<td>0.39</td>
</tr>
<tr>
<td>2</td>
<td>0.14</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td>3</td>
<td>0.08</td>
<td>0.10</td>
<td>0.10</td>
</tr>
</tbody>
</table>

From Table A.3, for the 3 story steel plate shear wall frame, the methodology to estimate the periods considering shear mode only using Eq. A.8 agrees better with the periods from the Opensees. For the 3 story SPSW there is a little difference between shear and shear flexural indicating that flexural are neglectable.

### A.5 SPSW9M Period Calculation

This section presents a summary of the frequency estimation methodology using Dunkerley’s equation for a 9-story SPSW frame (SPSW9M). The section properties for the SPSW9M frame are presented in Table A.4.
Table A.4 SPSW9M section properties

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column Section (VBE)</th>
<th>Beam Section (HBE)</th>
<th>Web Plate (in)</th>
<th>H (ft)</th>
<th>L (ft)</th>
<th>Mass (k•sec²/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14x730</td>
<td>W18x86</td>
<td>0.25</td>
<td>13</td>
<td>15</td>
<td>0.96</td>
</tr>
<tr>
<td>2</td>
<td>W14x730</td>
<td>W18x86</td>
<td>0.25</td>
<td>13</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td>3</td>
<td>W14x500</td>
<td>W24x146</td>
<td>0.25</td>
<td>13</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td>4</td>
<td>W14x500</td>
<td>W18x86</td>
<td>0.187</td>
<td>13</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td>5</td>
<td>W14x500</td>
<td>W18x86</td>
<td>0.187</td>
<td>13</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td>6</td>
<td>W14x500</td>
<td>W24x146</td>
<td>0.125</td>
<td>13</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td>7</td>
<td>W14x370</td>
<td>W16x57</td>
<td>0.125</td>
<td>13</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td>8</td>
<td>W14x370</td>
<td>W24x146</td>
<td>0.0626</td>
<td>13</td>
<td>15</td>
<td>1.01</td>
</tr>
<tr>
<td>9</td>
<td>W14x370</td>
<td>W18x106</td>
<td>0.0626</td>
<td>13</td>
<td>15</td>
<td>1.01</td>
</tr>
</tbody>
</table>

Using Eq. A.2, A.3 and A.4, the frame (i.e., columns), panel and story stiffness of the SPSW9M are calculated. A summary of the results are provide in Table A.5.

Table A.5 SPSW9M story stiffness calculation summary

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column Stiffness $k_{ci}$ (kips/in)</th>
<th>Panel Stiffness $k_{pi}$ (kips/in)</th>
<th>Total Stiffness $k_{i}$ (kips/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2621.6</td>
<td>2003.2</td>
<td>4624.8</td>
</tr>
<tr>
<td>2</td>
<td>2621.6</td>
<td>2003.2</td>
<td>4624.8</td>
</tr>
<tr>
<td>3</td>
<td>1505.1</td>
<td>2058.6</td>
<td>3563.8</td>
</tr>
<tr>
<td>4</td>
<td>1505.1</td>
<td>1522.7</td>
<td>3027.8</td>
</tr>
<tr>
<td>5</td>
<td>1505.1</td>
<td>1522.7</td>
<td>3027.8</td>
</tr>
<tr>
<td>6</td>
<td>1505.1</td>
<td>1548.0</td>
<td>3053.2</td>
</tr>
<tr>
<td>7</td>
<td>997.3</td>
<td>1016.7</td>
<td>2014.0</td>
</tr>
<tr>
<td>8</td>
<td>997.3</td>
<td>1040.7</td>
<td>2038.0</td>
</tr>
<tr>
<td>9</td>
<td>997.3</td>
<td>522.1</td>
<td>1519.4</td>
</tr>
</tbody>
</table>

and the mass matrix becomes:

\[
M = \begin{bmatrix}
0.96 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0.94 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0.94 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.94 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0.94 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0.94 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0.94 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.94 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1.01
\end{bmatrix} \text{k•sec}^2\text{in}^{-1} \tag{A.25}
\]
The total mass is calculated as the sum of the mass on each floor:

\[ m = 8.566 \text{ kips-sec}^2/\text{in} \] \hspace{1cm} (A.26)

and the mass per unit length is calculated as:

\[ \bar{m} = \frac{(0.958 + 0.942 \times 7 + 1.014)}{1404} = 6.1 \times 10^{-3} \text{ kips-sec}^2/\text{in}^2 \] \hspace{1cm} (A.27)

From the story stiffness (Table A.4) and Eq. A.9, the global stiffness matrix is:

\[
K = \begin{bmatrix}
9224 & -4612 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
-4612 & 8156 & -3544 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & -3544 & 6560 & -3016 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & -3016 & 6031 & -3016 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & -3016 & 6058 & -3043 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & -3043 & 5048 & -2006 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & -2006 & 4038 & -2032 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & -2032 & 3550 & -1518 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & -1518 & 1518 \\
\end{bmatrix} \text{ kips/in} \] \hspace{1cm} (A.28)

From an eigenvalue analysis (Eq. A.6, A.7 and A.10), the shear mode frequency is calculated as:

\[
\omega_i = \begin{bmatrix}
9.8 \\
26.2 \\
42.7 \\
56.0 \\
71.2 \\
82.3 \\
94.2 \\
107.0 \\
122.8 \\
\end{bmatrix} \text{ rad/sec} \] \hspace{1cm} (A.29)

Using Eq. A.11, the shear mode periods are:
and the first, second and third mode periods due to shear are 0.64, 0.24 and 0.15, respectively.

To estimate the flexural mode frequencies the average moment of inertia \( I_{\text{average}} \) is calculated as:

\[
 I_1 = 2 \times \left( 14300 + 215 \times \frac{180}{2} \right)^2 = 3511600 \ \text{in}^4 \\
 I_2 = 2 \times \left( 8210 + 147 \times \frac{180}{2} \right)^2 = 2397820 \ \text{in}^4 \\
 I_3 = 2 \times \left( 5440 + 109 \times \frac{180}{2} \right)^2 = 1776680 \ \text{in}^4
\]

\[ I_{\text{average}} = 2438280 \ \text{in}^4 \] (A.32)

Using Eq. A.12, the first three flexural modes are:

\[
 \omega_{F1} = (1.875)^2 \sqrt{ \frac{29000 \times 2438280}{6.1 \times 10^{-3} \times (1404)^4} } = 6.07 \ \text{rad/sec} \ \\
 \omega_{F2} = (4.694)^2 \sqrt{ \frac{29000 \times 2438280}{6.1 \times 10^{-3} \times (1404)^4} } = 38.1 \ \text{rad/sec} \ \\
 \omega_{F3} = (7.855)^2 \sqrt{ \frac{29000 \times 2438280}{6.1 \times 10^{-3} \times (1404)^4} } = 106.6 \ \text{rad/sec}
\]

Using Dunkerley’s equation (Eq.A.1), the shear frequency and flexural frequency can be combined to estimate the SPSW Frame frequencies for the first three modes:
\[ \frac{1}{\omega_1^2} = \frac{1}{9.8^2} + \frac{1}{6.07^2} \]  
(A.34a)

\[ \frac{1}{\omega_2^2} = \frac{1}{26.2^2} + \frac{1}{38.1^2} \]  
(A.34b)

\[ \frac{1}{\omega_3^2} = \frac{1}{42.7^2} + \frac{1}{106.6^2} \]  
(A.34c)

\[ \omega = \begin{bmatrix} 5.2 \\ 21.7 \\ 39.3 \end{bmatrix} \text{rad/sec} \]  
(A.35)

The first three modal periods are calculated using Eq. A.11:

\[ T = \begin{bmatrix} 1.22 \\ 0.29 \\ 0.16 \end{bmatrix} \text{sec} \]  
(A.36)

The estimated periods for the first three modes are summarized in Table A.6 accounting for shear (Eq.A.8) and shear-flexural (Eq. A.1). These values are also compared to periods estimated from a finite element model of the 9 story SPSW using OpenSees (McKenna and Fenves 2006).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Opensees Period (sec)</th>
<th>Shear-Flexural Period (sec)</th>
<th>Shear Frequency (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.14</td>
<td>1.22</td>
<td>0.64</td>
</tr>
<tr>
<td>2</td>
<td>0.31</td>
<td>0.29</td>
<td>0.24</td>
</tr>
<tr>
<td>3</td>
<td>0.16</td>
<td>0.16</td>
<td>0.15</td>
</tr>
</tbody>
</table>

For the 9 story SPSW, the periods estimated using Dunkerley’s equation to account for shear-flexural match well with the results from Opensees with the errors of 7% and 8% for the 1st and 2nd mode periods, respectively. Considering only shear deformations the first mode period is 0.64 sec. That has a 44% error related to the period from OpenSees indicating that the flexural deformation is need to be considered for the 9 story SPSW.
APPENDIX B

Floor Isolation Systems Design

B.1 General

This appendix presents the calculation for Floor isolation systems (FISs) design on level 2 for 3-story SPSW (SPSW3M) and level 3, 5 and 8 for 9-story SPSW (SPSW9M). The general equation Eq. 3.12 in Section 3.4.4 is utilized to design the FIS system. For each design, the floor spectrum with one iteration is provided with all other iterations summarized in the tables.

B.2 Design Example of FIS at level 2 in SPSW3M

Fig. B.1 presents the floor average response spectrum for level 2 in SPSW3M from time history analysis in Opensees (McKenna et al. 2006) using scaled ground motions. The design acceleration limit is 0.3g and the design target effective period is 3.47 sec. A second-slope period $T_d = 4.0 \text{ sec}$ was also assumed that for a friction pendulum bearing gives $K_d = 0.042 \text{ kips/in}$. Additionally a coefficient of friction of 0.07 ($\mu$) was assumed (Constantinou et al. 1999). $\mu = 0.07$ is a reasonable coefficient of friction for friction pendulum bearings.

![Fig. B.1 Floor average response spectrum for level 2 in SPSW3M](image)

To initiate the process, the characteristic strength is calculated as:
\[ Q_d = \mu W = 0.07 \times 6.5 \text{ kips} = 0.455 \text{ kips} \]  

(A.1)

A FIS displacement of 20 in is assumed and the effective stiffness of a FIS isolator is calculated as:

\[ K_{eff} = \frac{0.07 \times 6.5 \text{ kips}}{20 \text{ in}} + 0.042 \text{ kips/in} = 0.0648 \text{ kips/in} \]  

(A.2)

From which \( T_{eff} \) is calculated as:

\[ T_{eff} = 2\pi \sqrt{\frac{W}{K_{eff} \cdot g}} = 2\pi \sqrt{\frac{6.5 \text{ kips}}{0.0648 \text{ kips/in} \times 386.4 \text{ in/sec}^2}} = 3.20 \text{ sec} \]  

(A.3)

The effective damping ratio is calculated as:

\[ \beta_{eff} = \frac{2Q_d}{\pi \cdot K_{eff} \cdot d_0} = \frac{2 \times 0.455 \text{ kips}}{\pi \times 0.0648 \text{ kips/in} \times 20 \text{ in}} = 0.224 \]  

(A.4)

By interpolation from Table 3.5 in Section 3.4.3, the damping coefficient \( B_D \) is determined to be 1.55.

The updated displacement is calculated using Eq. 3.11 as:

\[ d_i = \frac{10S_A(T_{eff}) \cdot T_{eff}^2}{B_D} = \frac{10 \times 0.309 \times (3.20)^2}{1.55} = 20.41 \text{ in} \]  

(A.5)

where \( S_A(3.20) = 0.309 g \) is the spectrum ordinate at the period of 3.20 sec obtained from Fig. B.1. The relative error in designed displacement is:

\[ \text{Relative Error} = \left| \frac{d_i - d_d}{d_i} \right| \times 100\% = \left| \frac{20.41 \text{ in} - 20 \text{ in}}{20.41 \text{ in}} \right| \times 100\% = 2\% \]  

(A.6)

One more iteration was performed so that the relative error for the displacement estimation is less than 2% and the design displacement is 20.53 in. After one iteration the design displacement of \( d = 20.53 \text{ in} \) results in an effective period of \( T_{eff} = 3.22 \text{ sec} \) and the design acceleration of \( S_A(3.22) = 0.305 g \) for the isolation system with \( \mu = 0.07 \) and \( T_d = 4.0 \text{ sec} \). Table B.1 presents the summary of the one iteration.
Table B.1 Summary of design for FIS at level 2 in SPSW3M

<table>
<thead>
<tr>
<th>Iteration</th>
<th>Design displacement (in)</th>
<th>Effective stiffness $K_{eff}$ (kips/in)</th>
<th>Effective Period $T_{eff}$ (sec)</th>
<th>Damping ratio $\beta_{eff}$</th>
<th>Damping coefficient $B_p$</th>
<th>Updated displacement (in)</th>
<th>Relative error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Estimation</td>
<td>20in</td>
<td>0.0648</td>
<td>3.20</td>
<td>0.224</td>
<td>1.55</td>
<td>20.41</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>20.41</td>
<td>0.64</td>
<td>3.22</td>
<td>0.222</td>
<td>1.54</td>
<td>20.53</td>
<td>0.58</td>
</tr>
</tbody>
</table>

B.3 Design Example of FIS at level 3 in SPSW9M

Fig. B.2 presents the floor average response spectrum for level 3 in SPSW9M from time history analysis using scaled ground motions (DBE), the design acceleration limit is 0.3g and the design target effective period is 3.26 sec. A second-slope period $T_d = 4.0$ sec was assumed that for a friction pendulum bearing gives $K_d = 0.042$ kips/in. Additionally a coefficient of friction of 0.053 ($\mu$) was assumed.

![Floor Spectrum](image)

Fig. B.2 Floor average response spectrum for level 3 in SPSW9M

To initiate the process, the characteristic strength is calculated as:

$$Q_d = \mu W = 0.053 \times 6.5 \text{ kips} = 0.345 \text{ kips}$$

A FIS displacement of 30 in is assumed and the effective stiffness of a FIS isolator is calculated as:
\[ K_{\text{eff}} = \frac{0.053 \times 6.5 \text{ kips}}{30 \text{ in}} + 0.042 \text{ kips/in} = 0.053 \text{ kips/in} \]  \hspace{1cm} (B.8)

From which \( T_{\text{eff}} \) is calculated as:

\[ T_{\text{eff}} = 2\pi \sqrt{\frac{W}{K_{\text{eff}} \cdot g}} = 2\pi \sqrt{\frac{6.5 \text{ kips}}{0.053 \text{ kips/in} \times 386.4 \text{ in/sec}^2}} = 3.54 \text{ sec} \]  \hspace{1cm} (B.9)

The effective damping ratio is calculated as:

\[ \beta_{\text{eff}} = \frac{2Q_d}{\pi \cdot K_{\text{eff}} \cdot d_0} = \frac{2 \times 0.053 \times 6.5 \text{ kips}}{\pi \times 0.053 \text{ kips/in} \times 30 \text{ in}} = 0.138 \]  \hspace{1cm} (B.10)

By interpolation from Table 3.5 in Section 3.4.3, the damping coefficient \( B_\rho \) is determined to be 1.314.

The updated displacement is calculated using Eq. 3.11 as:

\[ d_i = \frac{10S_A(T_{\text{eff}}) \cdot T_{\text{eff}}^2}{B_\rho} = \frac{10 \times 0.288 \times (3.54)^2}{1.314} = 27.05 \text{ in} \]  \hspace{1cm} (B.11)

where \( S_A(3.54) = 0.288g \) is the spectrum ordinate at the period of 3.54 sec obtained from Fig. B.2. The relative error in designed displacement is:

\[ \text{Relative Error} = \frac{|d_i - d_4|}{d_i} \times 100\% = \frac{|27.05 \text{ in} - 30 \text{ in}|}{27.05 \text{ in}} \times 100\% = 10.9\% \]  \hspace{1cm} (B.12)

One more iteration is performed so that the relative error for the displacement estimation is less than 2% and the design displacement is 26.83 in. After one iteration the design displacement of \( d = 26.8 \text{ in} \) results in an effective period of \( T_{\text{eff}} = 3.22 \text{ sec} \) and the design acceleration of \( S_A(3.22) = 0.305g \) for the isolation system with \( \mu = 0.07 \) and \( T_g = 4.0 \text{ sec} \). Table B.2 presents the summary of the one iteration.
Table B.2 Summary of design for FIS at level 3 in SPSW9M

<table>
<thead>
<tr>
<th>Iteration</th>
<th>Design displacement (in)</th>
<th>Effective stiffness $K_{eff}$ (kips/in)</th>
<th>Effective Period $T_{eff}$ (sec)</th>
<th>Damping ratio $\beta_{eff}$</th>
<th>Damping coefficient $B_D$</th>
<th>Updated displacement (in)</th>
<th>Relative error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Estimation</td>
<td>30in</td>
<td>0.053</td>
<td>3.54</td>
<td>0.138</td>
<td>1.314</td>
<td>27.05</td>
<td>10.9</td>
</tr>
<tr>
<td>1</td>
<td>27.05</td>
<td>0.0547</td>
<td>3.48</td>
<td>0.148</td>
<td>1.345</td>
<td>26.83</td>
<td>0.8</td>
</tr>
</tbody>
</table>

**B.4 Design Example of FIS at level 5 in SPSW9M**

Fig. B.3 presents the floor average response spectrum for level 5 in SPSW9M, the design acceleration limit is 0.3g and the design target effective period is 3.81 sec. A second-slope period $T_d = 4.0$ sec was assumed that for a friction pendulum bearing gives $K_d = 0.042$ kips/in. Additionally a coefficient of friction of 0.027 ($\mu$) was assumed.

![Fig. B.3 Floor average response spectrum for level 5 in SPSW9M](image)

To initiate the process, the characteristic strength is calculated as:

$$Q_d = \mu W = 0.027 \times 6.5 \text{ kips} = 0.1755 \text{ kips}$$  \hspace{1cm} (B.13)

A FIS displacement of 30 in is assumed and the effective stiffness of a FIS isolator is calculated as:
\[ K_{\text{eff}} = \frac{0.027 \times 6.5 \text{kips}}{30 \text{ in}} + 0.042 \text{ kips/in} = 0.0479 \text{ kips/in} \]  
(B.14)

From which \( T_{\text{eff}} \) is calculated as:

\[ T_{\text{eff}} = 2\pi \sqrt{\frac{W}{K_{\text{eff}} \cdot g}} = 2\pi \sqrt{\frac{6.5 \text{kips}}{0.0479 \text{ kips/in} \times 386.4 \text{ in/sec}^2}} = 3.73 \text{ sec} \]  
(B.15)

The effective damping ratio is calculated as:

\[ \beta_{\text{eff}} = \frac{2Q_d}{\pi K_{\text{eff}} \cdot d_0} = \frac{2 \times 0.027 \times 6.5 \text{kips}}{\pi \times 0.0479 \text{ kips/in} \times 30 \text{ in}} = 0.078 \]  
(B.16)

By interpolation from Table 3.5 in Section 3.4.3, the damping coefficient \( B_D \) is determined to be 1.112.

The updated displacement is calculated using Eq. 3.11 as:

\[ d_i = \frac{10 S_a(T_{\text{eff}}) T_{\text{eff}}^2}{B_D} = \frac{10 \times 0.31 \times (3.73)^2}{1.112} = 39.17 \text{ in} \]  
(B.17)

where \( S_a(3.73) = 0.31g \) is the spectrum ordinate at the period of 3.73 sec obtained from Fig. B.3. The relative error in calculated displacement is:

\[ \text{Relative Error} = \left| \frac{d_i - d_0}{d_i} \right| \times 100\% = \left| \frac{39.17 \text{ in} - 30 \text{ in}}{39.17 \text{ in}} \right| \times 100\% = 23.4\% \]  
(B.18)

Two more iteration are performed so that the relative error for the displacement estimation is less than 2% and the design displacement is 41.6 in. After two iterations the design displacement of \( d = 41.6 \text{ in} \) results in an effective period of \( T_{\text{eff}} = 3.80 \text{ sec} \) and the design acceleration of \( S_a(3.80) = 0.30g \) for the isolation system with \( \mu = 0.027 \) and \( T_d = 4.0 \text{ sec} \). Table B.3 presents the summary of the one iteration.
Table B.3 Summary of design for FIS at level 5 in SPSW9M

<table>
<thead>
<tr>
<th>Iteration</th>
<th>Design displacement (in)</th>
<th>Effective stiffness $K_{eff}$ (kips/in)</th>
<th>Effective Period $T_{eff}$ (sec)</th>
<th>Damping ratio $\beta_{eff}$</th>
<th>Damping coefficient $B_D$</th>
<th>Updated displacement (in)</th>
<th>Relative error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Estimation</td>
<td>30in</td>
<td>0.04785</td>
<td>3.73</td>
<td>0.078</td>
<td>1.112</td>
<td>39.2</td>
<td>23.4</td>
</tr>
<tr>
<td>1</td>
<td>39.2</td>
<td>0.0465</td>
<td>3.78</td>
<td>0.061</td>
<td>1.045</td>
<td>41.0</td>
<td>4.5</td>
</tr>
<tr>
<td>2</td>
<td>41.0</td>
<td>0.0463</td>
<td>3.80</td>
<td>0.059</td>
<td>1.035</td>
<td>41.63</td>
<td>1.5</td>
</tr>
</tbody>
</table>

B.5 Design Example of FIS at level 8 in SPSW9M

Fig. B.4 presents the average floor response spectrum for floor 8 in SPSW9M, the design acceleration limit is 0.3g and the design target effective period is 4.21 sec. A second-slope period $T_d = 4.42$ sec was assumed that for a friction pendulum bearing gives $K_d = 0.034$ kips/in. Additionally a coefficient of friction of 0.027 ($\mu$) was assumed.

To initiate the process, the characteristic strength is calculated as:

$$Q_w = \mu W = 0.027 \times 6.5 \text{ kips} = 0.1755 \text{ kips} \quad \text{(B.19)}$$
A FIS displacement of 50 in is assumed and the effective stiffness of a FIS isolator is calculated as:

\[ K_{\text{eff}} = \frac{0.027 \times 6.5 \text{ kips}}{50 \text{ in}} + 0.034 \text{ kips/in} = 0.0375 \text{ kips/in} \]  \hspace{1cm} (B.20)

From which \( T_{\text{eff}} \) is calculated as:

\[ T_{\text{eff}} = 2\pi \sqrt{\frac{W}{K_{\text{eff}} \cdot g}} = 2\pi \sqrt{\frac{6.5 \text{ kips}}{0.0375 \text{ kips/in} \times 386.4 \text{ in/sec}^2}} = 4.21 \text{ sec} \]  \hspace{1cm} (B.21)

The effective damping ratio is calculated as:

\[ \beta_{\text{eff}} = \frac{2Q_d}{\pi \cdot K_{\text{eff}} \cdot d_0} = \frac{2 \times 0.027 \times 6.5 \text{ kips}}{\pi \times 0.0375 \text{ kips/in} \times 50 \text{ in}} = 0.06 \]  \hspace{1cm} (B.22)

By interpolation from Table 3.5 in Section 3.4.3, the damping coefficient \( B_D \) is determined to be 1.04.

The updated displacement is calculated using Eq. 3.11 as:

\[ d_1 = \frac{10 S_d(T_{\text{eff}}) \cdot T_{\text{eff}}^2}{B_D} = \frac{10 \times 0.299 \times (4.21)^2}{1.04} \approx 51 \text{ in} \]  \hspace{1cm} (B.23)

where \( S_d(4.21) = 0.299g \) is the spectrum ordinate at the period of 4.21 sec obtained from Fig. B.4. The relative error in calculated displacement is:

\[ \text{Relative Error} = \frac{|d_1 - d|}{d_1} \times 100\% = \frac{51 \text{ in} - 50 \text{ in}}{51 \text{ in}} \times 100\% = 1.96\% \]  \hspace{1cm} (B.24)

No more iteration is performed because the relative error for the displacement estimation is less than 2% and the design displacement is 51 in. The effective period \( T_{\text{eff}} \) is 4.21 sec and the design acceleration of \( S_d(4.21) \) is 0.30g for the isolation system with \( \mu = 0.027 \) and \( T_d = 4.42 \text{ sec} \).
Appendix C

Sensitivity Study Results

C.1 General

This appendix presents detail results for sensitivity analysis for the FIS at level 1, 2 in SPSW3M and level 3, 5 and 8 in SPSW9M. For FIS at each floor, the variations in web plate is either yield strength $F_y$ or the thickness $t$. Each case is considered with both DBE and MCE hazard level.
Fig. C.1 FIS on level 1 in SPSW3M under DBE (strength)

Fig. C.2 FIS on level 1 in SPSW3M under MCE (strength)
Fig. C.3 FIS on level 2 in SPSW3M under DBE (strength)

Fig. C.4 FIS on level 2 in SPSW3M under MCE (strength)
Fig. C.5 FIS on level 1 in SPSW3M under DBE (Thickness)

Fig. C.6 FIS on level 1 in SPSW3M under MCE (Thickness)
Fig. C.7 FIS on level 2 in SPSW3M under DBE (thickness)

Fig. C.8 FIS on level 2 in SPSW3M under MCE (thickness)
Fig. C.9 FIS on level 3 in SPSW9M under DBE (strength)

Fig. C.10 FIS on level 3 in SPSW9M under MCE (strength)
Fig. C.11 FIS on level 5 in SPSW9M under DBE (strength)

Fig. C.12 FIS on level 5 in SPSW9M under MCE (strength)
Fig. C.13 FIS on level 8 in SPSW9M under DBE (strength)

Fig. C.14 FIS on level 8 in SPSW9M under MCE (strength)
Fig. C.15 FIS on level 3 in SPSW9M under DBE (thickness)

Fig. C.16 FIS on level 3 in SPSW9M under MCE (thickness)
Fig. C.17 FIS on level 5 in SPSW9M under DBE (thickness)

Fig. C.18 FIS on level 5 in SPSW9M under MCE (thickness)
Fig. C.19 FIS on level 8 in SPSW9M under DBE (thickness)

Fig. C.20 FIS on level 8 in SPSW9M under MCE (thickness)