SEISMIC PERFORMANCE ASSESSMENT OF SELF-CENTERING DUAL
SYSTEMS WITH DIFFERENT CONFIGURATIONS

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SYSTEMS WITH DIFFERENT CONFIGURATIONS

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ABSTRACT

Concentrically braced frame (CBF) systems have limited capacity for lateral displacement, causing structural damage that leads to high post-earthquake costs under moderate earthquakes. Self-centering concentrically braced frame (SC-CBF) systems have been developed to soften the lateral force-lateral drift response of the system without causing structural damage; however, the construction cost of SC-CBF systems is expected to be high due to the precision required in fabrication and erection. To take advantage of the ductility of SC-CBFs and the economy of conventional CBFs, a novel dual system has been proposed in which the lower stories of the lateral force resisting system are SC-CBFs and the upper stories are conventional CBFs. The SC-CBF is intended to act as a ductile sub-structure for the CBF, reducing the construction costs and post-earthquake costs.

This study presents the design basis of the dual system and an investigation of the influence of dual system configuration on structural performance. To achieve this goal and demonstrate the effectiveness of the proposed dual system, a probabilistic performance assessment is conducted in this study, considering both structural and non-structural components. Firstly, eight prototype buildings are designed with the same configurations and gravity systems but different lateral load resisting systems (CBF and

dual systems). Then, nonlinear time history analysis is performed using 3D numerical modeling to obtain seismic responses under a suite of ground motions. Next, probabilistic demand models are developed to predict the selected engineering demand parameters. To develop probabilistic demand models, several intensity measures are investigated to determine the best link between ground motion properties and structural and non-structural responses. The demand model parameters are estimated from regression analysis based on the responses obtained from nonlinear time history analysis of the finite element models. Moreover, hazard analysis is performed to quantify the seismic hazard for the structures. Three different locations inside the United States are considered in the hazard analysis: Los Angeles (high-seismic zone), Seattle (moderate-seismic zone), and Boston (low-seismic zone). Using detailed demand models, the corresponding capacity limits, and calculated intensity hazards, seismic fragility curves and engineering demand hazard curves are developed for three performance levels for structural and non-structural components. Finally, loss analysis is conducted to estimate expected annual loss and pay-off periods for different structural systems. The results of this study show that the dual system can be an efficient structural system for mid-rise buildings in high-seismic zones. Moreover, the time in which the higher initial construction costs of the dual systems are compensated by lower earthquake-induced losses in the lifetime of structures is calculated and presented for different configurations of the dual system.

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TABLE OF CONTENTS

	Page
LIST OF TABLES	ix
LIST OF FIGURES	xii
CHAPTER	
I. INTRODUCTION.....	1
1.1 Overview	1
1.2 Research Objectives.....	3
1.3 Research Scope	4
1.4 Organization of Dissertation	6
II. BACKGROUND AND LITERATURE REVIEW.....	9
2.1 Review of Research Related to the SC-CBF System.....	9
2.2 Review of Research Related to Comparative Study of Different Structural Systems.....	12
2.3 Review of Research Related to Cost Studies	14
III. DUAL SYSTEM BEHAVIOR, DESIGN PROCESS AND CONFIGURATION.....	17
3.1 System behavior.....	17
3.2 Design Process	20
3.2.1 Dual system design.....	21

3.2.1.1 Initial Design Phase	22
3.2.1.2 Analytical Design Process	27
3.2.2 CBF Design	36
IV. NUMERICAL ANALYSIS.....	48
4.1 Prototype Structures.....	48
4.2. Numerical Modeling	49
4.2.1 CBF Model	50
4.2.2 Dual System Model	51
4.3 Ground Motion Selection.....	52
4.3.1 Maximum Considered Earthquake (MCE).....	52
4.3.2 Design Basis Earthquake (DBE)	53
4.3.3 Frequently Occurring Earthquake (FOE)	53
4.4 Ground Motion Suite	53
4.4 Numerical Modeling Results	54
V. PROBABILISTIC DEMAND MODEL.....	74
5.1 Formulation of Probabilistic Demand Model	75
5.2 Deterministic Demand Model.....	76
5.3 Correction Terms	79
5.4 Proposed Probabilistic Demand Models.....	81
VI. SEISMIC PERFORMANCE EVALUATION OF THE DUAL SYSTEM.....	94
6.1 Seismic Fragility Assessment	95
6.2 Seismic Hazard Analysis	100

6.3 Engineering Demand Hazard Analysis	102
6.4 Comparative Study of Dual Systems with Different Configurations	104
VII. PROBABILISTIC FRAMEWORK FOR STRUCTURAL AND NON- STRUCTURAL DAMAGE ASSESSMENT	143
7.1 Introduction.....	143
7.2 Calculation of the Initial Cost.....	145
7.3 Expected Annual Loss (EAL).....	146
7.4 Site Effect Investigation.....	151
7.5 Pay-off Period	153
VIII. SUMMARY AND CONCLUSIONS	172
8.1 Summary of Study	172
8.2 Conclusions.....	174
8.3 Future Work	178
REFERENCES	180
APPENDICES	185
APPENDIX A: PEAK DEMAND RESPONSES OF THE STRUCTURES	186
APPENDIX B: WEIGHT CALCULATION OF THE STRUCTURES	202

LIST OF TABLES

Table	Page
3.1 Regression coefficients a , b , c , and d for Equations 13 and 14 (Seo 2005)	38
3.2 Member types and size for the six-story dual systems and CBF system	39
3.3 Member types and size for ten-story dual systems and CBF system.....	40
4.1 – Dead loads at each floor level for prototype structures	56
4.2 – Live loads at each floor level for prototype structures	56
4.3 – Summary of tributary seismic masses for 6-story prototype structures.....	57
4.4 – Summary of tributary seismic masses for 10-story prototype structures.....	57
4.5 Summary of SAC report ground motion characteristics.....	58
5.1 Candidate explanatory functions of normalized intensity measures.....	85
5.2 Statistics of the model parameters in the interstory drift demand models for structural elements in six-story buildings	86
5.3 Statistics of the model parameters in the interstory drift demand models for structural elements in ten-story buildings	87
5.4 Statistics of the model parameters in the interstory drift demand models for non- structural elements in six-story buildings	88
5.5 Statistics of the model parameters in the interstory drift demand models for non- structural elements in ten-story buildings	89
5.6 Statistics of the model parameters in the floor acceleration demand models for non- structural elements in six-story buildings	90
5.7 Statistics of the model parameters in the floor acceleration demand models for non- structural elements in ten-story buildings	90

5.8 Statistics of the model parameters in the roof acceleration demand models for non-structural elements in six-story buildings	91
5.9 Statistics of the model parameters in the roof acceleration demand models for non-structural elements in ten-story buildings	91
6.1 Nonstructural component capacities based on ASCE41 (2007).	109
6.2 Interstory drift capacities (ASCE 2007)	109
6.3 Assumed statistics of uncertain structural properties.....	109
6.4 Parameters of IM Hazards for the 6-story frames for LA.....	110
6.5 Parameters of IM Hazards for the 6-story frames for SE	110
6.6 Parameters of IM Hazards for the 6-story frames for BO	110
6.7 Parameters of IM Hazards for the 10-story frames for LA.....	111
6.8 Parameters of IM Hazards for the 10-story frames for SE	111
6.9 Parameters of IM Hazards for the 10-story frames for BO	111
6.10 Correlation coefficient (ρ) between the selected normalized IMs in LA.....	112
6.11 Mean annual probability of failure (structural interstory drift) for 10-story building	113
6.12 Mean annual probability of failure (structural interstory drift) for 10-story building in BO.....	113
6.13 Mean annual probability of failure (structural interstory drift) for 10-story building in SE.....	113
6.14 Mean annual probability of failure (structural interstory drift) for 6-story building in LA	114
6.15 Mean annual probability of failure (structural interstory drift) for 6-story building in BO	114
6.16 Mean annual probability of failure (structural interstory drift) for 6-story building in SE.....	114
6.17 Mean annual probability of failure (non-structural roof acceleration) for 10-story building	115

6.18 Mean annual probability of failure (non-structural roof acceleration) for 6-story building	115
7.1 Square foot costs (RS Means 2013).....	155
7.2 Percentage of sub-total costs for building's components (RS Means 2013)	155
7.3. Damage state descriptions (Bai et al. 2013).....	156
7.4 Damage factors (Bai et al. 2013)	156
A.1 Peak Interstory drift ratio (%) for 6-story buildings	186
A.2 Peak Interstory drift ratio (%) for 10-story buildings	190
A.3 Peak floor acceleration (g) for 6-story buildings	194
A.4 Peak floor acceleration (g) for 10-story buildings	198

LIST OF FIGURES

Figure	Page
3.1 SC-CBF concept: (a) schematic of SC-CBF system; and (b) rocking behavior.....	41
3.2 Prototype structure of one proposed dual system elevation (D4-2).....	41
3.3 Dual system lateral force-displacement behavior	42
3.4 Typical floor plan of buildings: (a) 6-story building; (b) 10-story building.....	42
3.5 Free-body diagram of Dual system (D5-5) Frame at decompression.....	43
3.6 Force-displacement (F-u) behavior of SC systems with varying β_E (Seo and Sause 2005): (a) SC model with β_E ; (b) $\beta_E = 0\%$; (c) $\beta_E = 25\%$; (d) $\beta_E = 50\%$	44
3.7 – Schematic of idealized overturning moment versus roof drift response of a Dual system	45
3.8 – Design response spectrum (ASCE 2010).....	46
3.9 Dual system configurations: (a) D2-4; (b) D3-3; (c) D4-2; (d) D4-6; (e) D5-5 and (f) D7-3	47
4.1 Elevations of 10-story and 6-story CBFs and dual system D4-2.....	62
4.2 Response spectra for MCE-level ground motions	63
4.3 Response spectra for DBE-level ground motions.....	64
4.4 Response spectra for FOE-level ground motions	65
4.5 Sample results for (site) DBE-level Imperial Valley ground motion for D3-3: (a) roof displacement, (b) ground acceleration, and (c) ground velocity.....	66
4.6 Sample results for the (site) DBE-level Imperial Valley ground motion for 6-story CBF: (a) roof displacement, (b) ground acceleration, and (c) ground velocity	67

4.7 D3-3 floor displacement responses for the (site) DBE-level Imperial Valley ground motion	68
4.8 6-story CBF floor displacement responses for the (site) DBE-level Imperial Valley ground motion	69
4.9 D3-3 response to the (site) DBE-level Imperial Valley ground motion: (a) ground acceleration, (b) relative roof acceleration and (c) absolute roof acceleration	70
4.10 6-story CBF responses to the (site) DBE-level Imperial Valley ground motion: (a) ground acceleration, (b) relative roof acceleration and (c) absolute roof acceleration	71
4.11 Roof drift response for 6-story buildings to the LA DBE-level Imperial Valley ground motion	72
4.12 Floor acceleration response for 10-story buildings to the LA DBE-level Imperial Valley ground motion	73
5.1 Computed demand using FEM vs. deterministic demand for (a) CBF, (b) D4-2 system, (c) D3-3 system, and (d) D2-4 system	92
5.2 Computed demand using FEM vs. prediction from proposed probabilistic inter-story demand models for (a) CBF, (b) D4-2 system, (c) D3-3 system and (d) D2-4 system	93
6.1 Fragility surface of interstory drift in D5-5 system: (a) IO, (b) LS, and (C) CP	116
6.2 Fragility surface of interstory drift in D7-3 system: (a) IO, (b) LS, and (C) CP	117
6.3 Fragility surface of roof acceleration (HVAC) in D5-5: (a) IO and (b) LS	118
6.4 Fragility surface of roof acceleration (HVAC) in D7-3: (a) IO and (b) LS	119
6.5. Joint PDF of IMs (PSA/g and $PGV \cdot T_n/h_c$) for D5-5 in (a) LA, (b) BO, and (C) SE	120
6.6. Joint PDF of IMs (PSA/g and PGA/g) for D5-5 in LA	121
6.7 (a) Fragility surface of inter-story drift in D5-5 system for 0.5% inter-story drift capacity and (b) joint PDF of IMs (S_a/g and $PGV \cdot T_n/h_c$)	122
6.8 Fragility curves of inter-story drift in CBF (solid blue lines) and D2-4 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance	123
6.9 Fragility curves of inter-story drift in CBF (solid blue lines) and D3-3 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance	124

6.10 Fragility curves of inter-story drift in CBF (solid blue lines) and D4-2 (dashed red lines) for (a) IO (b) LS (c) CP levels of performance	125
6.11 Fragility curves of inter-story drift in CBF (solid blue lines) and D4-6 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance	126
6.12 Fragility curves of inter-story drift in CBF (solid blue lines) and D5-5 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance	127
6.13 Fragility curves of inter-story drift in CBF (solid blue lines) and D7-3 (dashed red lines) for (a) IO (b) LS (c) CP levels of performance	128
6.14 Fragility curves of inter-story drift in dual system with different configuration for (a) IO (b) LS (c) CP levels of performance	129
6.15 Fragility curves of inter-story drift in dual system with different configuration for (a) IO (b) LS (c) CP levels of performance	130
6.16 Fragility curves of drift-sensitive non-structural components in CBF (solid blue lines) and D3-3 (dashed red lines) for (a) Exterior wall - IO (b) Exterior wall - LS (c) Partitions – IO (d) Partitions – LS	131
6.17 Fragility curves of drift-sensitive non-structural components in CBF (solid blue lines) and D7-3 (dashed red lines) for (a) Exterior wall - IO (b) Exterior wall - LS (c) Partitions – IO (d) Partitions – LS	132
6.18 Fragility curves of acceleration-sensitive non-structural components in CBF (solid blue lines) and D3-3 (dashed red lines) for (a) HVAC - IO (b) HVAC - LS (c) Fire suppression – IO (d) Fire suppression - LS	133
6.19 Fragility curves of acceleration-sensitive non-structural components in CBF (solid blue lines) and D7-3 (dashed red lines) for (a) HVAC - IO (b) HVAC - LS (c) Fire suppression – IO (d) Fire suppression - LS	134
6.20 Mean annual probability of failure (structural interstory drift) for 10-story CBF and 10-story Dual systems with different configurations in LA	135
6.21 Mean annual probability of failure (structural interstory drift) for 10-story CBF and 10-story Dual system with different configurations in BO	136
6.22 Mean annual probability of failure (structural interstory drift) for 10-story CBF and 10-story Dual system with different configurations in SE	137
6.23 Mean annual probability of failure (structural interstory drift) for 6-story CBF and 6-story Dual system with different configurations in LA	138

6.24 Mean annual probability of failure (structural interstory drift) for 6-story CBF and 6-story Dual system with different configurations in BO	139
6.25 Mean annual probability of failure (structural interstory drift) for 6-story CBF and 6-story Dual system with different configurations in SE.....	140
6.26 Mean annual probability of failure (non-structural roof acceleration) for 10-story CBF and 10-story Dual system with different configurations	141
6.27 Mean annual probability of failure (non-structural roof acceleration) for 6-story CBF and 6-story Dual system with different configurations.....	142
7.1 Illustration of relationship between fragility curves and damage states (Bai et al. 2009)	157
7.2 Illustration of relationship between α and β (a) D4-2 system, (b) D3-3 system, (c) D5-5 system, and (d) D7-3 system.....	158
7.3 Structural Expected Annual Loss (\$) in 6-story buildings in LA.....	159
7.4 Total Expected Annual Loss (\$) in 6-story buildings in LA	160
7.5 Structural Expected Annual Loss (\$) in 10-story buildings in LA.....	161
7.6 Total Expected Annual Loss (\$) in 10-story buildings in LA	162
7.7 Computed demand using FEM vs. prediction from proposed probabilistic inter-story demand models for (a) D4-2, (b) D3-3, (c) D2-4, and (d) 6-story CBF system. 163	
7.8 Computed demand using FEM vs. prediction from proposed probabilistic inter-story demand models for (a) D7-3 system, (b) D5-5 system, (c) D4-6 system, and (d) 10-story CBF system.....	164
7.9 Structural Expected Annual Loss (\$) in 6-story buildings in BO.....	165
7.10 Structural Expected Annual Loss (\$) in 10-story buildings in BO.....	166
7.11 Structural Expected Annual Loss (\$) in 6-story buildings in SE	167
7.12 Structural Expected Annual Loss (\$) in 10-story buildings in SE	168

7.13 Pay-off period (year) in 6-story buildings considering total EAL	169
7.14 Pay-off period (year) in 6-story buildings considering structural EAL	170
7.15 Pay-off period (year) in 10-story buildings considering structural EAL	171

CHAPTER I

INTRODUCTION

1.1 Overview

Steel concentrically braced frame (CBF) systems are commonly used lateral force resisting systems in earthquake-resistant design, despite their limited capacity for lateral displacement prior to structural damage. Due to the low drift capacity of CBFs, building codes stipulate comparatively low values for the response modification factor (which accounts for the inherent ductility of a system) used in the design. Based on the current standard practice in seismic design, CBF brace members are expected to buckle or yield under a moderate earthquake, including the design-basis earthquake (DBE), causing structural and non-structural damage (Mahin & Uriz 2004).

Several different strategies can be employed to reduce permanent damage to non-ductile structures (e.g., CBF systems), including base isolation and various implementations of active and semi-active structural control. Alternatively, the drift capacity of non-ductile CBFs can be improved by implementing special CBFs (SCBFs) that have more stringent detailing requirements than ordinary CBFs (AISC 2010a). Eccentrically braced frames (EBFs) represent another approach to reduce damage to the

braces in braced frames and increase ductility of the system by restricting inelastic deformations to shear links (Malley and Popov 1984).

Buckling-restrained braced frames (BRBFs) can be used to further increase the drift capacity of the system (e.g., Fahnestock et al. 2007). BRBFs are designed to permit ductile yielding of the braces in both tension and compression. This yielding is still a form of damage, however, and may lead to significant residual drift. The significant residual drift due to their low post-yield stiffness is the most important concern in BRBFs (Kinali and Ellingwood 2007).

Self-centering CBFs (SC-CBFs) have been developed recently (Roke et al. 2006) to increase the drift capacity of the system without incurring structural damage. SC-CBFs soften the lateral force-lateral drift response without inelastic deformation of structural members by allowing gaps to open at the base of the SC-CBF columns. These joints are post-tensioned to provide a restoring force to return the structure to its undeformed state (i.e., self-centering the system). Unlike conventional frames, SC-CBFs dissipate energy not due to damage of structural members but through supplemental energy dissipating devices (Roke et al. 2010). SC-CBF systems are intended to sustain minimal damage under the DBE; however, they are expected to cost more than conventional CBFs due to the precision required in fabrication and erection.

In this research a novel dual structural system is proposed by combining conventional CBFs and SC-CBFs to take advantage of the ductility of the SC-CBF system and the economy of the conventional CBF system. The purpose of the research

discussed in this dissertation is to establish the dual system as a viable and economical lateral force resisting system for earthquake-resistant buildings. Analytical studies and cost analyses are included to establish the dual system's performance and to develop dual system design criteria.

1.2 Research Objectives

The goals of this study are to develop the dual system and evaluate the seismic performances of different configurations of the proposed dual system and compare them with the seismic performance of a conventional CBF system. Moreover, a cost analysis is conducted to demonstrate the economic efficiency of the dual system in comparison with a conventional CBF system. This investigation will approach an optimal combination of SC-CBFs and CBFs for the dual system. The specific objectives necessary to achieve the overall objectives are the following:

1. To establish a design procedure for the proposed dual system;
2. To design different configurations of the dual system for the 6-story and 10-story prototype buildings;
3. To develop a nonlinear analytical model to simulate the seismic responses of the dual systems and CBF systems;
4. To develop a probabilistic demand model for the selected engineering demand parameters (EDPs) to approximate the responses of the buildings due to the seismic excitation.

5. To conduct seismic fragility and hazard assessments to compare the performance of the dual systems and CBF systems under different earthquake intensities.

6. To execute cost analysis to validate the economic efficiency of the dual system with respect to the CBF system.

1.3 Research Scope

To achieve the research objectives, six research tasks were undertaken. These tasks are as follows:

1. Use the performance-based design approach to design six dual systems with different configurations. Preliminary performance-based seismic design (PBD) criteria were used to design the six dual systems in this study. The design procedure is adapted and modified from the one for rocking SC-CBFs with friction-based energy dissipation (Roke et al. 2010).
2. Develop an analytical model for dual systems and CBF systems. Detailed nonlinear analytical models are created in OPENSEES (Mazzoni et al. 2009) to represent the CBF systems and the dual systems. The main components of the models are the SC-CBFs and the adjacent gravity columns in the dual systems, conventional beams, columns and braces in the upper stories of the dual systems and in the CBF systems, and the lean-on columns.
3. Perform nonlinear dynamic analyses using the analytical models of the designed dual systems and CBF systems. Nonlinear dynamic analyses were

performed to assess the responses of the six dual system configurations and the two CBF systems under several ground motions. The analysis results were used to investigate seismic response of different dual system and CBF systems.

4. Develop probabilistic seismic demand model for the dual systems and CBFs.

Accurate probabilistic seismic demand models are crucial in order to perform seismic performance evaluation of the structures using seismic fragility and EDP hazard curves. In this study, the selected EDPs are inter-story drifts and floor accelerations. The proposed demand models will consider all the relevant uncertainties due to seismic excitations associated with the EDPs in the CBF and dual systems. Such uncertainties include uncertainties in the ground motions, structural properties, and model errors, as well as statistical uncertainties in the model parameters.

5. Perform fragility analysis and seismic hazard analysis. Seismic fragility curves are used to assess and compare the seismic performance of dual systems and CBF systems. However, the fragility analyses have their own shortcomings; most importantly, they cannot capture the probability of exceedance of a specific intensity measure (IM) at a specific site. Moreover, direct comparison of structural fragility may be impossible if those fragilities are conditioned on different IMs. Due to these limitations of fragility analysis, seismic hazard curves are presented. Seismic hazard of an engineering demand parameter is defined as the mean annual rate of exceedance of a specified demand quantity.

Therefore, in addition to fragility analysis, seismic hazard curves are developed in this study to compare seismic performance of the CBF systems and the dual systems.

6. Perform expected annual loss analysis of the dual systems and CBF systems.

To fully conduct comparative analysis, expected annual losses of the different structural systems are calculated and compared. The pay-off period (the time over which the higher initial investment is recouped) is also calculated for the dual systems with respect to the CBF systems based on the structural and non-structural damage costs.

1.4 Organization of Dissertation

The remaining chapters of this dissertation are organized as follows:

- Chapter 2 discusses previous research related to the different SC-CBF systems. This chapter also discusses previous research related to comparative studies of buildings with different structural systems. Moreover, previous research on structural cost analysis is discussed in this chapter.
- Chapter 3 introduces dual system behavior, configurations, and the proposed preliminary design procedure. A total of six different dual system configurations for 6-story and 10-story buildings are presented in this chapter.
- Chapter 4 presents a numerical analysis of the CBF and dual systems. Prototype buildings and numerical modeling assumptions are introduced in this chapter. Moreover, the MCE-level, DBE-level, and FOE-level ground motions used in the

analyses are introduced in this chapter. This chapter also discusses the nonlinear numerical dynamic analysis results for the different configurations of the dual systems and CBFs, and the response to one ground motion is presented graphically to illustrate typical dynamic responses of the studied systems.

- Chapter 5 discusses the estimation of seismic demands for the dual systems and CBFs. These design demands are determined probabilistically. The proposed formulation of the probabilistic demand models consists of correction terms added to a deterministic demand prediction used in practice.
- Chapter 6 discusses seismic performance evaluation of the studied structural systems. Based on the developed probabilistic demand models, fragility curves and hazard curves for three performance levels (immediate occupancy, life safety, and collapse prevention) are developed. Seismic fragility curves and hazard curves are then used to assess and compare the seismic performance of dual systems and CBF systems.
- Chapter 7 presents expected annual loss analysis of the dual systems and CBF systems. Expected annual loss (EAL) of the buildings includes the initial construction cost, as well as the cost of both structural and non-structural damage due to earthquakes. EAL assessment will demonstrate the benefits of the dual system through time, as it is expected to exhibit better seismic performance that mitigates the earthquakes induced losses offsetting its higher initial construction cost.

- Chapter 8 summarizes the research program and offers conclusions and recommendations for future research.

CHAPTER II

BACKGROUND AND LITERATURE REVIEW

Self-centering (SC) lateral force resisting systems are a relatively recent development in earthquake-resistant structural systems. SC systems are intended to sustain less structural damage and residual drift than conventional structural systems under severe earthquakes, while maintaining the strength and stiffness of conventional structural systems.

Since self-centering systems are relatively new, there are few studies on their behavior and possible ways to improve their performance. In this chapter, results of recent research related to the development of self-centering concentrically-braced frames (SC-CBFs) are considered. The previous studies are organized into three sections: (1) studies related to the development and improvement of the SC systems, (2) studies related to the methods of comparing different structural systems, and (3) cost studies.

2.1 Review of Research Related to the SC-CBF System

The key aspect in the mechanics and behavior of the self-centering systems is the restoring force component that often utilizes a gap opening mechanism. The gap could occur at the base of the structure, at the connections, or in the braces themselves. Several

different SC systems can be characterized according to the location of the gap and the restoring mechanism.

Christopoulos et al. (2008) developed a self-centering energy dissipative (SCED) steel brace element that is comprised of traditional steel bracing elements, a dissipative mechanism, and a tensioning system that is used to pre-stress the device. Full-scale quasi-static and dynamic tests were conducted on a prototype SCED member. The tests confirmed that the SCED brace performed as predicted and it can achieve a stable and repeatable self-centering hysteretic response under cyclic loading.

Another self-centering system which can achieve its desired performance by rocking behavior at the base of the structure has been studied by Eatherton et al. (2010). Quasi-static tests were performed to apply cyclic loading to the structure, which employs shear fuses at the base to dissipate energy and vertically oriented post-tensioning to increase the restoring capacity of the system. The experimental results show that the controlled rocking CBF system can concentrate structural damage into replaceable elements and eliminate residual drift after unloading.

Although the base rocking response is intended to limit the force demands in the SC-CBF system, higher modes effects can amplify member force responses (Roke et al. 2008) and reduce the viability of the system. Wiebe et al. (2013) suggested that seismic forces may be reduced by providing multiple force-limiting mechanisms. They proposed two techniques: (1) detailing one or more rocking joints above the base rocking joint, and (2) providing a self-centering energy dissipative brace at one or more levels. 0.3-scale

subassembly tests were performed on an eight-story prototype rocking frame using a shaking table. The study showed that the proposed mechanisms can lead to higher capacity of the system by reducing the variability of peak seismic force demands without causing excessive displacement.

Another study by Tremblay et al. (2008) introduced an innovative braced steel frame system. The system designed to rock at its base under strong ground motions, incorporating viscous dampers that are vertically mounted between the foundation and the column bases to dissipate energy and control the structural lateral displacements while limiting the impact forces induced in the columns.

Another strategy for reducing damage and improving the seismic behavior of non-ductile structural systems is combining two or more different structural systems. A novel dual system is proposed in this study that combines conventional CBFs with SC-CBFs to take advantage of the ductility of the SC-CBF system and the economy of the conventional CBF system. In this proposed system, the lower stories of the dual lateral force resisting system will be SC-CBFs and the upper stories will be conventional CBFs. As the lower stories have much larger ductility capacity than the upper stories, ductility demands will concentrate in the lower stories and the SC-CBF will act as a base-isolation system for the upper stories. Therefore, the construction cost will be reduced (relative to a full SC-CBF system) and the seismic performance will be improved (relative to a conventional CBF system).

2.2 Review of Research Related to Comparative Study of Different Structural Systems

To show the effectiveness of the dual system in comparison with the conventional CBF, it is crucial to conduct a comparative study of the seismic performance of the two systems. Several methods have been used by researchers to compare seismic performance of different structural systems.

Yang and Murphy (2012) compared prototype five-story office buildings designed according to the national building code of Canada using three steel seismic force resisting systems (moment resisting frame (MRF), CBF, and BRBF). They considered two engineering demand parameters (EDPs) in their study, interstory drift ratio and floor acceleration, and studied seismic performance for both structural and non-structural elements. They calculated the median peak interstory drift ratio and the median peak floor acceleration and then compared the mean values together to find the structure with better performance. Their result showed that MRF has the highest median interstory drift, followed by BRBF, then the CBF for all hazard levels considered.

Kinali and Ellinwood (2007) considered three steel frames (the first two are moment frames with fully restrained (FR) and partially restrained (PR) connections, respectively, and the third is a braced frame) of different heights, designed following different building codes. Interstory drift was selected as the EDP for their study. Seismic fragilities for these frames were determined from assessment of their responses to groups of synthetic ground motions generated for the central and eastern US, probabilities of four distinct performance states were determined, and implications for seismic risk assessment were

discussed. According to their findings, in the event of a selected earthquake ($M=7.5$, $R=20\text{km}$), the moment frames with FR and PR connections are more likely to suffer collapse than the braced frame.

Another comparative study was conducted by Badpay and Arbabi (2008). They compared performance of CBFs and BRBFs using seismic fragility curves, considering interstory drift as the EDP. In their study, the comparison of seismic fragilities for different performance levels of models suggests that CBFs are more vulnerable than BRBFs and are more likely to reach or exceed a performance level for a given ground motion intensity measure.

Lin et al. (2010) evaluated the seismic reliability of EBF and BRBF systems with respect to that of MRF systems. In this study, a series of three-dimensional nonlinear time-history analyses were carried out for six example buildings. Inter-story drift hazard curves were used to compare the seismic performance of 6-story and 20-story buildings with different structural systems. The comparison shows that the effectiveness of braced frames can vary significantly with ground motions and building heights. The added braced frames help to reduce story drifts, decreasing the probability of failure under ordinary ground motions. They also found that near-source ground motions tend to input energy into structural systems in a very short time. This pulse-like feature makes it difficult to concentrate damage at the weak story only, and decreases the drift demands on MRF systems. This feature also makes it difficult to concentrate damage to EBF links or BRB members only, and increases the drift demands on EBF and BRBF systems. As a

result, adding braced frames can help to resist near source ground motions, but only to a limited extent.

In this research study, both structural and non-structural performance of the buildings is considered. The selected EDPs are interstory drift and absolute floor accelerations. Floor accelerations may be suitable for expressing damage to some types of non-structural building contents. Interstory drift, on the other hand, is closely correlated to damage to non-structural partitions or exterior walls, also provides insight regarding the potential for local or global collapse of the structure due to excessive P- Δ effects. In this study, seismic fragility and seismic hazard curves for the selected EDPs are used to evaluate the seismic performance of the CBF and the dual system.

2.3 Review of Research Related to Cost Studies

To fully conduct comparative analysis, loss assessments of the different structural systems are crucial. In this study, the main goal of the cost analysis is to investigate if the higher construction cost of the dual system would be compensated by lower earthquake-induced losses during the effective life time of the buildings. Life cycle cost (LCC), or its annual equivalence, which is called expected annual loss (EAL), assessment has been used as a measure of the economic effectiveness of a structure. Wen & Ang (1991) and Wen & Shinozuka (1994, 1998) developed an LCC formulation to investigate the cost effectiveness of an active control system in structures during earthquakes. Kang & Wen (2000) used a minimum LCC concept to develop an optimal design for structures under single and multiple hazards. Padgett et al. (2009) developed a retrofit strategy for bridges

based on cost-benefit analysis using LCC to determine the most cost-effective retrofit method that differs based on the seismic hazard characteristics of the location. Goda et al. (2010) used LCC assessment to investigate the cost-effectiveness of a seismic isolation technology. Cha & Elingwood (2013) used the LCC of buildings considering wind hazards to develop a methodology to quantify various attitudes towards hazard risk (risk taking, risk neutral, and risk adverse).

Building specific loss estimation is concerned with monetary loss (direct loss), downtime loss, and casualties. Although the proposed method is general and can incorporate all three categories of loss, this study considered only direct loss because of the very limited information available on downtime loss and casualties. More specifically, this study focuses on the capital-related loss (i.e., repair/replacement cost of building and components due to damage induced by ground motion), and not the income related loss (e.g. rental income loss, relocation expenses, etc.). This is consistent with previous research conducted by Zareian and Krwinkler (2009). Construction cost estimation is straightforward and can be generally estimated using expert opinions or tools such as R.S. Means Building Construction Cost Data (RS Means 2013), which is used in this study.

The LCC estimation is the last step in the performance-based assessment methodology presented by Moehle and Deierlein (2004). This methodology (which is adopted in this research as well) is decomposed into four analysis steps: (1) seismic hazard analysis, which quantifies the annual rate of exceeding a given value of seismic intensity measure (IM); (2) response analysis, in which the outcomes of response analysis

are expressed as statistical functions that relate engineering demand parameters (such as interstory drift or acceleration) to the hazard experienced by the structure; (3) damage analysis, which characterizes the structural and non-structural damage in terms of fragility curves; and (4) loss analysis, which consists of determining costs/losses due to the structural and non-structural damage. Each step involves both inherent (aleatory) randomness and model (epistemic) uncertainty that should be accounted for appropriately.

CHAPTER III

DUAL SYSTEM BEHAVIOR, DESIGN PROCESS AND CONFIGURATION

The seismic behavior of the proposed dual system is similar to that of the self-centering concentrically braced frame (SC-CBF) due to the structural details at the base of the structure. Therefore, in this chapter, the seismic behavior of the SC-CBF is illustrated briefly and different corresponding performance levels are introduced. Then, the design basis of the proposed dual system is illustrated and different configurations of the dual system are presented based on the proposed design process.

3.1 System behavior

The SC-CBF system has been developed to maintain the stiffness of a conventional CBF system while increasing its lateral drift capacity before structural damage initiates (Roke et al. 2006). As shown in Fig. 3.1(a), an SC-CBF is similar to a conventional CBF, with beams, columns, and braces in a conventional arrangement. However, the details at the SC-CBF column bases permit decompression and uplift of the columns under higher levels of lateral force, enabling rocking of the SC-CBF. The SC-CBF includes two sets of columns: SC-CBF columns, which are not directly connected to the floor diaphragms and are permitted to uplift from the foundation as shown in Fig. 3.1(b); and adjacent gravity columns, which are directly connected to the floor diaphragms and do not uplift. The

adjacent gravity columns support the gravity loads for the tributary area for the SC-CBF. Lateral-load bearings, located between these two sets of columns at each floor level, are designed to transmit lateral inertia forces from the adjacent gravity columns to the SC-CBF, while allowing relative vertical displacement between the gravity columns and SC-CBF columns. As a compressive load is applied between the two surfaces of the bearing, a friction force develops. The coefficient of friction for SC-CBF systems is about 0.45 (Roke et al. 2010) for steel on brass. Forces in the post-tensioning (PT) bars, gravity loads on the SC-CBF (self-weight), and friction at the lateral load bearings resist column uplift. After uplift, PT forces and self-weight provide restoring forces to eliminate residual drift (i.e., self-centering the system). During rocking, the response of the system is softened, reducing the force demands in the structural members as the deformation demands are concentrated in the rocking response. Thus, the resulting reduction the incremental member deformation demand increases the lateral drift capacity of the SC-CBF prior to structural damage.

To take advantage of the ductile behavior of the SC-CBF while reducing the additional construction costs of the system, a dual system is proposed in this study as a novel lateral load resisting system. The dual system includes both SC-CBFs and CBFs in elevation (e.g., the six story dual system shown in Fig. 3.2). The lower stories have SC-CBFs as the lateral load resisting system to take advantage of column uplift (i.e., the softening behavior of SC-CBFs). The PT bar in the dual system connects the top of the SC-CBF to the foundation of the structure and provide a positive stiffness to the post-decompression lateral force-lateral drift behavior of the lower stories of the dual system.

Lateral-load bearings and details for connections in the lower part of the dual system are the same as those used in SC-CBFs. The upper part of the structure has CBFs as the lateral load resisting system. As mentioned earlier, since the lower stories of the dual systems are SC-CBFs, the lateral force-lateral drift behavior of the dual system are quite similar to that of the SC-CBF due to rocking at the base. This is mainly due to the fact that decompression and the subsequent uplift of the base of the SC columns is the most significant and dominant feature of the SC-CBF and dual system behavior. Moreover, in both the SC-CBF and dual systems, PT bar elongation after column decompression will create a positive stiffness in the lateral force-lateral drift behavior. It should be noted that, although both SC-CBFs and CBFs are shown between the same columns in elevation, they would be in separate bays in practice to prevent any conflict between beams and braces at the interface level (shown as the fourth floor in Fig. 3.2) due to SC-CBF rocking behavior.

Specific limit states are considered in the SC-CBF or dual system design process. These limit states are related to three major seismic performance-based-design approach. The three performance levels are defined as follows:

Immediate Occupancy (IO): A structure achieving IO performance essentially retains its pre-earthquake strength and stiffness, and the structure is suitable for use immediately following inspection (BSSC 2003). Since decompression and uplift of the SC-CBF columns at the base of the structure do not cause structural damage if the column base is detailed properly, the proposed dual system can achieve IO performance after column

decompression. Minor PT bar yielding can still occur within the limits of IO performance.

Life Safety (LS): A structure achieving LS performance has sustained damage to structural and nonstructural components but retains a margin against partial or total collapse (BSSC 2003).

Collapse Prevention (CP): A structure achieving CP performance has sustained damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse (BSSC 2003). In this study, member failure is assumed to represent the limit of CP performance.

3.2 Design Process

For the proposed design of the dual system, the three major seismic performance levels used in the performance-based-design approach developed for the SC-CBF system (Roke et al. 2010) are adopted. These three performance levels (IO, LS, and CP) are related to four primary limit states of the dual system: (1) column decompression at the base of the structure; (2) PT bar yielding; (3) significant frame member yielding, which primarily occurs in the CBF part of the dual system; and (4) frame member failure.

Figure 3.3 schematically illustrates the relationship between the limit states and the performance levels.

3.2.1 Dual system design

The design of the proposed dual system has been adapted from the design procedure developed for SC-CBF systems (Roke et al. 2010), which is modified from the equivalent lateral force (ELF) and response spectrum analysis (RSA) procedures (ASCE 2010), to meet the three previously discussed performance levels under different earthquake hazard levels (design objectives). The earthquake hazard levels will be introduced in the next chapter.

The dual system design process has two stages: the initial design phase and the analytical design phase. The initial design phase determines the number of the braced frames and their location in the building. The gravity load resisting system in the building is designed in this stage. Values of the initial design parameters (an approximation of base shear, initial sizes of structural members, the area of the PT bars and the initial force in the PT bars) are also evaluated and selected in this preliminary design phase.

The second stage of the design is the analytical design process of the dual system which has two interrelated phases: the PT steel design phase and the structural member design phase. The PT steel design phase controls the occurrence of PT bar yielding by designing the PT bars to accommodate the lateral deformation at the top of the SC-CBF part of the dual system. The structural member design phase controls the occurrence of member failure (yielding or buckling). The required size of the structural members is a function of the inertia forces that develop for each mode and the PT bar yield force. In the following section each design phase of the dual system is elaborated in further detail.

3.2.1.1 Initial Design Phase

The initial design phase determines the number of braced frames and their location in the plan of the building. The number of braced frames is calculated based on the magnitude of the base shear and architectural consideration. The arrangement of braced frames should avoid having a column included in more than one SC frame, as simultaneous rocking deformations in orthogonal directions may complicate the member response. In this study four braced frames were used in each direction in the 6-story building and six braced frames were used in each direction in the 10-story building. Moreover, the initial design phase consists of two following sub-steps: equivalent lateral force procedure and determination of initial PT bar force and PT bar area, which are discussed here in detail:

Equivalent Lateral Force (ELF) Procedure

An ELF procedure is used to determine the lateral load forces to calculate the initial area of the PT bars. In this procedure the initial value of the base shear ($V_{b,ELF}$) and its distribution over the height of the building (F_{ELF}) are calculated based on ASCE-7 (ASCE 2010). The first step in the ELF procedure is to approximate the fundamental period of the prototype building. The approximate period is given as:

$$T_a = C_t \cdot h_n^x \quad (3.1)$$

where,

$$C_t = 0.02$$

$$x = 0.75 \text{ (ASCE7-10 Table 12.8-2 (ASCE 2010))}$$

h_n = the total height of the structure

The seismic response coefficient is determined as follows (ASCE 2010):

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \quad (3.2)$$

where,

R = the response modification coefficient, assumed to be 8 for Dual systems

I = the importance factor, equal to 1.0 for Occupancy Category II (ASCE 2010)

Maximum and minimum values for the seismic response coefficient are (ASCE 2010):

$$C_{S,max} = \left\{ \begin{array}{ll} \frac{S_{D1}}{T_a \cdot \left(\frac{R}{I}\right)} & \text{if } T_a \leq T_L \\ \frac{S_{D1} \cdot T_L}{T_a^2 \cdot \left(\frac{R}{I}\right)} & \text{if } T_a > T_L \end{array} \right\} \quad (3.3)$$

$$C_{S,min} = \left\{ \begin{array}{ll} 0.01 & \\ \frac{0.5 \cdot S_1}{\left(\frac{R}{I}\right)} & \text{if } S_1 \geq 0.6g \end{array} \right\} \quad (3.4)$$

Using C_s , the design base shear V is (ASCE 2010):

$$V = C_s \cdot W \quad (3.5)$$

where W = the total dead weight of the building, equal to the dead load plus the partition load. The individual lateral forces at each floor level are then calculated as follows (ASCE 2010):

$$F_x = C_{vx} \cdot V \quad (3.6)$$

$$C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^n w_i \cdot h_i^k} \quad (3.7)$$

where,

F_x = lateral seismic force at the floor x

C_{vx} = vertical distribution factor for floor x

w_i and w_x = weight assigned to floor level i or x

h_i and h_x = height from the base of the building to floor level i or x

$k = 1$ for buildings with a fundamental period of 0.5 s or less, $k = 2$ for buildings with a fundamental period of 2.5 s or more; k is linearly interpolated between 1 and 2 for buildings with a fundamental period between 0.5 s and 2.5 s

The lateral forces from Eq. 3.6 are distributed among the dual systems acting in each direction of the building. In the case of the prototype building, there are four dual systems

oriented in each direction in the 6-story building and six dual systems oriented in each direction in the 10-story building, as shown in Figure 3.4. These dual systems are assumed to be identical; therefore, each SC-CBF is assumed to carry one fourth of the lateral force in the 6-story building and one sixth of the lateral force in the 10-story building, and the equivalent lateral force vector $\{F_{ELF}\}$ for one six-story prototype dual system is:

$$\{F_{ELF}\} = \frac{1}{4} \{F_{ELF,1} \quad F_{ELF,2} \quad F_{ELF,3} \quad F_{ELF,4} \quad F_{ELF,5} \quad F_{ELF,6}\}^T \quad (3.8)$$

For design purposes, the ELF overturning moment is determined as follows:

$$OM_{ELF} = \{h\}^T \{F_{ELF}\} \quad (3.9)$$

where,

$\{h\}$ = vector of height from the base of the building to each floor level.

Other initial design parameters ($PT_{0,in}$ and $A_{PT,in}$)

The next step in the initial design phase is to approximate the initial pre-tensioning force in the PT bar and the area of the PT bar. When the initial overturning moment resistance of the dual system is overcome by the applied overturning moment, the tension column decompresses, which is referred to as gap-opening behavior. The overturning moment at column decompression is referred to as OM_D . For the initial design phase, the overturning moment demand at decompression, $OM_{D,d}$, is expressed in terms of OM_{ELF} as follows:

$$OM_{D,d} = \alpha_D \cdot OM_{ELF} \quad (3.10)$$

where α_D is arbitrarily set to 0.8. This means that column decompression will occur at an overturning moment equal to 80% of the code base overturning moment (OM_{ELF}). The decompression overturning moment capacity, OM_D , can be calculated from a free body diagram, including the overturning moment resistance from the weight of the SC-CBF part (W_{SC-CBF}), the initial PT bar force (PT_0), and forces from energy damping (ED) elements (V_{ED}) as shown in Figure 3.5. From this figure, OM_D is calculated as follows:

$$OM_D = (PT_0 + W_{SC-CBF}) \frac{b_{SC}}{2} + OM_{ED} \quad (3.11)$$

where,

b_{SC} = center-to-center spacing of the SC-CBF columns

OM_{ED} = the overturning moment provide by ED elements

PT_0 = initial force in PT bars

W_{SC-CBF} = total weight of SC-CBF part members

Using Eq. 3.11, PT_0 can be determined as follows:

$$PT_0 = OM_D \cdot \left(\frac{2}{b_{SC}} \right) - OM_{ED} \cdot \left(\frac{2}{b_{SC}} \right) - W_{SC-CBF} \quad (3.12)$$

OM_{ED} can be calculated based on a free body diagram as follows:

$$OM_{ED} = \eta \cdot OM_D \quad (3.13)$$

where

$$\eta = \tau \cdot \mu \cdot \frac{b_{ED}}{h_1^*} \quad (3.14)$$

$$\tau = \frac{\sum_{i=1}^n F_{ELF,i}}{V_{b,ELF}} \quad (3.15)$$

where

μ = coefficient of friction at the lateral-load bearings

n = number of SC-CBF stories in the dual system frame

$V_{b,ELF}$ = the design base shear (ASCE2010)

h_1^* = the effective height of the lateral forces (i.e., the ratio of OM to V_b)

After finding PT_0 , the initial area of the PT bars can be calculated as:

$$A_{PT} = \frac{PT_0}{\sigma_y} \quad (3.16)$$

Where σ_y is the yield stress of PT bars, which is equal to 120 ksi

3.2.1.2 Analytical Design Process

In the initial design process, PT bar design parameters (PT_0 , A_{PT}) were calculated approximately for the first iteration of design. In the following step, those parameters will be calculated analytically in an iterative design process. For the first iteration of design, the dual system members are selected arbitrarily. The first phase of the analytical design

process is the PT bar design phase, which is followed by the structural member design phase. These two phases will be alternated iteratively until the design has converged.

PT Steel Design Phase

The PT steel design phase selects the quantity of PT bars and the initial PT stress via an iterative process such that the roof drift capacity at PT bar yielding exceeds the factored DBE roof drift demand for the DBE. The roof drift design demand θ_{DBE} can be approximately calculated as (Roke et al. 2010):

$$\theta_{DBE} = \mu_{DBE} \cdot \theta_D \quad (3.17)$$

where θ_D is the roof drift at column decompression, determined from a linear elastic analysis of the fixed-based dual system as explained later. The ductility demand factor μ_{DBE} is calculated from μ - R - T a relationship that was developed for self-centering systems (Seo 2005). The μ - R - T relationship is used to approximate a median design value (Roke et al. 2010) of the ductility factor (μ_{DBE}) for self-centering systems.

$$\mu_{DBE} = R_D^{p(T_1)} \quad (3.18)$$

where

$$p(T_1) = \exp\left(\frac{c_1}{T_1^{c_2}}\right) \quad (3.19)$$

$$c_1 = \left(a - b\sqrt{\alpha_k}\right)^2 \quad (3.20)$$

$$c_2 = \left(c - d \sqrt{\alpha_k} \right)^2 \quad (3.21)$$

α_k is the ratio of the post-decompression stiffness of the dual system to the initial stiffness, which will be described later. The coefficients a, b, c, and d are functions of the hysteretic energy dissipation ratio for the structure, β_E , and the site soil conditions (site class). The hysteretic energy dissipation ratio, β_E , is defined as the ratio of the energy dissipated by the hysteresis loop of an self-centering system to the energy dissipated by a bilinear elasto-plastic system with the same strength under cyclic loading to the same peak displacement (Seo and Sause 2005). Schematics of force-displacement behaviors for SC systems with various values of β_E are shown in Figure 3.6. Values of coefficients a, b, c, and d are given in Table 3.1, based on the secant frequency damping model (Seo 2005).

R_D is defined as the strength of the system relative to the elastic response force demands, considering only the effective mass in the first mode (Roke et al. 2010), and is calculated as follows:

$$R_D = \frac{OM_{e,1}}{OM_D} \quad (3.22)$$

where $OM_{e,1}$ is the required strength of the dual system to remain linear elastic without column uplift under the DBE, which is determined from the ELF procedure with the R value assume to be 1 (or $OM_{e,1} = R \cdot OM_{ELF}$) and OM_D is the decompression overturning moment capacity (determined from the free body diagram of the dual system

at column decompression). It should be noted that this $\mu - R - T$ relationship was developed specifically for self-centering systems, and is therefore expected to be a conservative approximation for calculating the roof drift demand of the dual system. In Eq. 3.17, θ_D is the decompression roof drift and can be determined as follows:

$$\theta_D = \frac{\Delta_D}{h} \quad (3.23)$$

where, h is the total height of the structure and Δ_D is the roof displacement at decompression.

The roof displacement at decompression, Δ_D , is calculated from static analysis with the following forces. The applied forces in the decompression analysis of dual system are: lateral forces, $\{F_D\} = \alpha_{DI} \cdot \{F_I\}$; friction forces, $\{F_{ED}\} = \mu \cdot \{F_D\}$; $PT\phi$; and W_{SC-CBF} ; where,

$$\alpha_{DI} = \frac{OM_D}{OM_I} \quad (3.24)$$

$$OM_I = \{h\}^T \cdot \{F_I\} \quad (3.25)$$

$$\{F_I\} = \{S_I\} \cdot g = \Gamma_I[m]\{\phi_I\}g \quad (3.26)$$

$$\Gamma_I = \frac{\{\phi_I\}^T[m]\{i\}}{M_I} \quad (3.27)$$

$$M_I = \{\phi_I\}^T[m]\{\phi_I\} \quad (3.28)$$

$[m]$ = matrix of seismic masses tributary to the SC frame

$\{i\}$ = influence vector = $\{1 \ 1 \ 1 \ 1 \ 1 \ 1\}^T$ for a six story frame and
 $\{1 \ 1 \ 1 \ 1 \ 1 \ 1 \ 1 \ 1 \ 1 \ 1\}^T$ for a ten story frame

After calculating the roof drift demand, the roof drift capacity must be determined. Similar to Eq. 3.17, the roof drift capacity at PT bar yielding (θ_{YN}) for SC-CBF can be determined as follows (Roke et al. 2010):

$$\theta_{YN} = \mu_Y \cdot \theta_D \quad (3.29)$$

where μ_Y is the ductility capacity at PT yield and can be expressed as:

$$\mu_Y = \frac{\alpha_y + \alpha_k - 1}{\alpha_k} \quad (3.30)$$

where α_y is the ratio of OM_Y (the overturning moment capacity at PT yield) to OM_D , and α_k can be written as:

$$\alpha_k = \frac{k_{pd}}{k_{elastic}} \quad (3.31)$$

where $k_{elastic}$ is the elastic stiffness of the dual system and k_{pd} is the post-decompression stiffness (after column uplift and before PT bar yielding). It can be calculated from the pushover curve of the dual system, shown in Figure 3.7, and a free-body diagram of the system.

Since the dual system comprises both SC-CBFs and CBFs, the procedure used to approximate SC-CBF drift capacity has been modified to be applicable for the dual system. The modified roof drift capacity at PT bar yielding (θ_{YNM}) for the dual system includes drift capacities for both the SC-CBF and the CBF and can be written as:

$$\theta_{YNM} = \frac{h_s \cdot \mu_y \cdot \theta_{DS} + \delta}{h} \quad (3.32)$$

where h is the total height of the dual system; h_s is the height of SC-CBF part; θ_{DS} is the drift at the top of the SC-CBF part at decompression, determined from a linear elastic analysis of the fixed-based dual system; and δ is the maximum drift capacity of the CBF part, calculated based on 1.5% transient drift for life-safety performance (ASCE 2007). In Eq. 3.32, $h_s \cdot \mu_y \cdot \theta_{DS}$ accounts for the SC-CBF lateral displacement capacity and δ accounts for the CBF lateral displacement capacity.

In the PT steel design phase, the PT bar area, A_{PT} , must be selected such that the modified roof drift capacity (θ_{YNM}) exceeds the design roof drift demand (θ_{DBE}):

$$\theta_{YNM} \geq \theta_{DBE} \quad (3.33)$$

Structural Member Design Phase

The structural member design phase determines the factored member force design demands (axial force and bending moment) and selects the member sizes based on AISC-LRFD criteria (AISC 2010a). To create a margin between the limit state of member yielding and member failure, the seismic provisions (AISC 2010b) for special

concentrically-braced frames (SCBFs) are considered when selecting the structural members (Roke et al. 2010).

As shown in Figure 3.3, PT bar yielding should occur before member yielding. PT bar yielding will limit the design demand forces on the columns, beams and braces, and is considered an acceptable form of structural damage. Therefore, the member force design demands are calculated assuming that PT bars have yielded, and are determined using a modified RSA procedure (Roke et al. 2010), in which the first mode lateral forces are scaled to equilibrate the PT bar yield force. The scaling factor ($\alpha_{y,1}$) is calculated as follows:

$$\alpha_{y,1} = \frac{OM_y}{OM_1} \quad (3.34)$$

OM_y = the overturning moment capacity at the PT yield

OM_1 = the overturning moment based on the first mode lateral force profile

The applied lateral forces in the first mode are then calculated as:

$$\{F_{y,1}\} = \alpha_{y,1} \cdot \{F_1\} \quad (3.35)$$

where $\{F_1\}$ is calculated in Eq. 3.26.

To estimate the first mode member force demands, the following forces are applied to the SC frame: the first mode lateral forces, $\{F_{y,1}\}$; the weight of the SC frame, W_{SC-CBF} ; the PT force at yield, PT_y ; and the ED forces, V_{ED} ; due to their contribution to the

overturning moment resistance of the system. The corresponding first mode member force demands are the moments and axial loads from this linear analysis.

Nonlinear dynamic analyses under earthquake loading (Sause et al. 2006) have shown that the maximum member force demands during dynamic response significantly exceed the first mode design demands. More accurate member force design demands must include higher mode response in addition to the first mode response (Roke et al. 2008). The higher mode lateral forces are determined from a linear response spectrum analysis and applied to the frame where the n th mode spectral acceleration, $S_{a,n}$, is taken from the DBE design spectrum without a force reduction factor.

The design response spectrum for the DBE level is defined based on ASCE-7 (2010) as:

$$SA(T) = \left\{ \begin{array}{ll} S_{DS} \left(0.4 + 0.6 \cdot \frac{T}{T_0} \right) & T \leq T_0 \\ S_{DS} & T_0 < T \leq T_S \\ \frac{S_{DI}}{T} & T_S < T \leq T_L \\ \frac{S_{DI} \cdot T_L}{T^2} & T_L < T \end{array} \right\} \quad (3.36)$$

where,

S_{DS} , S_{DI} = spectral response acceleration parameters for short periods and a period of 1 sec., respectively (ASCE 2010)

T_0 , T_S , and T_L = transition periods (ASCE 2010)

The DBE-level design response spectrum given by Equation 3.36 is shown in Fig. 3.8.

The lowest modes should be included in the analysis such that the cumulative effective modal mass in those modes is greater than or equal to 95% of the total mass of the structure (Roke et al. 2010). The modal member force demands are then combined using the complete quadratic combination (CQC) method to calculate the member force design demands.

Each structural member is then selected such that its capacity exceeds design demand forces by meeting the AISC-LRFD criteria for axial force-bending moment interaction (AISC 2010b).

$$\frac{P_r}{\phi_c P_n} + \frac{8}{9} \left(\frac{M_{rx}}{\phi_b M_{nx}} + \frac{M_{ry}}{\phi_b M_{ny}} \right) \leq 1.0 \text{ for } \frac{P_r}{\phi_c P_n} \geq 2.0 \quad (3.36)$$

$$\frac{P_r}{2\phi_c P_n} + \left(\frac{M_{rx}}{\phi_b M_{nx}} + \frac{M_{ry}}{\phi_b M_{ny}} \right) \leq 1.0 \text{ for } \frac{P_r}{\phi_c P_n} < 2.0 \quad (3.37)$$

where P_r is the axial demand determined from modified response spectrum analysis, $\phi_c P_n$ is the axial capacity of the member, M_{rx} is the strong axis bending moment demand determined from modified response spectrum analysis, M_{ry} is the weak axis bending moment, $\phi_b M_{nx}$ and $\phi_b M_{ny}$ are strong axis bending moment capacity of the member.

All the connections considered as pin connections in the CBF part of the structure and the brace members are designed for axial load only. SC-CBF braces, however,

require more robust connections and the moment capacities of these connections are not negligible (Roke et al. 2010). Therefore, the SC-CBF braces are designed for axial load-moment interaction.

3.2.2 CBF Design

The code-based design procedure has been applied for the CBF system, which is based on the ELF procedure (ASCE 2010). The CBF members are designed using the AISC specification (AISC 2010a) and members are further modified to satisfy the AISC seismic provisions (AISC 2010b) so that the special concentrically braced frame (SCBF) coefficients and factors have been applied. Inter-story drift limitations have also been considered based on the code criteria (ASCE 2010).

3.3 Dual System Configuration

There are several ways to combine SC-CBFs and CBFs to comprise the dual system. In this study six different combinations are investigated in two groups of structures, considering six-story and ten-story buildings (as shown in Figure 3.9). For the group of six-story structures, the first studied configuration has a two-story SC-CBF in the lower stories and four-story CBF in the upper stories (dual system 2-4, or D2-4), the second studied configuration has a three-story SC-CBF and a three-story CBF (D3-3) and the final studied configuration consists of a four-story SC-CBF and a two-story CBF (D4-2). For the group of ten-story structures, the first studied configuration has a four-story SC-CBF in the lower stories and six-story CBF in the upper stories (D4-6), the second

studied configuration has a five-story SC-CBF and a five-story CBF (D5-5) and the final studied configuration consists of a seven-story SC-CBF and a three-story CBF (D7-3).

The SC-CBF part of each dual system includes a distribution strut that distributes the mid-bay PT force at the top of the SC-CBF part (e.g., the fourth floor level for D4-2) over multiple stories, reducing the axial forces in the upper story's braces and allowing for more uniformly-sized braces over the height of the SC-CBF.

Based on the described design process, the member selections for the CBF design and all six dual system designs (as well as the required design PT bar area for the dual system designs) are summarized in Table 3.2 and Table 3.3 for the six-story and ten-story structures, respectively.

Table 3.1 Regression coefficients a , b , c , and d for Equations 13 and 14 (Seo 2005)

Site class	β_E (%)	a	b	c	d
D	0	0.729	0.399	0.624	0.0657
	12.5	0.657	0.327	0.678	0.0756
	25	0.597	0.288	0.728	0.0677
	100	0.457	0.500	0.872	-0.305

Table 3.2 Member types and size for the six-story dual systems and CBF system

Structure	Story	SC-CBF part					A_{PT} (in ²)	CBF part		
		Brace	Beam	Gravity Column	SC-CBF Column	Strut		Brace	Beam	CBF Column
D2-4	1	W14×311	W21×111	W14×132	W14×342	—	23.58	—	—	—
	2	W14×211	W24×192	W14×132	W14×342	W14×311		—	—	—
	3	—	—	—	—	—		W14×132	W18×97	W14×132
	4	—	—	—	—	—		W14×132	W18×65	W14×132
	5	—	—	—	—	—		W12×120	W18×65	W14×132
	6	—	—	—	—	—		W12×96	W18×65	W14×132
D3-3	1	W14×283	W21×122	W14×145	W14×283	—	25.28	—	—	—
	2	W14×193	W21×166	W14×145	W14×283	W14×257		—	—	—
	3	W14×933	W21×201	W14×132	W14×233	W14×257		—	—	—
	4	—	—	—	—	—		W14×132	W18×97	W14×132
	5	—	—	—	—	—		W14×132	W18×86	W14×82
	6	—	—	—	—	—		W12×96	W18×86	W14×82
D4-2	1	W14×211	W21×93	W14×145	W14×342	—	23.72	—	—	—
	2	W14×176	W21×111	W14×145	W14×342	—		—	—	—
	3	W14×257	W21×122	W14×132	W14×233	—		—	—	—
	4	W14×193	W21×166	W14×132	W14×233	W14×342		—	—	—
	5	—	—	—	—	—		W12×120	W18×86	W14×82
	6	—	—	—	—	—		W12×120	W18×65	W14×82
CBF	1							W12×96	W21×57	W14×283
	2							W14×82	W21×57	W14×257
	3							W14×82	W21×57	W14×145
	4							W14×68	W21×57	W14×145
	5							W14×68	W21×57	W14×53
	6							W14×68	W21×57	W14×53

Table 3.3 Member types and size for ten-story dual systems and CBF system

Structure	Story	SC-CBF part					A_{PT} (in ²)	CBF part		
		Brace	Beam	Gravity Column	SC-CBF Column	Strut		Brace	Beam	CBF Column
D4-6	1	W14×342	W18×86	W14×176	W14×605	—	39.3	—	—	—
	2	W14×283	W18×130	W14×176	W14×605	W14×342		—	—	—
	3	W14×342	W18×130	W14×176	W14×426	W14×342		—	—	—
	4	W14×370	W21×201	W14×176	W14×426	W14×500		—	—	—
	5	—	—	—	—	—		W14×145	W18×86	W14×176
	6	—	—	—	—	—		W14×145	W18×86	W14×176
	7	—	—	—	—	—		W12×136	W18×86	W14×145
	8	—	—	—	—	—		W12×136	W18×86	W14×145
	9	—	—	—	—	—		W12×120	W18×86	W14×82
	10	—	—	—	—	—		W12×120	W18×65	W14×82
D5-5	1	W14×342	W18×86	W14×176	W14×550	—	36.68	—	—	—
	2	W14×283	W18×86	W14×176	W14×550	—		—	—	—
	3	W14×311	W18×86	W14×159	W14×426	—		—	—	—
	4	W14×257	W18×97	W14×159	W14×426	W14×233		—	—	—
	5	W14×311	W21×166	W14×159	W14×426	W14×233		—	—	—
	6	—	—	—	—	—		W12×136	W18×86	W14×159
	7	—	—	—	—	—		W12×136	W18×86	W14×159
	8	—	—	—	—	—		W12×136	W18×86	W14×159
	9	—	—	—	—	—		W12×136	W18×86	W14×132
	10	—	—	—	—	—		W12×96	W18×86	W14×132
D7-3	1	W14×311	W18×86	W14×176	W14×455	—	31.44	—	—	—
	2	W14×283	W18×86	W14×176	W14×455	—		—	—	—
	3	W14×283	W18×86	W14×159	W14×426	—		—	—	—
	4	W14×257	W18×86	W14×159	W14×426	—		—	—	—
	5	W14×257	W18×86	W14×145	W14×370	—		—	—	—
	6	W14×257	W18×86	W14×145	W14×370	W14×233		—	—	—
	7	W14×311	W21×201	W14×132	W14×342	W14×233		—	—	—
	8	—	—	—	—	—		W12×136	W18×86	W14×132
	9	—	—	—	—	—		W12×136	W18×86	W14×132
	10	—	—	—	—	—		W12×106	W18×86	W14×132
CBF	1							W12×136	W18×65	W14×370
	2							W12×106	W18×65	W14×342
	3							W12×106	W18×65	W14×233
	4							W12×96	W18×65	W14×233
	5							W12×96	W18×65	W14×132
	6							W12×96	W18×65	W14×132
	7							W14×68	W18×65	W12×96
	8							W14×68	W18×65	W12×96
	9							W14×68	W18×65	W12×96
	10							W14×68	W18×65	W12×96

Figure 3.1 SC-CBF concept: (a) schematic of SC-CBF system; and (b) rocking behavior.

Figure 3.2 Prototype structure of one proposed dual system elevation (D4-2)

Figure 3.3 Dual system lateral force-displacement behavior

Figure 3.4 Typical floor plan of buildings: (a) 6-story building; (b) 10-story building

Figure 3.6 Force-displacement (F - u) behavior of SC systems with varying β_E (Seo and Sause 2005): (a) SC model with β_E ; (b) $\beta_E = 0\%$; (c) $\beta_E = 25\%$; (d) $\beta_E = 50\%$

Figure 3.7 – Schematic of idealized overturning moment versus roof drift response of a Dual system

Figure 3.8 – Design response spectrum (ASCE 2010)

Figure 3.9 Dual system configurations: (a) D2-4; (b) D3-3; (c) D4-2; (d) D4-6; (e) D5-5 and (f) D7-3

CHAPTER IV

NUMERICAL ANALYSIS

In order to compare the seismic performance of the prototype structures, numerical analysis is conducted using nonlinear finite element modeling of the prototype structures subjected to a suite of ground motions. The dynamic responses obtained from the numerical analysis will then be used to develop probabilistic demand models for assessing seismic fragilities.

In this chapter, first the prototype structures and loading assumptions used for the dual system and CBF system are introduced. Then, the nonlinear analytical model used in the numerical simulation is discussed. Next, the ground motions used in this study are introduced. Finally, a summary of the nonlinear dynamic responses to FOE-level, DBE-level and MCE-level ground motions from numerical simulations is presented.

4.1 Prototype Structures

Two groups of prototype structures with identical configurations in plan are designed, one using a conventional CBF and the other one using the proposed dual system as the lateral force resisting system. The prototype structures are six-story and ten-story office buildings designed for a site with stiff soil conditions (site class D (ASCE 2010)) near Los Angeles. Typical elevations of the CBFs and one configuration of the

dual system are shown in Figure 4.1. The design loads for the prototype structure are as follows. The dead loads include the concrete floor slab, steel floor deck, mechanical equipment, floor and ceiling finishes, cladding weight and the an estimated weight per square foot of structural steel. Tables 4.1 and 4.2 show the dead loads and live loads, respectively, used for the design of the dual system and CBF system. The seismic mass of each floor, excluding the roof level, is taken as the dead load plus 15 psf for partitions (ASCE 2010). The tributary area for each lateral-load resistant system is equal to one-fourth of the total area in six-story building and one-sixth of the total area in ten-story building. The tributary seismic masses associated with each lateral load resisting systems (CBF and dual) are presented in Tables 4.3 and 4.4, taking into account the dead loads and partitions loads (ASCE 2010) within the tributary area, consistent with the values used in previous studies of SC-CBF systems (e.g., [Roke and Jeffers 2012, Roke and Hasan 2012]).

4.2. Numerical Modeling

For each prototype structure (one CBF system and six dual system configurations), one entire braced frame is modeled in OpenSees (McKenna 1997) for nonlinear dynamic analysis. OpenSees is an open-source computation framework for earthquake engineering simulations. The object-oriented framework of this software permits a wide variety of materials, sections, members and structure level characteristics to be combined to achieve a versatile physical theory representation of various members, including simple braces and more general three-dimensional beam-columns. Rigid floor diaphragms are assumed and implemented through an equal degree-of-freedom

command. The foundation is modeled as rigid. A lean-on column is used to approximate the stiffness contribution of the gravity columns within the tributary area of the modeled braced bay and also to capture global P- Δ effects. To estimate the properties of the lean-on column, it is assumed that typical gravity columns are arranged such that half are oriented with their strong axis in the same plane as the braced frame and half are oriented with their weak axis in the same plane as the braced frame. The lean-on column is modeled as a continuous column with a pinned base. Rayleigh damping is applied to the dynamic analysis models, with damping ratios of 2% for the first mode and 5% for the third mode. The seismic mass of the structure, including dead loads, member weights, and 25% of live load, is applied at the lean-on-column as a lumped mass at each floor level. Further details of the CBF model and dual system models are provided in the following sections.

4.2.1 CBF Model

The elevations of the CBF systems for the six-story and ten-story buildings are shown in Figure 4.1. For the CBF model, the beams and columns and braces are modeled using elastic-plastic materials with fiber cross-sections, and the columns are modeled with pinned bases. In this study, 96 fiber sections were created for each beam, column, and brace member. The elastic-plastic material model used for the members is a uniaxial Giuffre-Menegotto-Pinto steel material model with isotropic strain hardening (McKenna 1997) with a 50 ksi yield strength. To simulate the hysteretic behavior of conventional braces, the methodology presented by Uriz & Mahin (2004) is used, in which each brace member is modeled as two separate elements along its length. Uriz and Mahin (2004)

found that the number of elements that make up a brace has small effect on the force-displacement of the braces. They conclude that braces can be modeled by only two segments to avoid complexity. To model potential buckling in braces, initial imperfections are introduced by slightly offsetting the mid-length node out of plane. The connections between the braces and the beams and columns are pinned following common CBF design practice so that the braces carry no moment. Without the initial out-of-straightness, the pin-ended brace will behave as an ideal, perfectly straight uniaxial strut, with no global buckling possible. A small initial camber introduces a perturbation that triggers buckling. In this study, initial out-of-straightness is assumed to be 1% of the total length of the braces based on previous research (Uriz and Mahin 2004).

4.2.2 Dual System Model

The beams, columns, and struts of the SC-CBF part of the dual system are modeled as elastic-plastic members with isotropic strain hardening. The SC-CBF braces are modeled as the same as those of the CBF system: elastic-plastic material properties are used to model the material nonlinearity and a slight out-of-plane offset is implemented to model the buckling behavior. Due to the magnitude of the forces in the SC-CBF, the connections at the end of the SC-CBF members are fixed and can carry moments (Roke et al. 2010). The PT bars are modeled using bilinear yielding beam-column elements with negligible flexural and shear stiffness. The elements used for the beams, columns, braces, struts, and PT bars include P- Δ effects. The lateral-load bearings are modeled utilizing horizontally-oriented contact-friction elements, which are zero-length elements that can resist only compressive normal forces (in this case, in the horizontal direction) and

develop friction forces perpendicular to the normal force. The friction force capacity is proportional to the compressive normal force. The gap opening behavior at the base of each SC-CBF column is modeled using a vertically-oriented contact element. The CBF part of the dual system is modeled as described for the CBF model.

4.3 Ground Motion Selection

An important step in evaluating the seismic behavior of structures through nonlinear dynamic analysis is the selection of a representative suite of earthquake ground motions. The selected ground motion records should capture the characteristics of the possible seismic hazard, including return period, intensity, frequency content, and duration. Three seismic hazard levels are defined in FEMA 450 (BSSC 2003): the maximum considered earthquake (MCE); the design basis earthquake (DBE); and the maximum probable event, which is often called the frequently occurring earthquake (FOE). These hazard levels are described briefly here.

4.3.1 Maximum Considered Earthquake (MCE)

FEMA 450 (BSSC 2003) defines the MCE as a ground motion intensity that has a 2% probability of exceedance in 50 years, corresponding to a 2500-year return period. This intensity level is intended to be “reasonably representative of the most severe ground motion ever likely to affect a site.”

4.3.2 Design Basis Earthquake (DBE)

The DBE is defined in FEMA 450 (BSSC 2003) as a ground motion intensity that is two-thirds that of the MCE. The DBE corresponds approximately to a ground motion intensity that has a return period of several hundred years corresponding to probabilities of exceedance of 10% in 50 years.

4.3.3 Frequently Occurring Earthquake (FOE)

FEMA 450 (BSSC 2003) defines a “maximum probable event,” or frequently occurring earthquake (FOE), as a ground motion intensity that has a 50% probability of exceedance in 50 years, corresponding to a 72-year return period. The FOE is used to establish the performance of structures under earthquake ground motions that are less severe but occur more frequently than the MCE or DBE.

4.4 Ground Motion Suite

The nonlinear dynamic analysis presented in this study utilized the ground motion suite collected by Somerville et al. (1997), which corresponded to three different sites: Los Angeles (LA), Seattle (SE), and Boston (BO). This ground motion suite has been widely used to simulate DBE- and MCE-level earthquakes (e.g., Uriz & Mahin 2004, Lee & Foutch 2004, Mayes et al. 2005). The full suite of ground motions includes two 20-record ground motion sets for each site, corresponding to probabilities of exceedance of 10% in 50 years (approximately DBE level) and 2% in 50 years (MCE level). For the LA site, a third set of 20 ground motions with a probability of exceedance of 50% in 50 years (FOE level) is included. The summary of the SAC ground motions is presented in Table

4.5 All of the ground motions are for stiff soil sites and are scaled to approximately match the target response spectra. The target response spectra for this study are for NEHRP site class *D*, which corresponds to a stiff soil (ASCE 2010). The spectral accelerations for the selected MCE, DBE and FOE level ground motions are shown in Figures 4.2, 4.3 and 4.4 respectively.

4.4 Numerical Modeling Results

Nonlinear dynamic analyses were conducted to determine the system response of the simulated buildings under the selected scaled ground motions. A comprehensive database of structural response parameters (engineering demand parameters, or EDPs) are developed as part of this study. There are many possible choices for EDPs such as roof displacements, interstory drift ratios, joint rotations, and floor accelerations. EDP selection depends on type of structure, type of analysis and the performance level of interest (Kinali and Elingwood 2007). Interstory drift ratios provide insight regarding the potential for local or global instability or collapse of the structural system. It also related closely to non-structural elements that are sensitive to relative displacements, such as partitions, exterior walls, and glass blocks. Floor accelerations, on the other hand, are suitable for expressing damage to non-structural building contents that are sensitive to inertia forces, such as fire suppression systems and HVAC systems. In this study, interstory drift and absolute floor acceleration were selected as EDPs. Tables A.1 to A.4 show the maximum interstory drift and maximum floor acceleration for the 6-story and 10-story buildings under 140 ground motion records. These peak responses of the

buildings are used to develop probabilistic demand models for the EDPs, as presented in the following chapter.

Sample results from nonlinear dynamic analyses for the selected EDPs are presented in this section. Figures 4.5 to 4.10 show some sample responses of the dual system and CBF system. Figure 4.5(a) shows the roof displacement response of Dual3-3 system under the (site) DBE-level Imperial Valley ground motion. Figures 4.5(b) and 4.5(c) show the ground acceleration and ground velocity for the earthquake record. Figure 4.6 shows similar results for 6-story CBF building. Figures 4.7 and 4.8 show the time history of displacement of the lean-on column at different floor levels for D3-3 and the 6-story CBF system, respectively. Figure 4.9(a) shows ground acceleration history for one DBE-level ground motion. Figure 4.9(b) show relative roof acceleration with respect to the base of the building for D3-3 system. Figure 4.9(c) shows the absolute roof acceleration for the D3-3 system under the (site) DBE-level Imperial Valley ground motion. Figure 4.10 shows similar results for 6-story CBF building. Figure 4.11 shows the comparative roof drift response for the CBF system and the Dual4-2 system under the LA DBE-level Imperial Valley ground motion. As expected, the dual system tends to have less residual drift and its dynamic response tends to damp out faster than that of the CBF. The difference in the rate of damping is due to friction energy dissipation in the lateral-load bearings in the dual system. Figure 4.12 shows typical comparative floor acceleration response for 10-story CBF and dual system. As expected, the dual system has slightly larger acceleration due to the rocking behavior of the system.

Table 4.1 – Dead loads at each floor level for prototype structures

Dead Load	1 st Floor (psf)	Other floors (psf)	Roof (psf)
Floor/roof slab	43	43	43
Floor/roof deck	3	3	3
Roofing material	0	0	10
Mechanical weight	10	10	25
Ceiling material	5	5	0
Floor finish	2	2	0
Steel fireproofing	2	2	2
Structural steel	15	15	15
Exterior wall (per sq. ft. of floor area)	7.9	6.9	5.1
Total	87.9	86.9	103.1

Table 4.2 – Live loads at each floor level for prototype structures

Live Load	Floors (psf)	Roof (psf)
Office	50	0
Partitions	15	0
Roof live load	0	20

Table 4.3 – Summary of tributary seismic masses for 6-story prototype structures

Floor	Mass (k-s ² /in)
1	2.153
2	2.139
3	2.139
4	2.139
5	2.139
6	1.01

Table 4.4 – Summary of tributary seismic masses for 10-story prototype structures

Floor	Mass (k-s ² /in)
1	1.436
2	1.426
3	1.426
4	1.426
5	1.426
6	1.426
7	1.426
8	1.426
9	1.426
10	0.673

Table 4.5 Summary of SAC report ground motion characteristics

Record	M	R (km)	Site	Level	T (sec)	SF
Imperial Valley, 1940, El Centro	6.9	10	LA	DBE	39.38	2.01
Imperial Valley, 1940, El Centro	6.9	10	LA	DBE	39.38	2.01
Imperial Valley, 1979, Array #05	6.5	4.1	LA	DBE	39.38	1.01
Imperial Valley, 1979, Array #05	6.5	4.1	LA	DBE	39.38	1.01
Imperial Valley, 1979, Array #06	6.5	1.2	LA	DBE	39.08	0.84
Imperial Valley, 1979, Array #06	6.5	1.2	LA	DBE	39.08	0.84
Landers, 1992, Barstow	7.3	36	LA	DBE	79.98	3.2
Landers, 1992, Barstow	7.3	36	LA	DBE	79.98	3.2
Landers, 1992, Yermo	7.3	25	LA	DBE	79.98	2.17
Landers, 1992, Yermo	7.3	25	LA	DBE	79.98	2.17
Loma Prieta, 1989, Gilroy	7	12	LA	DBE	39.98	1.79
Loma Prieta, 1989, Gilroy	7	12	LA	DBE	39.98	1.79
Northridge, 1994, Newhall	6.7	6.7	LA	DBE	59.98	1.03
Northridge, 1994, Newhall	6.7	6.7	LA	DBE	59.98	1.03
Northridge, 1994, Rinaldi RS	6.7	7.5	LA	DBE	14.945	0.79
Northridge, 1994, Rinaldi RS	6.7	7.5	LA	DBE	14.945	0.79
Northridge, 1994, Sylmar	6.7	6.4	LA	DBE	59.98	0.99
Northridge, 1994, Sylmar	6.7	6.4	LA	DBE	59.98	0.99
North Palm Springs, 1986	6	6.7	LA	DBE	59.98	2.97
North Palm Springs, 1986	6	6.7	LA	DBE	59.98	2.97
Long Beach, Vernon CMD Bldg.	6.5	1.2	SE	DBE	39.08	0.49
Long Beach, Vernon CMD Bldg.	6.5	1.2	SE	DBE	39.08	0.49
Morgan Hill, 1984, Gilroy	6.2	15	SE	DBE	59.98	2.84
Morgan Hill, 1984, Gilroy	6.2	15	SE	DBE	59.98	2.84
West. Washington, Olympia, 1949	6.5	56	SE	DBE	79.98	1.86
West. Washington, Olympia, 1949	6.5	56	SE	DBE	79.98	1.86
West. Washington, Seattle Army B., 1949	6.5	80	SE	DBE	66.68	5.34
West. Washington, Seattle Army B., 1949	6.5	80	SE	DBE	66.68	5.34
North Palm Springs, 1986	6	6.7	SE	DBE	59.98	1.71
North Palm Springs, 1986	6	6.7	SE	DBE	59.98	1.71
Puget Sound, Wa., Olympia, 1949	7.1	80	SE	DBE	81.82	4.3
Puget Sound, Wa., Olympia, 1949	7.1	80	SE	DBE	81.82	4.3
Puget Sound, Wa., Federal OFC B., 1949	7.1	61	SE	DBE	74.08	5.28
Puget Sound, Wa., Federal OFC B., 1949	7.1	61	SE	DBE	74.08	5.28
Eastern Wa., Tacoma County, 1949	7.1	60	SE	DBE	59.98	8.68
Eastern Wa., Tacoma County, 1949	7.1	60	SE	DBE	59.98	8.68
Llolleo, Chile 1985	8	42	SE	DBE	99.975	1.24
Llolleo, Chile 1985	8	42	SE	DBE	99.975	1.24
Vinadel Mar, Chile, 1985	8	42	SE	DBE	99.975	1.69

Table 4.5 Summary of SAC report ground motion characteristics (Continued)

Record	M	R (km)		Level	T (sec)	SF
Vinadel Mar, Chile, 1985	8	42	SE	DBE	99.975	1.69
Simulation, hanging wall	6.5	30	BO	DBE	29.99	0.39
Simulation, hanging wall	6.5	30	BO	DBE	29.99	0.39
Simulation, foot wall	6.5	30	BO	DBE	29.99	0.54
Simulation, foot wall	6.5	30	BO	DBE	29.99	0.54
New Hampshire, 1982	4.3	8.4	BO	DBE	19.23	10.75
New Hampshire, 1982	4.3	8.4	BO	DBE	19.23	10.75
Nahanni, 1985	6.9	9.6	BO	DBE	20.335	0.09
Nahanni, 1985	6.9	9.6	BO	DBE	20.335	0.09
Nahanni, 1985	6.9	6.1	BO	DBE	18.755	0.2
Nahanni, 1985	6.9	6.1	BO	DBE	18.755	0.2
Nahanni, 1985	6.9	18	BO	DBE	19.015	0.92
Nahanni, 1985	6.9	18	BO	DBE	19.015	0.92
Saguenay, 1988	5.9	96	BO	DBE	17.735	1.57
Saguenay, 1988	5.9	96	BO	DBE	17.735	1.57
Saguenay, 1988	5.9	98	BO	DBE	29.57	3.21
Saguenay, 1988	5.9	98	BO	DBE	29.57	3.21
Saguenay, 1988	5.9	118	BO	DBE	39.05	3.25
Saguenay, 1988	5.9	118	BO	DBE	39.05	3.25
Saguenay, 1988	5.9	132	BO	DBE	33.24	3.34
Saguenay, 1988	5.9	132	BO	DBE	33.24	3.34
1995 Kobe	6.9	3.4	LA	MCE	59.98	1.15
1995 Kobe	6.9	3.4	LA	MCE	59.98	1.15
1989 Loma Prieta	7	3.5	LA	MCE	24.99	0.82
1989 Loma Prieta	7	3.5	LA	MCE	24.99	0.82
1994 Northridge	6.7	7.5	LA	MCE	14.945	1.29
1994 Northridge	6.7	7.5	LA	MCE	14.945	1.29
1994 Northridge	6.7	6.4	LA	MCE	59.98	1.61
1994 Northridge	6.7	6.4	LA	MCE	59.98	1.61
1974 Tabas	7.4	1.2	LA	MCE	49.98	1.08
1974 Tabas	7.4	1.2	LA	MCE	49.98	1.08
Elysian Park (simulated)	7.1	17.5	LA	MCE	29.99	1.43
Elysian Park (simulated)	7.1	17.5	LA	MCE	29.99	1.43
Elysian Park (simulated)	7.1	10.7	LA	MCE	29.99	0.97
Elysian Park (simulated)	7.1	10.7	LA	MCE	29.99	0.97
Elysian Park (simulated)	7.1	11.2	LA	MCE	29.99	1.1
Elysian Park (simulated)	7.1	11.2	LA	MCE	29.99	1.1
Palos Verdes (simulated)	7.1	1.5	LA	MCE	59.98	0.9
Palos Verdes (simulated)	7.1	1.5	LA	MCE	59.98	0.9

Table 4.5 Summary of SAC report ground motion characteristics (Continued)

Record	M	R (km)		Level	T (sec)	SF
Palos Verdes (simulated)	7.1	1.5	LA	MCE	59.98	0.88
Palos Verdes (simulated)	7.1	1.5	LA	MCE	59.98	0.88
1992 Mendocino	7.1	8.5	SE	MCE	59.98	0.98
1992 Mendocino	7.1	8.5	SE	MCE	59.98	0.98
1949 Olympia	6.5	56	SE	MCE	79.98	4.35
1949 Olympia	6.5	56	SE	MCE	79.98	4.35
1965 Seattle	7.1	80	SE	MCE	81.82	10.04
1965 Seattle	7.1	80	SE	MCE	81.82	10.04
1985 Valpariso	8	42	SE	MCE	99.975	2.9
1985 Valpariso	8	42	SE	MCE	99.975	2.9
1985 Valpariso	8	42	SE	MCE	99.975	3.96
1985 Valpariso	8	42	SE	MCE	99.975	3.96
Deep Interplate (simulation)	7.9	65	SE	MCE	79.98	3.84
Deep Interplate (simulation)	7.9	65	SE	MCE	79.98	3.84
1978 Miyagi-oki	7.4	66	SE	MCE	79.98	1.78
1978 Miyagi-oki	7.4	66	SE	MCE	79.98	1.78
Shallow Interplate (simulation)	7.9	15	SE	MCE	79.98	0.94
Shallow Interplate (simulation)	7.9	15	SE	MCE	79.98	0.94
Shallow Interplate (simulation)	7.9	15	SE	MCE	79.98	1.49
Shallow Interplate (simulation)	7.9	15	SE	MCE	79.98	1.49
simulation, foot wall	6.5	30	BO	MCE	29.99	0.99
simulation, foot wall	6.5	30	BO	MCE	29.99	0.99
simulation, foot wall	6.5	30	BO	MCE	29.99	0.84
simulation, foot wall	6.5	30	BO	MCE	29.99	0.84
simulation, foot wall	6.5	30	BO	MCE	29.99	0.63
simulation, foot wall	6.5	30	BO	MCE	29.99	0.63
Nahanni, 1985 Station 1	6.9	9.6	BO	MCE	20.335	0.27
Nahanni, 1985 Station 1	6.9	9.6	BO	MCE	20.335	0.27
Nahanni, 1985 Station 2	6.9	6.1	BO	MCE	18.755	0.56
Nahanni, 1985 Station 2	6.9	6.1	BO	MCE	18.755	0.56
Nahanni, 1985 Station 3	6.9	18	BO	MCE	19.015	2.63
Nahanni, 1985 Station 3	6.9	18	BO	MCE	19.015	2.63
Saguenay, 1988	5.9	96	BO	MCE	17.735	4.48
Saguenay, 1988	5.9	96	BO	MCE	17.735	4.48
Saguenay, 1988	5.9	98	BO	MCE	29.57	9.21
Saguenay, 1988	5.9	98	BO	MCE	29.57	9.21
Saguenay, 1988	5.9	118	BO	MCE	39.05	9.3
Saguenay, 1988	5.9	118	BO	MCE	39.05	9.3
Saguenay, 1988	5.9	132	BO	MCE	33.24	9.58

Table 4.5 Summary of SAC report ground motion characteristics (Continued)

Record	M	R (km)		Level	T (sec)	SF
Saguenay, 1988	5.9	132	BO	MCE	33.24	9.58
Coyote Lake, 1979	5.7	8.8	LA	FOE	39.38	2.28
Coyote Lake, 1979	5.7	8.8	LA	FOE	39.38	2.28
Imperial Valley, 1979	6.5	1.2	LA	FOE	39.08	0.4
Imperial Valley, 1979	6.5	1.2	LA	FOE	39.08	0.4
Kern, 1952	7.7	107	LA	FOE	78.6	2.92
Kern, 1952	7.7	107	LA	FOE	78.6	2.92
Landers, 1992	7.3	64	LA	FOE	79.98	2.63
Landers, 1992	7.3	64	LA	FOE	79.98	2.63
Morgan Hill, 1984	6.2	15	LA	FOE	59.98	2.35
Morgan Hill, 1984	6.2	15	LA	FOE	59.98	2.35
Parkfield, 1966, Cholame 5W	6.1	3.7	LA	FOE	43.92	1.81
Parkfield, 1966, Cholame 5W	6.1	3.7	LA	FOE	43.92	1.81
Parkfield, 1966, Cholame 8W	6.1	8	LA	FOE	26.14	2.92
Parkfield, 1966, Cholame 8W	6.1	8	LA	FOE	26.14	2.92
North Palm Springs, 1986	6	9.6	LA	FOE	59.98	2.75
North Palm Springs, 1986	6	9.6	LA	FOE	59.98	2.75
San Fernando, 1971	6.5	1	LA	FOE	79.46	1.3
San Fernando, 1971	6.5	1	LA	FOE	79.46	1.3
Whittier, 1987	6	17	LA	FOE	39.98	3.62
Whittier, 1987	6	17	LA	FOE	39.98	3.62

Figure 4.1 Elevations of 10-story and 6-story CBFs and dual system D4-2

Figure 4.2 Response spectra for MCE-level ground motions

Figure 4.3 Response spectra for DBE-level ground motions

Figure 4.4 Response spectra for FOE-level ground motions

Figure 4.5 Sample results for (site) DBE-level Imperial Valley ground motion for D3-3:
(a) roof displacement, (b) ground acceleration, and (c) ground velocity

Figure 4.6 Sample results for the (site) DBE-level Imperial Valley ground motion for 6-story CBF: (a) roof displacement, (b) ground acceleration, and (c) ground velocity

Figure 4.7 D3-3 floor displacement responses for the (site) DBE-level Imperial Valley ground motion

Figure 4.8 6-story CBF floor displacement responses for the (site) DBE-level Imperial Valley ground motion

Figure 4.9 D3-3 responses to the (site) DBE-level Imperial Valley ground motion: (a) ground acceleration, (b) relative roof acceleration and (c) absolute roof acceleration

Figure 4.10 6-story CBF responses to the (site) DBE-level Imperial Valley ground motion: (a) ground acceleration, (b) relative roof acceleration and (c) absolute roof acceleration

Figure 4.11 Roof drift response for 6-story buildings to the LA DBE-level Imperial Valley ground motion

Figure 4.12 Floor acceleration response for 10-story buildings to the LA DBE-level Imperial Valley ground motion

CHAPTER V

PROBABILISTIC DEMAND MODEL

Seismic demand models are a critical component in performance-based seismic design and seismic risk assessment. Currently, nonlinear static procedures (NSPs) are commonly-used tools to predict seismic demands by making use of force-deformation curves generated from nonlinear static pushover analysis. However, NSPs are deterministic approaches and do not account for the uncertainties and variability in the ground motions and in the structural responses. Probabilistic seismic demand models are capable of capturing the uncertainties in the ground motions and the structural responses.

In this chapter, probabilistic demand models are developed using the numerical analysis results discussed in Chapter IV; Chapter VI shows how these demand models are used to assess seismic fragility to characterize the seismic performance of the CBF system and the dual system. In this study, the randomness from ground motions is considered and prevailing uncertainties, such as the uncertainties in the structural properties, statistical uncertainties, and the model errors, are incorporated to develop probabilistic seismic demand models. The proposed probabilistic demand models are formulated by adding correction terms to existing conventional deterministic demand models. The correction terms are used to correct the bias and improve the accuracy in the

selected deterministic models. The model parameters are estimated through the regression analysis based on the demand data obtained from nonlinear analysis of detailed numerical models (Tables A.1 to A.4) subjected to a set of representative ground motion records (Table 4.5). Various response quantities from nonlinear dynamic analysis can be selected to represent the seismic demand on a structure. In this study, as mentioned previously, interstory drift and floor acceleration are considered as the EDPs of interest.

In this chapter, the formulation of the proposed demand model is first introduced, and then the deterministic demand model that is commonly used in practice is discussed. Next, the correction term is presented. Finally, the developed seismic demand models for the selected EDPs are presented.

5.1 Formulation of Probabilistic Demand Model

Seismic demand models usually consist of earthquake intensity measures (IMs) as predictors. Some researchers use a single IM (e.g., peak ground acceleration (PGA) or response spectrum acceleration (S_a)) as the predictor (e.g., Nielson & DesRoches 2007); while others use multiple IMs (e.g., a vector-valued IM composed of $S_a(T_1)$ and the ratio $S_a(2T_1)/S_a(T_1)$ (Backer 2007), in which T_1 is first mode period of the structure). In this study, the probabilistic demand model is developed by adding correction terms to a selected deterministic demand model that is used in practice, following the formulation proposed by Huang et al. (2010). The formulation of the probabilistic demand model is written as:

$$D(\mathbf{x}, \Theta) = \theta_d \hat{d}(\mathbf{x}) + \gamma(\mathbf{x}, \Theta) + \sigma \varepsilon \quad (5.1)$$

where $D(\mathbf{x}, \Theta)$ is the demand measure (or a suitable transformation); $\hat{d}(\mathbf{x})$ is the deterministic demand (or a suitable transformation); θ_d is the coefficient for the deterministic demand; $\gamma(\mathbf{x}, \Theta)$ is a correction term for correcting the bias and random errors in $\hat{d}(\mathbf{x})$, $\Theta = (\theta, \sigma)$ is a vector of unknown model parameters, $\theta = (\theta_{k0}, \dots, \theta_{kpk})$; σ is the standard deviation of the model error; ε is a normal random variable with zero mean and unit variance; and $\mathbf{x}$ is a vector of basic variables (e.g., material properties, member dimensions, and imposed boundary conditions). Two assumptions are used in assessing the demand model in Eq. 5.1: the homoskedasticity assumption (i.e., σ is constant and independent of $\mathbf{x}$) and the normality assumption (i.e., the model error, $\sigma \varepsilon$, is normally distributed). Both assumptions can usually be satisfied by using a variance stabilizing transformation of the demand quantities of interest (Box and Cox 1964). Diagnostic plots can be used to check the suitability of the transformation (e.g., Rao and Toutenburg 1997, Sheather 2009). In this study, the natural logarithmic transformation of the maximum inter-story drift demands and maximum floor acceleration demands was adopted to stabilize the residual variance.

5.2 Deterministic Demand Model

The probabilistic demand model formulation shown in Eq. 5.1 uses correction terms to remove the bias and improve the accuracy of the deterministic models that are currently used in practice, preserving the simplicity of the deterministic model to facilitate the practical application of the probabilistic demand model. In this study, the

coefficient method (CM) (ASCE 2007) is adopted as the deterministic demand to predict interstory drift. CM uses an equivalent single degree of freedom (SDOF) system to determine the target displacement (i.e., the maximum roof displacement, Δ_{max}) for a corresponding multi-degree-of-freedom building. The value of Δ_{max} can be predicted as follows:

$$\Delta_{max} = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \quad (5.2)$$

where C_0, C_1 , and C_2 are modification factors (ASCE 2007), T_e is the effective fundamental period of the structure, S_a is the mean spectral acceleration at the fundamental period of structure, and g is the acceleration due to gravity. The modification factors are intended to account for different impacts on the seismic responses such as the earthquake properties, the structural system type, the relationship between the inelastic displacement demand and the linear elastic response, the effect of hysteresis shape, cyclic stiffness degradation, and strength deterioration. Additionally, T_e can be calculated as follows:

$$T_e = T_1 \sqrt{\frac{K_i}{K_e}} \quad (5.3)$$

where T_1 is the elastic fundamental period (seconds) in the direction under consideration, as calculated by elastic modal analysis (ASCE 2007); K_i is the initial elastic stiffness of the building; and K_e is the effective lateral stiffness of the building determined from an

idealized bilinear force-displacement curve that can be obtained from a pushover analysis.

Considering appropriate values for coefficients C_0 , C_1 , and C_2 , maximum roof displacement (Δ_{max}) values for the prototype structures are calculated for each location (SE, LA and BO) and hazard level (50% in 50 years, 10% in 50 years, and 2% in 50 years), corresponding to each ground motion set. Typically, CM predicts Δ_{max} for the mean spectral acceleration in a specific site. In this study, CM was used to calculate Δ_{max} using the pseudo-spectral acceleration ($PSA(T_I)$) of each individual earthquake record to create motion-specific drift predictions. The deterministic demand for inter-story drift ($\hat{d}(\mathbf{x})$) is then approximated by normalizing the roof displacement by the height of the structure.

Figure 5.1 shows the predictions from the deterministic models ($\hat{d}(\mathbf{x})$) vs. responses obtained from the finite element model (FEM) numerical analysis for the CBF system and the SC-CBF parts of the dual systems for the 6-story buildings. If the prediction is perfect, the data points will fall along the 1:1 line (the solid lines in Figure 5.1). The scatter of the data indicates the accuracy of the prediction; less scatter implies a better prediction. As shown in Figure 5.1(a), the majority of the CBF interstory drift prediction data falls under the 1:1 line, which shows that the prediction using the deterministic model underestimates demand. Meanwhile, the data in Figure 5.1(b), 5.1(c), and 5.1(d) is more evenly distributed around the 1:1 line, which indicates that the deterministic model provides a less biased prediction for the 6-story dual systems. It should be noted that no

standard deterministic demand models were found for the floor acceleration or roof acceleration. Therefore, the demand models for the floor acceleration and roof acceleration only consist of explanatory functions of constant and normalized IMs (i.e., the deterministic demand is assumed equal to zero).

5.3 Correction Terms

The candidate correction terms are selected based on their relationships to the residuals between the demand data and $\hat{d}(\mathbf{x})$. By correcting the bias in $\hat{d}(\mathbf{x})$, the developed demand models consider the nonlinear behavior and the inherent uncertainties of structures, while retaining the simplicity of $\hat{d}(\mathbf{x})$ to facilitate their practical implementation. Therefore, correction terms $\gamma(\mathbf{x}, \boldsymbol{\theta}) = \sum_{i=0}^{p_k} \theta_i h_i(\mathbf{x})$ are used to correct bias and improve the accuracy of the deterministic model, in which $h_i(\mathbf{x})$ are explanatory functions; $\boldsymbol{\Theta} = (\boldsymbol{\theta}, \sigma)$ is a vector of unknown model parameters; $\boldsymbol{\theta} = (\theta_{k0}, \dots, \theta_{kp_k})$; p_k+1 is the number of model parameters. In order to detect a potential constant bias, $h_0(\mathbf{x}) = 1$. To reduce the potential under- or over-estimation of the deterministic model, $h_1(\mathbf{x}) = \hat{d}(\mathbf{x})$. Moreover, since round motion characteristics, such as the intensity, frequency content, and duration, could also play a pivotal role in the response of structures (Krawinkler et al. 2003), explanatory functions containing earthquake parameters should be included. The earthquake-related explanatory functions considered in this study are the natural logarithms of the normalized IMs listed in Table 5.1. In Table 5.1, T_1 is the natural frequency of the structure's equivalent SDOF system; $u_g(t)$, $\dot{u}_g(t)$, and $\ddot{u}_g(t)$ denote the

ground motion displacement, velocity, and acceleration, respectively, at time t ; D is the duration of a ground motion record, and $T_D = t(0.95I_A) - t(0.05I_A)$ is the strong ground motion duration based on the time between 2.5% and 97.5% of the Arias intensity I_A (Trifunac and Brady 1975). Note that all the candidate explanatory functions are dimensionless so that they can be used for any unit systems. Among the candidate explanatory functions, only the statistically significant functions will be selected using the following model selection process.

With all the potential explanatory functions (i.e., the model now is a full model), a model selection process is adopted to select the functions that significantly contribute to the model statistically. Note that before conducting the model selection process, functions with high linear correlations (above approximately 90% correlation) among the explanatory functions were removed to eliminate potential problems that occur due to multicollinearity of IMs. The model selection process uses the Mallows CP statistical criterion (Kutner et al. 2004), in which models with the lowest CP values are selected. The advantage of using this criterion is that it attempts to balance the over-fitting and under-fitting regression models. The statistical significance of the parameters, θ , is checked using the extra sum of squares test. To build a robust model, the possible multicollinearity of the explanatory functions is checked using the variance inflation factor (VIF) criterion. The selected models include only statistically significant parameters with no multicollinearity.

5.4 Proposed Probabilistic Demand Models

As a result of the model selection, the exponential functions of deterministic demand ($\hat{d}(\mathbf{x})$) and the natural logarithm of normalized PGV are selected for the proposed demand models for the inter-story drift of the CBF and the dual system (both CBF and SC-CBF parts). Moreover, the natural logarithm of normalized PGA and PSA are selected for the proposed demand models for the floor accelerations and roof accelerations of the CBF and dual systems. The demand models can be expressed as follows:

$$D_{ID}(\mathbf{x}, \boldsymbol{\Theta}_{ID}) = \theta_{ID_0} + \theta_{\hat{d}} \cdot \hat{d}(\mathbf{x}, PSA) + \theta_{ID_1} \cdot \ln\left(\frac{PGV \cdot T_1}{H_c}\right) + \sigma_{ID} \varepsilon \quad (5.4)$$

$$D_A(\mathbf{x}, \boldsymbol{\Theta}_A) = \theta_{A_0} + \theta_{A_1} \cdot \ln\left(\frac{PSA}{g}\right) + \theta_{A_2} \cdot \ln\left(\frac{PGA}{g}\right) + \sigma_A \varepsilon \quad (5.5)$$

$$D_{RA}(\mathbf{x}, \boldsymbol{\Theta}_{RA}) = \theta_{RA_0} + \theta_{RA_1} \cdot \ln\left(\frac{PSA}{g}\right) + \theta_{RA_2} \cdot \ln\left(\frac{PGA}{g}\right) + \sigma_{RA} \varepsilon \quad (5.6)$$

where T_1 is the first natural period of the buildings, PSA is the pseudo-spectral acceleration at T_1 , H_c is the building height, PGV is the peak ground velocity, PGA is the peak ground acceleration, and g is the gravity acceleration.

Two important issues should be addressed before presenting the statistics of the model coefficients in Eqs. 5.4, 5.5, and 5.6. Firstly, it should be noted that the inter-story drift affects the CBF and SC-CBF systems differently, which must be accommodated in the probabilistic demand models. For the CBF system, only one probabilistic demand

model will be developed, as its failure is based on inter-story drift exceeding a code-specified value (ASCE 2007). For the dual system, however, failure could occur either in the CBF part or in the SC-CBF part. Therefore, probabilistic demand models of inter-story drifts for the CBF and SC-CBF parts must be developed separately for the dual system. For the CBF part of the dual system, the inter-story drift is calculated based on the lateral displacements of the gravity system, as the CBF is rigidly attached to each floor level of the gravity system. In the SC-CBF part of the dual system, the braced frame is no longer rigidly attached to each floor level; therefore, there are two types of inter-story drifts: (1) the inter-story drift that is calculated based on the lateral displacements of the gravity system; and (2) the inter-story frame displacement of the SC-CBF members that is calculated by subtracting the displacement due to rigid-body rotation of the braced frame from the gravity system inter-story drift. The failure mode for the gravity system inter-story drift of the SC-CBF part of the dual system is checked against moment resisting frame (MRF) criteria due to the ductility of the system. The inter-story frame displacement (i.e., the second type of inter-story drift listed above) for the SC-CBF part is checked against the CBF criteria, as the braced frame itself is less ductile. In this study, the probability of exceedance of the inter-story frame displacement of the SC-CBF part of the dual system was consistently less than that of the inter-story drift of the CBF part. Therefore, for the dual system, only two probabilistic demand models are presented, one for the gravity system inter-story drift in the CBF part and the other for the gravity system inter-story drift in the SC-CBF part.

Furthermore, it should be pointed out that there are two separate demand models are presented for the interstory drift for structural elements and non-structural elements in the dual system. This is due to the fact that non-structural elements are affected by pure interstory drift, regardless of the lateral load bearing systems (CBF or SC-CBF). However, for the structural elements (braces, beams and columns), it is important to know if the maximum interstory drift occurs in the CBF part of the building or in the SC-CBF part of the building since there are different capacities for those lateral load systems, as described in detail in Chapter 6.

Statistics of the model coefficients (from Eqs. 5.4, 5.5, and 5.6) are indicated in Tables 5.2 to 5.9 for the six- and ten-story prototype structures for the interstory drift, floor acceleration and roof acceleration demands. Note that PSA is used in calculating the interstory drift deterministic demand; thus, two IMs (PSA and PGV) are involved in the developed demand models for interstory drift and two IMs (PSA and PGA) are involved in the developed demand models for floor acceleration and roof acceleration.

Figure 5.2 plots the predicted values of inter-story drift versus the results obtained from FEM for the CBF system and the SC-CBF part of the 6-story dual systems, respectively. The solid lines in these figures have slopes equal to one (the 1:1 line) and the dashed lines are one standard deviation of model error above and below the solid line. For a perfect prediction, the data would lie along the solid line. As shown in Figures 5.2, the data points are evenly scattered about the solid line, with most of the data within two dashed lines, which indicates that the developed probabilistic demand models provide unbiased and accurate predictions. Figure 5.2 shows that the spread for dual systems D3-

3 and D4-2 are similar to that for the CBF, indicating that the model errors (σ) in the CBF and in the indicated dual system demand models are approximately equal. This is consistent with the data (σ values for the CBF and the SC-CBF part of D3-3 and D4-2) presented in Table 5.2. The spread value is less for D2-4 in comparison with that of the CBF, as shown in Figure 5.2(b). This is consistent with a smaller value of σ in D2-4 in compare with the CBF and dual systems D3-3 and D4-2 (Table 5.2).

Table 5.1 Candidate explanatory functions of normalized intensity measures

IM description	Normalized formula
Elastic pseudo spectral acceleration ^a , PSA	PSA/g
Elastic pseudo spectral acceleration ^b , PSA(2T _l)	PSA(2T)/g
PSA(2T)/ PSA	PSA(2T)/ PSA
Elastic pseudo spectral velocity ^a , PSV	PSV · T _l /H _c
Elastic pseudo spectral displacement ^a , PSD	PSD/H _c
Peak ground acceleration, PGA	PGA/g = max $\ddot{u}_g(t)$ /g
Peak ground velocity, PGV	PGV · T _l /H _c = max $\dot{u}_g(t)$ · T _l /H _c
Peak ground displacement, PGD	PGD/H _c = max $\dot{u}_g(t)$ /H _c
Cumulative absolute velocity, CAV	CAV · T _l /H _c = [$\int_0^D \ddot{u}_g(t) dt$] · T _l /H _c
Cumulative absolute displacement, CAD	CAD/H _c = [$\int_0^D \dot{u}_g(t) dt$] /H _c
Arias intensity, I _A	$I_A \cdot T_l / H_c = (\pi/2g) \int_0^D [\ddot{u}_g(t)]^2 dt \cdot T_l / H_c$
Velocity intensity, I _v	$I_v / H_c = 1 / \dot{u}_{g, \max} \cdot \int_0^D [\dot{u}_g(t)]^2 dt / H_c$
Root mean square acceleration, A _{rms}	$A_{rms} / g = 1 / T_D \cdot \int_0^{T_D} [\ddot{u}_g(t)]^2 dt / g$
Characteristic intensity, I _c	$I_c / (g^{1.5} \cdot T_l) = A_{rms}^{1.5} T_D / (g^{1.5} \cdot T_l)$

^a At the first mode period with 5% damping^b At twice first mode period with 5% damping

Table 5.2 Statistics of the model parameters in the interstory drift demand models for structural elements in six-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient				
				θ_{ID_0}	$\theta_{\dot{d}}$	θ_{ID_1}		
CBF	θ_{ID_0}	0.606	0.142	1.00	0.32	0.08		
	$\theta_{\dot{d}}$	0.356	0.068	0.32	1.00	-0.91		
	θ_{ID_1}	0.775	0.078	0.08	-0.91	1.00		
	σ_{ID}	0.342	-	-	-	-		
D2-4	SC-CBF part	θ_{ID_0}	-0.286	0.127	1.00	0.41	0.02	
		$\theta_{\dot{d}}$	0.481	0.054	0.41	1.00	-0.90	
		θ_{ID_1}	0.554	0.060	0.02	-0.90	1.00	
		σ_{ID}	0.278	-	-	-	-	
	CBF part	θ_{ID_0}	0.107	0.121	1.00	0.23	0.20	
		$\theta_{\dot{d}}$	0.507	0.052	0.23	1.00	-0.91	
		θ_{ID_1}	0.508	0.057	0.20	-0.91	1.00	
		σ_{ID}	0.266	-	-	-	-	
	D3-3	SC-CBF part	θ_{ID_0}	0.851	0.142	1.00	0.28	0.13
			$\theta_{\dot{d}}$	0.503	0.065	0.28	1.00	-0.91
			θ_{ID_1}	0.715	0.071	0.13	-0.91	1.00
			σ_{ID}	0.323	-	-	-	-
CBF part		θ_{ID_0}	-2.780	0.096	1.00	0.23	0.20	
		$\theta_{\dot{d}}$	0.330	0.044	0.23	1.00	-0.91	
		θ_{ID_1}	0.215	0.048	0.20	-0.91	1.00	
		σ_{ID}	0.217	-	-	-	-	
D4-2		SC-CBF part	θ_{ID_0}	0.268	0.163	1.00	-0.41	0.80
			$\theta_{\dot{d}}$	0.141	0.040	-0.41	1.00	-0.87
			θ_{ID_1}	0.953	0.066	0.80	-0.87	1.00
			σ_{ID}	0.348	-	-	-	-
	CBF part	θ_{ID_0}	-3.14	0.103	1.00	-0.23	0.74	
		$\theta_{\dot{d}}$	0.141	0.025	-0.23	1.00	-0.82	
		θ_{ID_1}	0.389	0.042	0.74	-0.82	1.00	
		σ_{ID}	0.221	-	-	-	-	

Table 5.3 Statistics of the model parameters in the interstory drift demand models for structural elements in ten-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient			
				θ_{ID_0}	$\theta_{\dot{d}}$	θ_{ID_1}	
CBF	θ_{ID_0}	0.329	0.098	1.00	0.38	-0.015	
	$\theta_{\dot{d}}$	0.589	0.049	0.38	1.00	-0.93	
	θ_{ID_1}	0.483	0.055	-0.015	-0.93	1.00	
	σ_{ID}	0.225	-	-	-	-	
D4-6	θ_{ID_0}	1.211	0.189	1.00	0.26	0.12	
	SC-CBF part	$\theta_{\dot{d}}$	0.593	0.096	0.26	1.00	-0.93
		θ_{ID_1}	0.710	0.107	0.12	-0.93	1.00
	σ_{ID}	0.439	-	-	-	-	
	CBF part	θ_{ID_0}	-1.706	0.161	1.00	0.26	0.12
		$\theta_{\dot{d}}$	0.237	0.082	0.26	1.00	-0.93
		θ_{ID_1}	0.485	0.091	0.12	-0.93	1.00
		σ_{ID}	0.373	-	-	-	-
	D5-5	θ_{ID_0}	0.497	0.143	1.00	0.03	0.34
		SC-CBF part	$\theta_{\dot{d}}$	0.475	0.074	0.03	1.00
θ_{ID_1}			0.723	0.083	0.34	-0.93	1.00
σ_{ID}			0.342	-	-	-	-
CBF part		θ_{ID_0}	-2.208	0.093	1.00	0.03	0.34
		$\theta_{\dot{d}}$	0.352	0.048	0.03	1.00	-0.93
		θ_{ID_1}	0.284	0.054	0.34	-0.93	1.00
		σ_{ID}	0.221	-	-	-	-
D7-3		θ_{ID_0}	0.374	0.158	1.00	0.41	-0.041
		SC-CBF part	$\theta_{\dot{d}}$	0.329	0.075	0.41	1.00
	θ_{ID_1}		0.780	0.083	-0.041	-0.93	1.00
	σ_{ID}		0.345	-	-	-	-
	CBF part	θ_{ID_0}	-2.65	0.102	1.00	0.41	-0.041
		$\theta_{\dot{d}}$	0.210	0.048	0.41	1.00	-0.93
		θ_{ID_1}	0.400	0.053	-0.041	-0.93	1.00
		σ_{ID}	0.221	-	-	-	-

Table 5.4 Statistics of the model parameters in the interstory drift demand models for non-structural elements in six-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient		
				θ_{ID_0}	$\theta_{\dot{d}}$	θ_{ID_1}
CBF	θ_{ID_0}	0.606	0.142	1.00	0.32	0.08
	$\theta_{\dot{d}}$	0.356	0.068	0.32	1.00	-0.91
	θ_{ID_1}	0.775	0.078	0.08	-0.91	1.00
	σ_{ID}	0.342	-	-	-	-
D2-4	θ_{ID_0}	0.128	0.121	1.00	0.23	0.20
	$\theta_{\dot{d}}$	0.510	0.052	0.23	1.00	-0.91
	θ_{ID_1}	0.508	0.057	0.20	-0.91	1.00
	σ_{ID}	0.266	-	-	-	-
D3-3	θ_{ID_0}	0.508	0.144	1.00	0.23	0.20
	$\theta_{\dot{d}}$	0.455	0.066	0.23	1.00	-0.91
	θ_{ID_1}	0.677	0.072	0.20	-0.91	1.00
	σ_{ID}	0.326	-	-	-	-
D4-2	θ_{ID_0}	0.256	0.163	1.00	-0.23	0.74
	$\theta_{\dot{d}}$	0.141	0.040	-0.23	1.00	-0.82
	θ_{ID_1}	0.950	0.066	0.74	-0.82	1.00
	σ_{ID}	0.348	-	-	-	-

Table 5.5 Statistics of the model parameters in the interstory drift demand models for non-structural elements in ten-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient		
				θ_{ID_0}	$\theta_{\dot{d}}$	θ_{ID_1}
CBF	θ_{ID_0}	0.329	0.098	1.00	0.38	-0.015
	$\theta_{\dot{d}}$	0.589	0.049	0.38	1.00	-0.93
	θ_{ID_1}	0.483	0.055	-0.015	-0.93	1.00
	σ_{ID}	0.225	-	-	-	-
D4-6	θ_{ID_0}	0.062	0.149	1.00	0.26	0.12
	$\theta_{\dot{d}}$	0.363	0.075	0.26	1.00	-0.93
	θ_{ID_1}	0.646	0.084	0.12	-0.93	1.00
	σ_{ID}	0.345	-	-	-	-
D5-5	θ_{ID_0}	-0.200	0.141	1.00	0.04	0.34
	$\theta_{\dot{d}}$	0.297	0.073	0.04	1.00	-0.92
	θ_{ID_1}	0.715	0.082	0.34	-0.92	1.00
	σ_{ID}	0.377	-	-	-	-
D7-3	θ_{ID_0}	0.302	0.158	1.00	0.41	-0.04
	$\theta_{\dot{d}}$	0.292	0.075	0.41	1.00	-0.92
	θ_{ID_1}	0.807	0.083	-0.04	-0.92	1.00
	σ_{ID}	0.345	-	-	-	-

Table 5.6 Statistics of the model parameters in the floor acceleration demand models for non-structural elements in six-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient		
				θ_{A_0}	θ_{A_1}	θ_{A_2}
CBF	θ_{A_0}	0.273	0.028	1.00	-0.27	0.67
	θ_{A_1}	0.108	0.028	-0.27	1.00	-0.78
	θ_{A_2}	0.566	0.040	0.67	-0.78	1.00
	σ_A	0.212	-	-	-	-
D3-3	θ_{A_0}	0.548	0.030	1.00	-0.43	0.76
	θ_{A_1}	0.148	0.031	-0.43	1.00	-0.80
	θ_{A_2}	0.443	0.042	0.76	-0.80	1.00
	σ_A	0.207	-	-	-	-
D4-2	θ_{A_0}	0.540	0.030	1.00	-0.48	0.78
	θ_{A_1}	0.161	0.031	-0.48	1.00	-0.81
	θ_{A_2}	0.539	0.042	0.78	-0.81	1.00
	σ_A	0.197	-	-	-	-

Table 5.7 Statistics of the model parameters in the floor acceleration demand models for non-structural elements in ten-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient		
				θ_{A_0}	θ_{A_1}	θ_{A_2}
CBF	θ_{A_0}	0.269	0.022	1.00	0.07	0.43
	θ_{A_1}	0.200	0.022	0.07	1.00	-0.76
	θ_{A_2}	0.631	0.031	0.43	-0.76	1.00
	σ_A	0.171	-	-	-	-
D5-5	θ_{A_0}	0.569	0.024	1.00	0.00	0.50
	θ_{A_1}	0.067	0.023	0.00	1.00	-0.75
	θ_{A_2}	0.627	0.033	0.50	-0.75	1.00
	σ_A	0.180	-	-	-	-
D7-3	θ_{A_0}	0.583	0.023	1.00	0.00	0.50
	θ_{A_1}	0.060	0.022	0.00	1.00	-0.76
	θ_{A_2}	0.678	0.032	0.50	-0.76	1.00
	σ_A	0.221	-	-	-	-

Table 5.8 Statistics of the model parameters in the roof acceleration demand models for non-structural elements in six-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient		
				θ_{RA_0}	θ_{RA_1}	θ_{RA_2}
CBF	θ_{RA_0}	0.122	0.028	1.00	-0.27	0.67
	θ_{RA_1}	0.093	0.028	-0.27	1.00	-0.78
	θ_{RA_2}	0.486	0.040	0.67	-0.78	1.00
	σ_{RA}	0.214	-	-	-	-
D3-3	θ_{RA_0}	0.492	0.030	1.00	-0.43	0.76
	θ_{RA_1}	0.187	0.030	-0.43	1.00	-0.80
	θ_{RA_2}	0.379	0.041	0.76	-0.80	1.00
	σ_{RA}	0.202	-	-	-	-
D4-2	θ_{RA_0}	0.49	0.030	1.00	-0.48	0.78
	θ_{RA_1}	0.179	0.031	-0.48	1.00	-0.81
	θ_{RA_2}	0.495	0.041	0.78	-0.81	1.00
	σ_{RA}	0.196	-	-	-	-

Table 5.9 Statistics of the model parameters in the roof acceleration demand models for non-structural elements in ten-story buildings

Frame Type	Parameter	Mean	Standard deviation	Correlation coefficient		
				θ_{RA_0}	θ_{RA_1}	θ_{RA_2}
CBF	θ_{RA_0}	0.130	0.029	1.00	0.07	0.43
	θ_{RA_1}	0.268	0.029	0.07	1.00	-0.76
	θ_{RA_2}	0.491	0.042	0.43	-0.76	1.00
	σ_{RA}	0.231	-	-	-	-
D5-5	θ_{RA_0}	0.461	0.029	1.00	0.00	0.50
	θ_{RA_1}	0.190	0.028	0.00	1.00	-0.75
	θ_{RA_2}	0.518	0.041	0.50	-0.75	1.00
	σ_{RA}	0.221	-	-	-	-
D7-3	θ_{RA_0}	0.541	0.029	1.00	0.00	0.50
	θ_{RA_1}	0.157	0.028	0.00	1.00	-0.76
	θ_{RA_2}	0.615	0.041	0.750	-0.76	1.00
	σ_{RA}	0.221	-	-	-	-

Figure 5.1 Computed demand using FEM vs. deterministic demand for (a) CBF, (b) D4-2 system, (c) D3-3 system, and (d) D2-4 system

Figure 5.2 Computed demand using FEM vs. prediction from proposed probabilistic inter-story demand models for (a) CBF, (b) D4-2 system, (c) D3-3 system and (d) D2-4 system

CHAPTER VI

SEISMIC PERFORMANCE EVALUATION OF THE DUAL SYSTEM

Seismic performance of structures can be evaluated using a probabilistic approach that provides stakeholders with information about the building (usually expressed in probabilistic terms) that facilitates decision making for risk management. Risk and its measurement are critical concepts for dealing with uncertainties in decision making. Another important concept associated with risk is hazard. In general, hazard refers to the probability of exceeding an intensity of ground motion in a specific time (e.g., one year). The relation between risk and hazard can simply expressed as follows:

$$Risk = Hazard \times Vulnerability \quad (6.1)$$

In this study, vulnerability of structures is quantified using fragility analysis as described in this chapter. On the other hand, seismic hazard, which can be expressed as the mean annual frequency of exceeding specific IMs, can be quantified using the probability distribution functions for the selected IMs, as described later. Finally, risk describes a probability of failure if structures are exposed to a seismic hazard over a certain time at a specific location. In this study, a period of one year is considered as the exposure time, and risk is expressed as the annual probability of failure. Annual probability of failure is calculated for both dual systems and CBF systems to evaluate and

compare the seismic performance of the dual system with that of the CBF system. To fully demonstrate the effectiveness of the dual systems over conventional CBF systems, it is necessary to consider the seismic performance of structural and non-structural components in both systems. Since it is not computationally plausible to evaluate fragility curves of all non-structural components, four common non-structural components are considered in this study. These components are: (1) exterior walls and glass blocks, (2) interior walls (heavy partitions), (3) a non-vibration-isolated heating, ventilation, and air conditioning (HVAC) system, and (4) fire suppression piping.

In this chapter, first, seismic fragility analysis is introduced, using interstory drift and floor and roof acceleration as EDPs. Then seismic hazard analyses are executed for the CBF and dual systems for three different locations, and annual probability of failure is calculated for the 6-story and 10-story buildings. In considering this, the values presented by ACSE-41 (2007) are adopted for the capacities for inter-story drift, floor acceleration, and roof acceleration, as described in this chapter.

6.1 Seismic Fragility Assessment

Compared with traditional reliability indices, fragility curves provide richer and more comprehensive perspective on system reliability because they convey more information about the reliability of the system. Although they are closely related to the relative reliability index (RRI), fragility curves and RRIs differ in several respects. In particular: (1) fragility curves are functions rather than point estimates, (2) the loads (or intensity measures) in fragility curves are treated deterministically, so fragility curves

express a probability of failure that is conditional on the load rather than an overall probability of failure, and (3) the probabilities from fragility curves are generally interpreted as absolute probabilities rather than nominal probabilities, implying knowledge of the underlying probability distributions (USACE 2010). In this study, seismic fragilities will be assessed based on the proposed probabilistic demand models for the CBF and dual systems and will be used for the seismic performance comparison of the studied systems.

Seismic fragility is defined as the conditional probability that a demand quantity attains or exceeds a specified capacity level for given values of earthquake IMs and can be formulated as follows:

$$F(\mathbf{s}) = P[g(\mathbf{x}, \Theta) \leq 0 | \mathbf{s}] = P[C - D(\mathbf{x}, \Theta) \leq 0 | \mathbf{s}] \quad (6.2)$$

where $g(\mathbf{x}, \Theta)$ is the limit-state function of the failure mode, $\mathbf{s}$ is the vector of normalized IMs, C is the capacity of the desired failure mode, and $D(\mathbf{x}, \Theta)$ is the probabilistic demand (for the EDPs used in this study: interstory drift, floor acceleration, and roof acceleration) calculated from Equations 5.4 to 5.6. In this study, Eq. 6.2 is calculated for both structural and non-structural components of the 6-story and 10-story dual and CBF systems. For non-structural components, fragility is evaluated separately for drift sensitive and acceleration sensitive elements. For each case, proper capacity values should be considered from ASCE-41 (2007) or another appropriate code. The inter-story drift capacity for structural components is taken from the code recommendations (ASCE-41 2007) for the IO, LS, and CP performance levels. Since the dual system is comprised

of an SC-CBF part and a CBF part, specific provisions should be considered in selecting the interstory drift capacities for the dual system, as described later. This is mostly due to the fact that the SC-CBF is a recently developed structural system and no criteria yet have been incorporated into the code.

The capacities for non-structural components are also obtained from ASCE-41 (2007), in which non-structural components are categorized as drift sensitive (i.e., exterior walls and heavy partitions) and acceleration sensitive (e.g., HVAC and fire suppression systems) components. The capacities for non-structural elements are defined based on to previous work by Dyanati et al. (2014) and are presented in Table 6.1. Fragility curves can be constructed using Eq. 6.2 based on the seismic demands calculated from Eqs. 5.4, 5.5, and 5.6, and capacities are based on the code provisions. Fragility curves in this study are constructed using the First Order Reliability Method (FORM). FORM is called a first-order method because the limit state function (g) is a linear (first-order) function of the capacity and demand variables.

For the dual system, structural component failures due to inter-story drift can occur either in the CBF part or in the SC-CBF part, as previously mentioned. The inter-story drift capacity for the braced frame system, specified in ASCE-41 (2007), is used for the CBF part and the MRF inter-story drift capacity is used for the SC-CBF part due to its high drift capacity (Table 6.2). The probability of failure of the dual system can be assessed as follows:

$$P(E_1 \cup E_2) = P(E_1) + P(E_2) - P(E_1 E_2) \quad (6.3)$$

where $P(E_1 \cup E_2)$ refer as the probability of failure for the dual system, E_1 refers to the failure in the CBF part, E_2 refers to the failure in the SC-CBF part, $P(E_1)$ is the probability of failure in the CBF part, $P(E_2)$ is the probability of failure in the SC-CBF part, and $P(E_1 E_2)$ refers to the joint probability of failure of the CBF part and the SC-CBF part.

To calculate the joint probability of failure, the cumulative probability of the multivariate normal distribution is used as follows:

$$P(E_1 E_2) = \text{MVNCDF}([0; 0], [\mu(E_1); \mu(E_2)], \text{SIGMA}) \quad (6.4)$$

where $\mu(E_1)$ and $\mu(E_2)$ are the mean values of E_1 and E_2 , respectively; MVNCDF is the multivariate normal cumulative distribution function; and SIGMA is the covariance matrix as follows:

$$\text{SIGMA} = \begin{bmatrix} \text{Var}(E_1) & \text{COV}(E_1, E_2) \\ \text{COV}(E_1, E_2) & \text{Var}(E_2) \end{bmatrix} \quad (6.5)$$

where $\text{VAR}(E_1)$ and $\text{VAR}(E_2)$ are the variance of E_1 and E_2 , respectively, and $\text{COV}(E_1, E_2)$ is the covariance between E_1 and E_2 . To determine the actual relationships between these variables, a reasonable number of samples for E_1 and E_2 are required. To create such samples, the bootstrapping technique is used. Bootstrap sampling for regression can be done as follows: when the regression function being fitted is a good model for the data, the error terms have constant variance, and the predictor variables can be regarded as fixed; therefore, fixed X sampling is appropriate. Here, the residuals e_i

from the original fitting are regarded as the sample data to be sampled with replacement. After a bootstrap sample of residuals of size n has been obtained, denoted by $e_1^*, \dots, e_n^*$, the bootstrap sample residuals are added to the fitted values from the original fitting to obtain new bootstrap Y values, denoted by $Y_1^*, \dots, Y_n^*$:

$$Y_i^* = Y_i + e_i^* \quad (6.6)$$

These bootstrap Y^* values are then regressed on the original X variables by the same procedure used initially to obtain the bootstrap estimate b_1^* . The number of bootstrap samples to be selected for evaluating the precision of an estimate depends on the special circumstances of each application. Sometimes, as few as 50 bootstrap samples are sufficient. Often, 200-500 bootstrap samples are adequate (Kutner et al. 2004). In this study, 500 bootstrap samples were used.

In general, the uncertainty in the fragility estimate comes from the variability in the structural properties, model parameters, and model error. In this study, the considered uncertainties are in the elastic modulus of the structural members, the yield stress of the structural members and PT bars, and the PT bar initial force. The assumed statistics of those structural properties are indicated in Table 6.3. Other structural properties (e.g., frame geometry and member sizes) are considered deterministically. In this study, uncertainties in structural variability did not affect fragility curves significantly. The model error and uncertainties in the model parameters in the probabilistic demand model dominate the variability in the limit states, which is consistent with previous studies (e.g., Huang 2010, Choe 2005, Song and Elingwood 1999). According to the aforementioned

discussion, some fragility plots are shown as a sample of the results. Figure 6.1 shows the fragility surfaces of interstory drift (with respect to structural components) for D5-5 for three different performance levels. In Figure 6.1, the horizontal axes show the selected normalized IMs (PSA/g and $PGV \cdot T_1/H_c$) and the vertical axis shows the probability of failure for structural components in D5-5. Figure 6.2 shows the same plots for D7-3.

Figure 6.3 shows the fragility surface of roof acceleration as a sample of fragilities of non-structural elements for D5-5. In Figure 6.3, the horizontal axes show the selected normalized IMs (PSA/g and PGA/g) and the vertical axis shows the probability of failure of HVAC system in two different performance levels (IO and LS). Figure 6.4 shows the similar results for D7-3.

6.2 Seismic Hazard Analysis

Seismic hazard describes a natural phenomenon associated with an earthquake that could cause harm and can be quantified by three parameters: level of severity (e.g., in terms of magnitude M , or PGA), occurrence frequency, and location. In this study, three ground intensity measures were considered to calculate the hazard distributions. These intensity measures, which are selected based on the model selection procedure (as described in Chapter 5), are PSA , PGA , and PGV . Furthermore, three different locations in the United States were considered to calculate the seismic hazard: Los Angeles, Seattle and Boston. In this study, the exposure time is considered to be one year. The distribution of the mean annual frequency of exceeding a given IM (i.e., IM hazard) can be estimated

using a type II extreme value distribution (i.e., Frechet distribution) (Lin et al. 2010, Song and Elingwood 1999) as follows:

$$\lambda(s) = 1 - \exp\left[-(s / \mu)^{-k}\right] \quad (6.7)$$

where s is the IM of interest, $\lambda(s)$ is the mean annual rate of exceeding selected IM ($\lambda(s) = p(IM > s)$), and μ and k are the location and slope of the distribution, respectively.

Parameters μ and k for each IM in each location can be evaluated using the mean values of each IM for 2% and 10% exceedance in 50 years (Lin et al. 2010, Song and Elingwood 1999). In this study, these two mean values are obtained from SAC earthquake records that have been used in the numerical analysis and the USGS hazard application (USGS 2014 database) with hazard levels corresponding to 2% and 10% probabilities of exceedance in 50 years for the three locations. Note that Los Angeles is the site for the designed prototype structures. Tables 6.4 to 6.9 indicate μ and k values for each normalized IM (PSA, PGA and PGV) in the three different locations for the six-story and ten-story prototype structures.

When two IMs are incorporated in the hazard analysis, the joint hazard distribution of the IMs should be used. The joint hazard distribution of the IMs is evaluated here using an m -type bivariate joint extreme value distribution for Frechet marginal distributions (Elshamy 1992, Gumbel and Mustafi 1967).

$$\lambda(s_1, s_2) = 1 - \exp\left\{-\left[\left(s_1 / \mu_1\right)^{-m \cdot k_1} + \left(s_2 / \mu_2\right)^{-m \cdot k_2}\right]^{1/m}\right\} \quad (6.8)$$

$$m = (1 - \rho)^{-1/2} \quad (6.9)$$

where ρ is the correlation between the two IMs (PSA and PGV), μ_1 and k_1 are the distribution parameters of s_1 , and μ_2 and k_2 are the distribution parameters of s_2 . In this study, s_1 and s_2 are normalized PGV and PSA ($s_1 = \text{PSA}/g$, $s_2 = \text{PGV} \cdot T_1/H_c$) when the failure due to the interstory drift is considered. When the failure due to the floor acceleration or roof acceleration is considered, then s_1 and s_2 are normalized PGA and PSA ($s_1 = \text{PSA}/g$, $s_2 = \text{PGA}/g$). The correlation factor between the selected IMs (ρ) for the 6-story and 10-story buildings is tabulated in Table 6.10 for 6-story and 10-story buildings. Figures 6.5 and 6.6 show the probability distribution (i.e., $\left(\left| \frac{d\lambda(s_1, s_2)}{ds_1 ds_2} \right| \right)$) for the selected IMs for D5-5. Figure 6.5 shows the joint probability distribution function (PDF) of the normalized IMs ($s_1 = \text{PSA}/g$, $s_2 = \text{PGV} \cdot T_1/H_c$) for Los Angeles, Boston and Seattle. These curves are then used to calculate annual probability of failure in all three locations, as described later in this chapter. Figure 6.6 shows joint PDF of normalized IMs ($s_1 = \text{PSA}/g$, $s_2 = \text{PGA}/g$) for LA. This curve will be used to calculate the annual probability of failure of non-structural elements, as described later.

6.3 Engineering Demand Hazard Analysis

Engineering demand hazard refers to the mean annual frequency of exceeding a specific demand. Engineering demand hazard can be computed as the probability of a probabilistic demand (D) exceeding capacity (C) given intensity measure (s), written as

$P(D > C|s)$, combined with a ground motion hazard curve to compute the mean annual rate of exceeding a given demand value. It can be estimated as follows (Backer 2007):

$$P(D > C) = \int_s G(C|s) \left| \frac{d\lambda(s)}{ds} \right| ds \quad (6.10)$$

where s = IM of interest, C = specific demand value (capacity), D = engineering demand parameter (inter-story drift, roof acceleration, or floor acceleration in this study), and $G(C/s) = P(D > C|s)$ = probability of demand exceeding capacity given s , also known as the complementary cumulative distribution function (CCDF).

When two IMs are used, Equation 6.10 can be written as follows:

$$P(D > C) = \int_{s_2} \int_{s_1} G(C|s_1, s_2) \left| \frac{d\lambda(s_1, s_2)}{ds_1 ds_2} \right| ds_1 ds_2 \quad (6.11)$$

where $\lambda(s_1, s_2)$ is the joint distribution of mean annual exceedance of two IMs (i.e., joint hazard for $s_1 = \text{IM}_1$ and $s_2 = \text{IM}_2$).

In this study, reliability analysis is used to assess $G(C/s) = P(C - D < 0|s)$, in which the probabilistic demand, D , is assessed through Eqs. 5.4, 5.5, and 5.6 for peak inter-story drift, roof acceleration and floor acceleration. To illustrate the comprising terms in Equation 6.11, each term is shown separately in Figure 6.7. Figure 6.7(a) shows the probability of failure for the D5-5 system (as an example) predicted by FORM analysis as a function of PSA/ g and PGV· T_1/H_c (i.e., $G(C/s_1, s_2)$). Figure 6.7(b) shows the second

term in the integrand $\left(\left| \frac{d\lambda(s_1, s_2)}{ds_1 ds_2} \right| \right)$, which is the joint probability distribution function

(PDF) of the two IMs (PSA/g and $PGV \cdot T_1/H_c$). By multiplying these two terms and integrating over the possible range of the IMs, the engineering demand hazard ($P(D>d)$) is calculated as shown in Eq. 6.11.

6.4 Comparative Study of Dual Systems with Different Configurations

In this section, the seismic fragilities of the six studied dual system configurations (D2-4, D3-3, and D4-2 for the six-story dual system and D4-6, D5-5, and D7-3 for the ten-story dual system) are compared with those of comparable CBF systems to evaluate the relative effectiveness of different configurations of the dual system with respect to the seismic performance of the CBF system. Seismic fragility curves are developed for both structural and non-structural components and for three different performance levels. The comparisons of the CBF and the dual system (D2-4, D3-3, D4-2, D4-6, D5-5, and D7-3) fragilities for structural components are shown in Figures 6.8, 6.9 and 6.10 for the six-story structures and Figures 6.11, 6.12 and 6.13 for the ten-story structures, respectively. Small circles on the plots represent the data from the FEM numerical analysis. As the probabilistic demand models used for the fragilities are developed based on the numerical results, those data points provide a reasonable approximation of expected values of IMs.

In the six-story buildings, for given values of PSA/g and $PGV \cdot T_1/H_c$, the structural components of the dual system show less probability of failure for all performance levels for the D3-3 and D4-2 configurations, as shown in Figures 6.9 and 6.10. However, Figure 6.8 shows that the D2-4 configuration does not show significant improvement in performance compared to the CBF system. In the ten-story structure, similar behavior is

found: the dual system with the fewest stories of SC-CBF shows the least improvement over the conventional CBF system. For given values of IMs, the dual system shows less probability of failure for all performance levels for the D5-5 configuration, as shown in the Figure 6.12. Figure 6.13 indicates that the D7-3 configuration shows less probability of failure from the CBF in the most ranges of IMs but not necessarily in all ranges. For the small values of PSA/g , the CBF shows slightly better performance in compare with the dual system. Figure 6.11 indicates that the D4-6 configuration does not show perceptible improvement in performance compared to the CBF, especially for small values of PSA/g . This can be understood intuitively: as fewer SC-CBF stories are introduced to the dual system, less performance improvement is obtained, as the structure is relatively similar to the conventional CBF design.

Figure 6.14 shows the comparison of the fragility curves for the three different six-story dual system configurations. For all performance levels, within the expected values of PSA/g and $PGV \cdot T_I / H_c$, both D3-3 and D4-2 have less probability of failure than D2-4. Furthermore, D4-2 shows less probability of failure than D3-3 for lower values of PSA/g and more probability of failure than D3-3 for higher values of PSA/g . This observation indicates that to compare the performances of D3-3 and D4-2, the IM values must be considered. As IM values vary from one site to another, such performance comparison may result in different dual system configurations being better suited for different sites. To consider the site effects on the probability of failure and consider the probability of occurrence of each IM, seismic hazard curves are incorporated as described later in this chapter.

Figure 6.15 shows the comparison of the fragility curves for the ten-story dual system for three different performance levels. Figures 6.15(b) and 6.15(c) show that for the LS and CP performance levels, within the expected values of IMs, both D5-5 and D7-3 have less probability of failure than D4-6. Moreover, D7-3 shows higher probability of failure than D5-5 for lower values of PSA/g and lower probability of failure than D5-5 for higher values of PSA/g . Figure 6.15(a) shows that in the IO level, D5-5 does not show clear improvement in performance in comparison with D4-6.

The comparison of the seismic fragilities for non-structural components is represented in Figures 6.16 to 6.19. Since there is no CP performance level defined for the non-structural elements in the code, the non-structural fragility curves are developed only for IO and LS performance levels. Figure 6.16 shows fragility curves for drift-sensitive non-structural elements (i.e., exterior walls and heavy partitions) in D3-3 and compare it with those of the 6-story CBF system. The drift-sensitive non-structural components show better performance (i.e., less probability of failure) in D3-3 than in the CBF system for both IO and LS performance levels. Figure 6.17 compares the fragility curves of drift-sensitive non-structural components in D7-3 and the 10-story CBF system. The results indicate that for small values of PSA/g , the CBF shows better performance, while for large values of PSA/g , D7-3 shows less probability of failure.

Figure 6.18 shows the fragility curves of acceleration-sensitive non-structural components (i.e., HVAC and fire suppression system) for the 6-story CBF and D3-3. As shown in this figure, the performance of acceleration sensitive non-structural components

in D3-3 is worse than that in the CBF system. Similar results are found for the ten-story buildings, as shown in Figure 6.19.

Since the fragility curves are not sufficient to consider the site effects and the probability of occurrence of each IM, engineering demand hazard curves (also called performance curves) are developed. The resulting performance curves are shown in Figures 6.20 to 6.27. Figure 6.20 shows the mean annual probability of failure in structural components for 10-story buildings in LA. As shown in this figure, the 10-story CBF structure shows a higher annual probability of failure compared with D5-5 and D7-3 in all three performance levels. The difference becomes more noticeable at the CP performance level. D4-6 shows worse performance (i.e., more annual probability of failure) than the CBF for the IO and LS performance limits. Table 6.11 also shows the exact values of annual probability of failure for 10-story building. It can be seen that D5-5 shows less annual probability of failure than the other dual systems in the LS and CP performance levels. In the IO performance level, both D5-5 and D7-3 show similar annual probabilities of failure (though D7-3 shows slightly better performance in the IO limit state). Figures 6.21 and 6.22 and Tables 6.12 and 6.13 show the similar results for buildings located in BO and SE. These results will be used to calculate the site effect on loss estimation as explained in the next chapter. Figure 6.23 shows similar results for 6-story buildings located in LA. As illustrated in Figure 6.23 and Table 6.14, the CBF system shows worse performance than all configurations of the dual system. D3-3 shows less annual probability of failure than D4-2 and D2-4 in all three performance levels. Since D2-4 in the six-story dual system and D4-6 in the ten-story dual system did not

represent significant improvement in structural performance compared with that in CBF system, those two dual system configurations are not considered economically efficient. Figures 6.24 and 6.25 and Tables 6.15 and 6.16 show the similar results for buildings located in BO and SE.

Figure 6.26 shows the mean annual probability of failure for acceleration-sensitive non-structural components. As shown in Figure 6.26 and Table 6.17, the 10-story CBF shows much less annual probability of failure compared with dual systems. In the IO performance limit, CBF shows less annual probability of failure by two orders of magnitude compared with the dual systems. In LS performance limit, D5-5 and D7-3 show a greater annual probability of failure by one order of magnitude and two orders of magnitude, respectively, compared with the 10-story CBF. Figure 6.27 and Table 6.18 show similar results for 6-story and 10-story buildings. As illustrated in Figure 6.27, the CBF shows a significantly lower annual probability of failure compared with D4-2 and D3-3 systems. Based on these results, it can be concluded that the dual system shows better performance in terms of structural and drift-sensitive non-structural components but the CBF has better performance in terms of acceleration-sensitive non-structural components. At this stage the important question is whether the structural damage advantages of the dual system can overcome its disadvantages in the non-structural damage. To quantitatively address this point, cost analysis for both CBF and dual systems must be implemented, as presented in Chapter 7.

Table 6.1 Nonstructural component capacities based on ASCE41 (2007).

Nonstructural component	Sensitivity quantities	Inter-story drift		Acceleration	
		IO	LS	IO	LS
Exterior walls and glass blocks	D	1.00%	2.00%	0.443g	1.015g
Heavy partitions	D	0.50%	1.00%	0.590g	1.354g
HVAC system (non-vibration isolated)	A	–	–	1.180g	2.707g
Fire suppression piping	A	0.50%	1.50%	2.175g	3.263g

A: Acceleration, D: Deformation

Table 6.2 Interstory drift capacities (ASCE 2007)

Performance level	MRF	CBF
IO	0.7%	0.5%
LS	2.5%	1.5%
CP	5.0%	2.0%

Table 6.3 Assumed statistics of uncertain structural properties

Parameter		Mean value	Distribution	COV (%)
Elastic modulus of brace members		200 GPa	Lognormal	5.00
Yield stress of brace members		345 MPa	Lognormal	10.00
Elastic modulus of PT bar		205 MPa	Lognormal	5.00
Yield stress of PT bar		830 MPa	Lognormal	10.00
Initial PT force	D2-4	5034 KN	Uniform	0.6
	D3-3	5397 KN	Uniform	0.6
	D4-2	5060 KN	Uniform	0.6
Initial PT force	D4-6	8391 KN	Uniform	0.6
	D5-5	7832 KN	Uniform	0.6
	D7-3	6713 KN	Uniform	0.6

Table 6.4 Parameters of IM Hazards for the 6-story frames for LA

Structure	$\lambda(\text{PSA}/g)$		$\lambda(\text{PGA}/g)$		$\lambda(\text{PGV} \cdot T_V/H_c)$	
	μ	k	μ	k	μ	k
CBF	0.0757	2.7286	0.1302	3.089	0.0030	2.7230
D2-4	0.0722	2.3879	0.1302	3.089	0.0024	2.7230
D3-3	0.0755	2.4162	0.1302	3.089	0.0023	2.7230
D4-2	0.1029	2.6959	0.1302	3.089	0.0022	2.7230

Table 6.5 Parameters of IM Hazards for the 6-story frames for SE

Structure	$\lambda(\text{PSA}/g)$		$\lambda(\text{PGA}/g)$		$\lambda(\text{PGV} \cdot T_V/H_c)$	
	μ	k	μ	k	μ	K
CBF	0.1302	4.0942	-	-	0.002977	2.7230
D2-4	0.0253	1.8091	-	-	0.000507	1.9681
D3-3	0.0313	1.9071	-	-	0.000492	1.9681
D4-2	0.0468	2.1092	-	-	0.000462	1.9681

Table 6.6 Parameters of IM Hazards for the 6-story frames for BO

Structure	$\lambda(\text{PSA}/g)$		$\lambda(\text{PGA}/g)$		$\lambda(\text{PGV} \cdot T_V/H_c)$	
	μ	k	μ	k	μ	K
CBF	0.1302	4.0942	-	-	0.002977	2.7230
D2-4	0.0037	1.6424	-	-	0.0000814	1.7355
D3-3	0.0036	1.6424	-	-	0.000079	1.7355
D4-2	0.0038	1.6268	-	-	0.0000743	1.7355

Table 6.7 Parameters of IM Hazards for the 10-story frames for LA

Structure	$\lambda(\text{PSA}/g)$		$\lambda(\text{PGA}/g)$		$\lambda(\text{PGV} \cdot T_v/H_c)$	
	μ	k	μ	k	μ	K
CBF	0.1302	4.0942	0.1302	3.089	0.002977	2.7230
D4-6	0.0762	2.7793	0.1302	3.089	0.002323	2.7230
D5-5	0.0805	2.8220	0.1302	3.089	0.002222	2.7230
D7-3	0.0816	2.8324	0.1302	3.089	0.0022	2.7230

Table 6.8 Parameters of IM Hazards for the 10-story frames for SE

Structure	$\lambda(\text{PSA}/g)$		$\lambda(\text{PGA}/g)$		$\lambda(\text{PGV} \cdot T_v/H_c)$	
	μ	k	μ	k	μ	K
CBF	0.1302	4.0942	-	-	0.002977	2.7230
D4-6	0.0185	1.8701	-	-	0.000485	1.9681
D5-5	0.0198	1.8932	-	-	0.000478	1.9681
D7-3	0.0201	1.8996	-	-	0.000476	1.9681

Table 6.9 Parameters of IM Hazards for the 10-story frames for BO

Structure	$\lambda(\text{PSA}/g)$		$\lambda(\text{PGA}/g)$		$\lambda(\text{PGV} \cdot T_v/H_c)$	
	μ	k	μ	k	μ	K
CBF	0.1302	4.0942	-	-	0.002977	2.7230
D4-6	0.0019	1.6621	-	-	0.0000779	1.7355
D5-5	0.0020	1.6589	-	-	0.0000767	1.7355
D7-3	0.0020	1.6584	-	-	0.0000764	1.7355

Table 6.10 Correlation coefficient (ρ) between the selected normalized IMs in LA

Frame Type	6-Story				Frame Type	10-Story			
	IMs	PSA	PGA	PGV		IMs	PSA	PGA	PGV
CBF	PSA	1.00	0.69	0.71	CBF	PSA	1.00	0.81	0.76
	PGA	0.69	1.00	N/A		PGA	0.81	1.00	N/A
	PGV	0.71	N/A	1.00		PGV	0.76	N/A	1.00
D2-4	PSA	1.00	0.78	0.74	D4-6	PSA	1.00	0.73	0.78
	PGA	0.78	1.00	N/A		PGA	0.73	1.00	N/A
	PGV	0.74	N/A	1.00		PGV	0.78	N/A	1.00
D3-3	PSA	1.00	0.80	0.75	D5-5	PSA	1.00	0.70	0.77
	PGA	0.80	1.00	N/A		PGA	0.70	1.00	N/A
	PGV	0.75	N/A	1.00		PGV	0.77	N/A	1.00
D4-2	PSA	1.00	0.82	0.76	D7-3	PSA	1.00	0.69	0.78
	PGA	0.82	1.00	N/A		PGA	0.69	1.00	N/A
	PGV	0.76	N/A	1.00		PGV	0.78	N/A	1.00

Table 6.11 Mean annual probability of failure (structural interstory drift) for 10-story building

Performance Level	Type of structure			
	CBF	D4-6	D5-5	D7-3
Immediate Occupancy	2.918×10^{-2}	6.032×10^{-2}	2.352×10^{-2}	2.073×10^{-2}
Life Safety	2.048×10^{-3}	1.438×10^{-3}	6.678×10^{-4}	2.943×10^{-3}
Collapse Prevention	1.067×10^{-3}	4.482×10^{-4}	1.546×10^{-4}	1.28×10^{-4}

Table 6.12 Mean annual probability of failure (structural interstory drift) for 10-story building in BO

Performance Level	Type of structure			
	CBF	D4-6	D5-5	D7-3
Immediate Occupancy	2.569×10^{-4}	3.705×10^{-4}	2.350×10^{-4}	1.877×10^{-4}
Life Safety	4.248×10^{-5}	3.145×10^{-5}	2.481×10^{-5}	5.303×10^{-5}
Collapse Prevention	2.917×10^{-5}	1.423×10^{-5}	1.032×10^{-5}	7.661×10^{-6}

Table 6.13 Mean annual probability of failure (structural interstory drift) for 10-story building in SE

Performance Level	Type of structure			
	CBF	D4-6	D5-5	D7-3
Immediate Occupancy	4.611×10^{-3}	6.618×10^{-3}	4.282×10^{-3}	2.793×10^{-3}
Life Safety	6.645×10^{-4}	4.091×10^{-4}	2.949×10^{-4}	6.502×10^{-4}
Collapse Prevention	4.269×10^{-4}	1.681×10^{-4}	1.154×10^{-4}	7.265×10^{-5}

Table 6.14 Mean annual probability of failure (structural interstory drift) for 6-story building in LA

Performance Level	Type of structure			
	CBF	D2-4	D3-3	D4-2
Immediate Occupancy	7.244×10^{-2}	2.716×10^{-2}	6.539×10^{-3}	2.145×10^{-2}
Life Safety	5.047×10^{-3}	1.293×10^{-3}	1.909×10^{-4}	4.213×10^{-4}
Collapse Prevention	2.394×10^{-3}	6.392×10^{-4}	6.083×10^{-5}	7.763×10^{-5}

Table 6.15 Mean annual probability of failure (structural interstory drift) for 6-story building in BO

Performance Level	Type of structure			
	CBF	D2-4	D3-3	D4-2
Immediate Occupancy	5.448×10^{-4}	2.285×10^{-4}	1.43×10^{-4}	3.95×10^{-4}
Life Safety	9.797×10^{-5}	2.448×10^{-5}	2.749×10^{-5}	5.611×10^{-5}
Collapse Prevention	5.947×10^{-5}	8.172×10^{-6}	1.505×10^{-5}	3.814×10^{-5}

Table 6.16 Mean annual probability of failure (structural interstory drift) for 6-story building in SE

Performance Level	Type of structure			
	CBF	D2-4	D3-3	D4-2
Immediate Occupancy	9.441×10^{-3}	4.911×10^{-3}	1.735×10^{-3}	2.84×10^{-3}
Life Safety	1.26×10^{-3}	5.488×10^{-4}	2.553×10^{-4}	2.329×10^{-4}
Collapse Prevention	7.248×10^{-4}	3.613×10^{-4}	1.318×10^{-4}	6.518×10^{-5}

Table 6.17 Mean annual probability of failure (non-structural roof acceleration) for 10-story building

Performance Level	Type of structure			
	CBF	D4-6	D5-5	D7-3
Immediate Occupancy	3.788×10^{-4}	-	2.407×10^{-2}	3.655×10^{-2}
Life Safety	1.396×10^{-5}	-	7.306×10^{-4}	1.446×10^{-3}

Table 6.18 Mean annual probability of failure (non-structural roof acceleration) for 6-story building

Performance Level	Type of structure			
	CBF	D2-4	D3-3	D4-2
Immediate Occupancy	3.358×10^{-4}	-	4.954×10^{-2}	4.557×10^{-2}
Life Safety	4.419×10^{-6}	-	6.437×10^{-4}	1.123×10^{-3}

Figure 6.1 Fragility surface of interstory drift in D5-5 system: (a) IO, (b) LS, and (c) CP

Figure 6.2 Fragility surface of interstory drift in D7-3 system: (a) IO, (b) LS, and (C) CP

Figure 6.3 Fragility surface of roof acceleration (HVAC) in D5-5: (a) IO and (b) LS

Figure 6.4 Fragility surface of roof acceleration (HVAC) in D7-3: (a) IO and (b) LS

(a)

(b)

(c)

Figure 6.5. Joint PDF of IMs (PSA/g and $PGV \cdot T_n / h_c$) for D5-5 in (a) LA, (b) BO, and (c) SE

Figure 6.6. Joint PDF of IMs (PSA/g and PGA/g) for D5-5 in LA

Figure 6.7 (a) Fragility surface of inter-story drift in D5-5 system for 0.5% inter-story drift capacity and (b) joint PDF of IMs (Sa/g and $PGV \cdot T_n/h_c$)

Figure 6.8 Fragility curves of inter-story drift in CBF (solid blue lines) and D2-4 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance

Figure 6.9 Fragility curves of inter-story drift in CBF (solid blue lines) and D3-3 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance

Figure 6.10 Fragility curves of inter-story drift in CBF (solid blue lines) and D4-2 (dashed red lines) for (a) IO (b) LS (c) CP levels of performance

Figure 6.11 Fragility curves of inter-story drift in CBF (solid blue lines) and D4-6 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance

Figure 6.12 Fragility curves of inter-story drift in CBF (solid blue lines) and D5-5 (dashed red lines) for (a) IO, (b) LS, and (c) CP levels of performance

Figure 6.13 Fragility curves of inter-story drift in CBF (solid blue lines) and D7-3 (dashed red lines) for (a) IO (b) LS (c) CP levels of performance

Figure 6.14 Fragility curves of inter-story drift in dual system with different configuration for (a) IO (b) LS (c) CP levels of performance

Figure 6.15 Fragility curves of inter-story drift in dual system with different configuration for (a) IO (b) LS (c) CP levels of performance

Figure 6.16 Fragility curves of drift-sensitive non-structural components in CBF (solid blue lines) and D3-3 (dashed red lines) for (a) Exterior wall - IO (b) Exterior wall - LS (c) Partitions – IO (d) Partitions – LS

Figure 6.17 Fragility curves of drift-sensitive non-structural components in CBF (solid blue lines) and D7-3 (dashed red lines) for (a) Exterior wall - IO (b) Exterior wall - LS (c) Partitions – IO (d) Partitions – LS

Figure 6.18 Fragility curves of acceleration-sensitive non-structural components in CBF (solid blue lines) and D3-3 (dashed red lines) for (a) HVAC - IO (b) HVAC - LS (c) Fire suppression - IO (d) Fire suppression - LS

Figure 6.19 Fragility curves of acceleration-sensitive non-structural components in CBF (solid blue lines) and D7-3 (dashed red lines) for (a) HVAC - IO (b) HVAC - LS (c) Fire suppression – IO (d) Fire suppression - LS

Figure 6.20 Mean annual probability of failure (structural interstory drift) for 10-story CBF and 10-story Dual systems with different configurations in LA

Figure 6.21 Mean annual probability of failure (structural interstory drift) for 10-story CBF and 10-story Dual system with different configurations in BO

Figure 6.22 Mean annual probability of failure (structural interstory drift) for 10-story CBF and 10-story Dual system with different configurations in SE

Figure 6.23 Mean annual probability of failure (structural interstory drift) for 6-story CBF and 6-story Dual system with different configurations in LA

Figure 6.24 Mean annual probability of failure (structural interstory drift) for 6-story CBF and 6-story Dual system with different configurations in BO

Figure 6.25 Mean annual probability of failure (structural interstory drift) for 6-story CBF and 6-story Dual system with different configurations in SE

Figure 6.26 Mean annual probability of failure (non-structural roof acceleration) for 10-story CBF and 10-story Dual system with different configurations

Figure 6.27 Mean annual probability of failure (non-structural roof acceleration) for 6-story CBF and 6-story Dual system with different configurations

CHAPTER VII

PROBABILISTIC FRAMEWORK FOR STRUCTURAL AND NON-STRUCTURAL DAMAGE ASSESSMENT

7.1 Introduction

As discussed in Chapter 6, dual systems, in general, show better seismic performance in comparison with CBF systems. However, the construction cost of the dual system is expected to be higher than that of the conventional CBF due to the precision required in fabrication and erection of the SC-CBF part of the system. Therefore, to fully demonstrate the effectiveness of the dual system, it is necessary to execute a cost analysis to find out if the higher construction cost of the dual system would be offset by lower earthquake-induced damages during the lifetime of the building.

In this study, a probabilistic approach has been considered to estimate losses due to earthquakes. This steps of this approach are structural analysis, hazard analysis, damage analysis and loss analysis. These steps are described briefly here:

Structural analysis: The responses of the structural and non-structural components of the buildings to seismic excitation are obtained using detailed 3D finite element models.

The ground motions used in this analysis are chosen to represent the seismic hazard range of interest for the site, and may induce inelastic response of the structures; thus appropriate nonlinear dynamic analysis is used in this step. The outcomes of the response analysis are statistical functions that relate engineering demand parameters (EDPs, such as interstory drift and floor and roof accelerations) to the hazard experienced by the structure (as discussed in Chapters 4 and 5).

Hazard analysis: Probabilistic seismic hazard is used to present the seismic hazard for the buildings. The outcome of this step is a seismic hazard curve that quantifies the annual rate of exceeding a given value of a seismic intensity measure (IM, such as PGA or PSA) (as discussed in Chapter 6 of this dissertation).

Damage analysis: Based on the data from two previous steps, structural and non-structural components' damage can be characterized in terms of fragility curves. The fragility curves are cumulative distribution functions (CDFs) representing the probability that a damage state has been attained or exceeded given a quantitative measure of the EDP (as discussed in Chapter 6 of this dissertation).

Loss analysis: A translation of damage analysis from damage quantities to decision variables, which can be used by building owners and stakeholders to make a risk management decision, is done during loss analysis. The outputs of the loss analysis can be expected annual loss (EAL), life cycle cost (LCC), or pay-off period of different structural systems. This step will be described in detail in this chapter.

Firstly, the initial cost calculation method is described. Then EALs of the different structural systems are presented. Next, the effect of seismicity of the construction site on the EAL is investigated. Finally, the pay-off periods for different configurations of the dual system are calculated and presented based on the initial costs of the buildings and the EALs.

7.2 Calculation of the Initial Cost

Initial cost is the construction cost for a new or retrofitted facility. Initial cost includes two types of cost: structural component cost and non-structural component cost. In general, initial cost can be estimated using expert opinions (e.g., subcontractors) or guidelines such as RS Means square foot costs (RS Means 2013b). In this study, the initial costs of the 6-story and 10-story structures are calculated using 2013 RS Means data. The major structural cost items are steel columns, beams and braces, concrete slab with metal decks, and open web steel joists. The construction cost consists of the bare cost (material, labor and equipment costs) and overhead and profit (O & P).

The non-structural cost items that are considered in this study are: (1) exterior wall and glass blocks, (2) heavy partitions, (3) heating, ventilation, and air conditioning (HVAC) systems, and (4) the fire suppression system. As previously mentioned, the first two components are representative of drift-sensitive non-structural components and the last two are representative of acceleration-sensitive non-structural components. Table 7.1 shows the national average total cost of 6-story and 10-story conventional CBF buildings per square foot for the first story and other stories. The distinction of the cost for the first

story and the other stories is due to the difference in the story heights (15 feet in the first story and 12.5 feet in the other stories). It should be noted that, in this study, it is assumed that the total construction cost is the summation of previously mentioned items (the structural cost plus the four non-structural costs). Since some non-structural components are not considered in this study (e.g., conveying system and electrical), the summation of the calculated cost for all considered items (presented in Table 7.1) would not be the same as the total building cost presented in the RS Means guidelines (RS Means 2013b). The percentages of sub-total costs for building's components (structure and non-structure) are presented in Table 7.2. It also should be noted that the costs presented in Table 7.1 are based on the national averages for materials and installation. To adjust these costs to a specific location, the given values should simply be multiplied by the location factor for that city. The location factors for office building in Los Angeles, Boston, and Seattle are 1.07, 1.18, and 1.04, respectively.

7.3 Expected Annual Loss (EAL)

Expected annual loss determines the value of the estimated annual loss due to the seismic events. In general, EAL contains the relationships between induced losses and associated structural or non-structural response parameters. Obtaining these relationships incorporates consideration of two sets of information: (1) relationships between structural and non-structural response parameters and damage states in building components, and (2) relationships between building components damage states and the component loss. For a given value of building response parameters, the value of loss is obtained by summation of the associated losses in building components, weighted by the probability

of being in different damage states conditioned on the value of a building response parameter. EAL can be calculated as follows:

$$EAL = V \cdot \sum_{i=1}^4 (c_i \cdot P_{i|IM}) \quad (7.1)$$

where C_i is a damage factor for the i^{th} damage state and $P_{i|IM}$ is the probability of being in the i^{th} damage state given IM. In Eq. 7.1, V denotes the replacement value of a building and any additional value associated with it. It should be noted that the cost of a new component of a building is less than the cost of replacement for that component in the sense that construction costs corresponding to removing the damaged component have not been included. In order to replace a damaged building, it is necessary to demolish the building and remove it from the site. Therefore, replacement cost includes the cost to demolish the damaged building and clear the site of debris, in addition to replacing the building “in-kind.” Demolition and site clearance can increase building replacement costs by as much as to 20% to 30% (FEMA 2012). In this study the replacement cost is considered as follows:

$$Replacement\ cost = 1.3 \times initial\ cost \quad (7.2)$$

As previously mentioned, three performance limits are considered in this study (IO, LS, and CP). These three limit states generate four corresponding damage states: Insignificant (I), Moderate (M), Heavy (H), and complete (C). Table 7.3 provides a description of the damage associated with each damage state. Damage state descriptions are based on previous research by Bai. et al. (2009). Figure 7.1 illustrates a schematic of

the assumed relationship between the performance levels, the fragility curves, and the damage state definitions.

Corresponding to each damage state, there is a damage factor. Damage factors are assigned to each damage state to quantify structural and non-structural damage as a percentage of structural and non-structural replacement cost. In other words, structural damage factors indicate the cost of structural repairs as a percentage of the replacement value of the structural portion of a building. Moreover, non-structural damage factors are defined as the cost of non-structural repairs as a percentage of the replacement value of the non-structural portion of a building. In this study, damage factors are considered deterministic and their mean values are presented in Table 7.4 based on the previous work by Bai et al. (2009). It should be noted that it is assumed that the same percentage holds for non-structural damage factors, according to Williams et al. (2009).

To calculate EAL using Eqs. 7.1 and 7.2, it is necessary to approximate the initial cost of the different structural systems. Calculation of the initial cost of the CBF system is straightforward, as discussed in section 7.2. To calculate the initial cost of the dual system, two parameters are defined:

$$\alpha = \frac{Cost_{ton}^{SC-CBF}}{Cost_{ton}^{CBF}} \quad (7.3)$$

$$\beta = \frac{Cost_{sq.ft}^{Dual}}{Cost_{sq.ft}^{CBF}} \quad (7.4a)$$

in which $Cost_{ton}^{CBF}$ is the cost per unit weight of the CBF system and $Cost_{ton}^{SC-CBF}$ is the cost per unit weight of the SC-CBF part of the dual system. $Cost_{ton}^{CBF}$ is taken from RS Means Building Construction Cost Data (2013a) for a structural steel project (section 05 12 23.77 0300) which is equal to \$3775/ton, including overhead and profit. On the other hand, $Cost_{sq.ft}^{CBF}$ is the total cost of the CBF system (structural and non-structural) per square foot. $Cost_{sq.ft}^{Dual}$ is the total cost of the Dual system per square foot considering all structural and non-structural components. The relationship between α and β is illustrated in Figure 7.2 for different configurations of the dual system. The relationship between α and β is calculated based on the assumption that both methods for calculating the initial cost (considering the weight of the steel frame and considering the area) should conclude the same cost for structural components. The detailed calculations of the weight of 6-story and 10-story buildings are presented in Appendix A. Figure 7.2(a) demonstrates the relationship between α and β in D4-2, and shows that if the construction cost per unit weight of the SC-CBF part of the dual system is assumed to be as much as twice of that in the conventional CBF system, then the total cost (including structure and non-structure) will increase by 10% in the D4-2 system compared with that of the conventional CBF system. This value is about 8.2% in D3-3, 6.5% in D5-5, and 9% in D7-3, as shown in Figure 7.2(b), (c), and (d), respectively. Eq. 7.4a can also be written in this format:

$$Cost_{sq.ft}^{Dual} = \beta \cdot Cost_{sq.ft}^{CBF} \quad (7.4b)$$

As previously mentioned, the construction cost of the dual system is expected to be higher than that of the conventional CBF due to the special details required in fabrication and erection. It means that the expected value for β is greater than one ($\beta > 1$). The important question is how large β could be to have an economically efficient dual system. To answer this question, comparative EAL curves for the CBF and dual system should be developed.

According to the discussion above, EALs for different structural systems are calculated and illustrated in Figures 7.3 to 7.6. Figure 7.3 shows the structural EAL for 6-story buildings. The EAL for the 6-story CBF is a constant value and for 6-story dual systems is a function of β . As expected, higher values of β will lead to larger EAL in the dual systems. Even for very large values of β (e.g., $\beta=1.20$, which corresponds to $\alpha > 3$), the dual system show much less structural EAL. Figure 7.4 shows the total EAL (structure and non-structure) in 6-story buildings. Adding non-structural components will reduce the difference between the EAL of the 6-story CBF and the 6-story dual systems. This is mainly due to the better performance of the acceleration-sensitive non-structural components in the CBF system. Figure 7.4 shows that the dual systems have less total EAL for a large range of β ($1 < \beta < 1.25$) which corresponds to a very large range of α ($1 < \alpha < 3$). Figure 7.5 shows the structural EAL for the 10-story buildings. Both D5-5 and D7-3 systems show less EAL for a reasonable range of β (recall that $\beta=1.065$ and 1.09 correspond to $\alpha=2$ in D5-5 and D7-3, respectively). However, adding the non-structural components will lead to higher EAL in the 10-story dual systems in comparison to the CBF system, as shown in Figure 7.6. Both D5-5 and D7-3 show similar EAL, higher than

that of the CBF system. This is mainly due to the reduced annual probability of failure of acceleration-sensitive non-structural components in the CBF, as shown in Chapter 6. It should be noted that the acceleration-sensitive non-structural components comprise more than 22% of the total cost of the buildings, as shown in Table 7.2. Figures 7.3 to 7.6 show the EAL of buildings in a high-seismic zone (LA). As demonstrated in these figures, both 6-story and 10-story dual systems show less structural EAL in comparison with the CBF system. The 6-story dual system also shows less *total* EAL in comparison with the 6-story CBF system. To investigate the effect of seismicity of the construction zone on the loss estimation, structural EAL for 6-story and 10-story buildings in two additional seismicity zones (moderate-seismic zone and low-seismic zone) are calculated and presented in the following section.

7.4 Site Effect Investigation

To investigate how a building's location and its seismicity affect the EAL, identical example buildings are considered in three different locations: Los Angeles, Boston, and Seattle. Los Angeles is a representative of a high-risk seismic zone, Boston is a representative of a low-risk seismic zone, and Seattle is a representative of a moderate-risk seismic zone.

In this study the prototype structures are designed for the Los Angeles site (i.e., the design is not altered to accommodate the reduced seismic loads in the lower-risk seismic zones). The same demand models are used to predict structural demands in Boston and Seattle. Figures 7.7 and 7.8 illustrate the justification of this assumption. Figure 7.7

shows computed demand using FEM versus prediction from proposed probabilistic demand models for 6-story buildings in LA, BO and SE. As previously mentioned, perfectly predicted data would lie along the 45-degree solid line. All data points in Figure 7.7 are evenly scattered about the solid line, with most of the data within the two dashed lines (plus or minus one standard deviation) for each location. This illustrates that the developed probabilistic demand models provide unbiased and accurate predictions for LA, BO, and SE. Figure 7.8 shows similar results for 10-story buildings. Figures 7.9 to 7.12 illustrate the structural EAL for the 6-story and 10-story buildings in Boston (low-seismic zone) and in Seattle (moderate-seismic zone). As shown in Figure 7.9, the structural EAL for the Dual system in the most reasonable ranges of β ($1.06 < \beta < 1.1$) are larger than that of the CBF system. Only for very small β values ($\beta < 1.04$) do the dual systems show less EAL than the CBF system. This is mainly due to the fact that the EAL is already so small for the conventional building in Boston, and the additional cost of the dual system cannot be compensated by better structural performance. Similar results are shown in Figure 7.10 for 10-story buildings. Figures 7.11 and 7.12 illustrate the structural EAL for 6-story and 10-buildings in Seattle (moderate-seismic zone). As expected, the result for Seattle is in between of the results for Los Angeles and Boston; the EAL for the 6-story dual systems is less than that of the CBF system in a wider range of β in comparison with BO and in a smaller range of β in comparison with LA. As shown in Figure 7.11, the structural EAL for 6-story Dual systems are less than that of the CBF for the entire reasonable range of β . However, only for very small values of β ($\beta < 1.05$) do the 10-story dual systems show less EAL than the CBF system.

Therefore, it is not economically justifiable to construct a building with the proposed lateral load bearing system (dual system) in the low-seismic and moderate-seismic zones, with the possible exception of low-rise structures in moderate-seismic zones (e.g., the 6-story dual systems in Seattle).

7.5 Pay-off Period

To calculate the benefit of using the dual system instead of the CBF system in a building, the pay-off period is calculated for each dual system as a function of β . Pay-off period is the time (in years) over which the higher initial construction costs of the dual systems are compensated by lower earthquake-induced losses throughout the lifetime of the buildings. As expected, larger values of β lead to larger pay-off periods. It should be noted that a discount rate per year is assumed to be zero, due to the negligible inflation rate in the US during the past five years. The pay-off period is calculated as follows:

$$PP(\beta) = \frac{Cost_{Dual}(\beta) - Cost_{CBF}}{EAL_{CBF} - EAL_{Dual}(\beta)} \quad (7.5)$$

where $PP(\beta)$ is the pay-off period (in years), $Cost_{Dual}(\beta)$ is the initial construction cost of the dual system (which is function of β), $Cost_{CBF}$ is the initial construction cost of the CBF system (which is a constant), EAL_{CBF} is the expected annual loss of the CBF system, and $EAL_{Dual}(\beta)$ is the expected annual loss of the dual system.

Using Eq. 7.5 and the values of total EAL in LA from Figure 7.4, the pay-off period for 6-story Dual systems as a function of β is shown in Figure 7.13. Assuming $\alpha=2$

(corresponds to $\beta=1.1$ in D4-2 and $\beta=1.082$ in D3-3), the pay-off period for D4-2 and D3-3 are 31 and 12 years, respectively. This means that the higher initial cost of the dual systems will be compensated after 31 years and 12 years for D4-2 and D3-3, respectively, due to better performance and less expected annual loss of the dual systems. Since the 10-story dual systems showed larger EAL than the CBF systems, there is no pay-off for 10-story dual systems (the 10-story dual system remains more expensive than the CBF system despite its better structural performance).

It is also interesting to calculate the pay-off period for 6-story and 10-story dual systems considering only the structural EAL. Figure 7.14 shows the pay-off period for 6-story dual systems D4-2 and D3-3, which are only 8 years and 3 years, respectively. It can be concluded that by enhancing the capacities of acceleration-sensitive non-structural components (which may not be significant in comparison with the total cost of the building), the pay-off period can reduce significantly in 6-story dual systems. Finally, Figure 7.15 shows the pay-off period in 10-story dual systems considering only structural EAL. Even by neglecting the non-structural EAL, the pay-off period for D5-5 system (assuming $\alpha=2$) is about 40 years, which indicates that the proposed system is not an effective system to improve the seismic performance of the high-rise buildings.

Table 7.1 Square foot costs (RS Means 2013)

	6-story (\$/sq.ft)	10-story (\$/sq.ft)
1st story	154.5	150.9
Other stories	143.8	140.9

Table 7.2 Percentage of sub-total costs for building's components (RS Means 2013)

Performance component		Percentage (%)
Structure		23.9
Non-structure	Exterior walls and glass blocks	25.4
	Heavy partitions	28.2
	HVAC	18.3
	Fire suppression	4.2
Total		100

Table 7.3. Damage state descriptions (Bai et al. 2013)

Damage state	Description
Insignificant (I)	Damage requires no more than cosmetic repair. No structural repairs are necessary. For non-structural elements, repairs could include spackling partition cracks, picking up spilled contents, putting back fallen ceiling tiles, and righting equipment.
Moderate (M)	Repairable structural damage has occurred. The existing elements can be repaired essentially in place, without substantial demolition or replacement of elements. For non-structural elements, repairs would include minor replacement of damaged partitions, ceilings, contents, and equipment or their anchorages.
Heavy (H)	While the damage is significant, the structure is still standing. Structural damage would require major repairs, including substantial demolition or replacement of elements. For non-structural elements, repairs would include major replacement of damaged partitions, ceilings, contents, equipment or their anchorages.
Complete (C)	Damage is so extensive that repair of most structural elements is not feasible. Structure is destroyed or most of the structural members have reached their ultimate capacities.

Table 7.4 Damage factors (Bai et al. 2013)

Damage state	Damage factor (%)
Insignificant (I)	0.50
Moderate (M)	15.5
Heavy (H)	55.0
Complete (C)	90.0

Figure 7.1 Illustration of relationship between fragility curves and damage states (Bai et al. 2009)

Figure 7.2 Illustration of relationship between α and β (a) D4-2 system, (b) D3-3 system, (c) D5-5 system, and (d) D7-3 system

Figure 7.3 Structural Expected Annual Loss (\$) in 6-story buildings in LA

Figure 7.4 Total Expected Annual Loss (\$) in 6-story buildings in LA

Figure 7.5 Structural Expected Annual Loss (\$) in 10-story buildings in LA

Figure 7.6 Total Expected Annual Loss (\$) in 10-story buildings in LA

Figure 7.7 Computed demand using FEM vs. prediction from proposed probabilistic inter-story demand models for (a) D4-2, (b) D3-3, (c) D2-4, and (d) 6-story CBF system.

Figure 7.8 Computed demand using FEM vs. prediction from proposed probabilistic inter-story demand models for (a) D7-3 system, (b) D5-5 system, (c) D4-6 system, and (d) 10-story CBF system

Figure 7.9 Structural Expected Annual Loss (\$) in 6-story buildings in BO

Figure 7.10 Structural Expected Annual Loss (\$) in 10-story buildings in BO

Figure 7.11 Structural Expected Annual Loss (\$) in 6-story buildings in SE

Figure 7.12 Structural Expected Annual Loss (\$) in 10-story buildings in SE

Figure 7.13 Pay-off period (year) in 6-story buildings considering total EAL

Figure 7.14 Pay-off period (year) in 6-story buildings considering structural EAL

Figure 7.15 Pay-off period (year) in 10-story buildings considering structural EAL

CHAPTER VIII

SUMMARY AND CONCLUSIONS

8.1 Summary of Study

Steel concentrically braced frame (CBF) systems are commonly used in structures in seismically-active regions, despite their limited capacity for lateral displacement due to structural damage. Self-centering CBFs (SC-CBFs) have been developed recently to improve the drift capacity of braced frame systems without incurring structural damage under moderate earthquakes. Since the SC-CBF is expected to have a greater construction cost than a conventional CBF, a novel dual structural system has been proposed in this study. In the dual system, the lower stories of the lateral force resisting system are SC-CBFs that act as a ductile sub-structure for the conventional CBFs in the upper stories.

This research introduces the design basis of the dual system, which is based on a modified response spectrum analysis procedure. The design process has two phases: the post-tensioning steel bar design phase and the structural member design phase. Six different configurations of the dual system were designed and modeled in finite element software, and their performances were evaluated and compared with a structural system that consisted of conventional CBFs. The different dual system configurations investigated in this study were: D2-4 with a two-story SC-CBF below a four-story CBF, D3-3 with a three-story SC-CBF below a three-story CBF, and D4-2 with a four-story

SC-CBF below a two-story CBF in the six-story buildings. Additionally, D4-6, D5-5, and D7-3 configurations are investigated with four, five and seven stories, respectively, of SC-CBF at the lower stories in the ten-story buildings. Then nonlinear dynamic analysis was conducted, subjecting the structures to various hazard levels of ground motions (corresponding to 50%, 10%, and 2% probability of exceedance in 50 years).

To assess the seismic performance of the buildings, both structural and non-structural components were considered. In this study, four non-structural components were considered: (1) exterior walls and glass blocks, (2) heavy partitions, (3) HVAC, and (4) a fire suppression system. Using numerical analysis results, probabilistic demand models were developed to predict the peak interstory drift, floor acceleration and roof acceleration demands. The formulation of the probabilistic demand models for interstory drift consisted of a selected deterministic model commonly used in practice, correction terms used to improve the prediction accuracy of the model, and model error. Since no conventional deterministic demand models were found for floor acceleration and roof acceleration, the propose demand model for acceleration only consisted of explanatory functions of constants and normalized intensity measures for floor and roof acceleration.

To fully demonstrate the effectiveness of the proposed dual system, probabilistic performance-based assessment (PBA) is executed in this study. In general, PBA has four major steps: (1) response analysis, (2) hazard analysis, (3) damage analysis, and (4) loss analysis. In the response analysis step, the response of structural and non-structural components of the buildings to seismic excitation is obtained using numerical model of the structures. The outcomes of this step are statistical functions that relate engineering

demand parameters (EDPs) to the hazard experienced by the structures. The hazard analysis step is used to describe the seismic hazard for the structures. The outcomes of this step are seismic hazard curves that quantify the annual rate of exceeding a given value of an earthquake intensity measure (IM). In the damage analysis step, seismic fragility curves and engineering demand hazard curves are developed from the proposed probabilistic demand models and code-defined capacities, considering three performance levels: immediate occupancy, life safety, and collapse prevention for structural and non-structural components. In the loss analysis step, to estimate structural and non-structural losses, a set of damage states and corresponding deterministic damage factors are adopted. The outcomes of loss analysis are expected annual loss (EAL) and pay-off period of different structural systems.

8.2 Conclusions

The focus of this research is to develop a methodology to design and assess seismic vulnerability of the proposed dual system and to estimate economic losses related to structural and non-structural damage due to seismic events. The findings and conclusions drawn from this study are summarized as follows:

- The proposed probabilistic demand models provide accurate and unbiased predictions for the selected engineering demand parameters EDPs (interstory drift, floor acceleration and roof acceleration).

- PGV and PSA are the best intensity measures to predict the interstory drift demand and PGA and PSA are the most statistically significant intensity measures to predict floor and roof acceleration demands.
- Comparison of the seismic fragility curves for the three performance levels for structural components indicate that the D4-2, D3-3, D5-5 and D7-3 dual systems are less vulnerable than the CBF system, and are less likely to attain or exceed specific structural capacity levels conditioned on the seismic intensity measures. However, the D2-4 and D4-6 dual systems did not show noticeably better performance than the CBF, as only few stories in the dual system are the SC-CBFs, limiting their effect on the performance.
- Furthermore, from a comparison of the fragility curves of the three different dual system configurations in the six-story buildings, both D3-3 and D4-2 have lower probabilities of failure than D2-4, while the performance comparison of D3-3 and D4-2 depends on the relative hazards of PGV and PSA. In the ten-story Dual system configurations both D5-5 and D7-3 have less probability of failure than D4-6 in the life safety and collapse prevention performance levels. For the immediate occupancy performance level, D5-5 did not show noticeable better performance than D4-6. The performance comparison of D5-5 and D7-3 depends on the relative hazards of PGV and PSA.
- Comparison of the seismic fragility curves of the non-structural components indicates that drift-sensitive components in dual systems are generally less

vulnerable than those in the CBF system; however, acceleration-sensitive components in the dual systems are more likely to attain or exceed specific capacity levels conditioned on the intensity measures.

- Fragility curves are not sufficient tools to assess the performance of the buildings since they are not capable of considering the probability of occurrence of the selected intensity measures.
- Engineering demand hazard curves in Los Angeles (high-seismic zone) show that using the dual system instead of the CBF leads to a decrease in the annual probability of failure in structural components, especially for the 6-story buildings.
- Engineering demand hazard curves in Los Angeles for acceleration-sensitive components show that annual probability of failure of (acceleration-sensitive) non-structural elements in the CBF system are much less (approximately two order of magnitudes) than that in the dual systems.
- Detailed cost analysis led to a relationship between α and β , where α is the ratio of the initial cost of the dual system (per ton) to that of the CBF system, and β is the ratio of the total construction cost of the dual system (per square foot) to that of the CBF system. An example of this relation is that by assuming the construction cost per unit weight of the dual system to be twice of that in the CBF system ($\alpha = 2$), the total cost of the Dual system will increase by about 10% considering all structural and non-structural elements

($\beta = 1.1$). The relationships between α and β are developed separately for D3-3, D4-2, D5-5, and D7-3.

- Cost analysis shows that the effectiveness of the dual systems can vary significantly with site location and building height. While the 6-story dual systems show much less total expected annual loss (EAL) than the CBF in Los Angeles, the 10-story dual systems show higher EAL(\$) than CBF in Los Angeles. On the other hand, dual systems show less structural EAL in Los Angeles (high-seismic zone) in a wide range of β , while CBF systems show less structural EAL than dual system in Boston (low-seismic zone).
- Cost analyses show that the dual system is economically efficient for low-rise buildings in high-seismic zones and may be economically efficient for low-rise buildings in moderate-seismic zones.
- Cost analyses also show that the drawback of the dual system is its high acceleration demands due to the rocking behavior of the system. These high demands lead to higher probability of failure (for acceleration-sensitive non-structural components) and higher EAL. By enhancing the capacities of acceleration-sensitive non-structural components (which is not a significant cost with respect to the total cost of the building), the pay-off period may reduce significantly in 6-story dual systems.
- Cost analysis show that the pay-off period for D3-3 and D4-2 systems are 12 and 31 years, respectively, which means that the higher initial cost of the dual systems will be compensated after 31 years and 12 years for D4-2 and D3-3

systems due to better performance and less expected annual loss of the dual systems.

- Cost analysis show that the pay-off period for the 10-story dual systems, considering only structural EAL, is more than 40 years in Los Angeles, which indicates that the proposed system is not an economically effective system to improve the seismic performance of the high-rise buildings.

8.3 Future Work

This study proposed a design method and a general framework to assess structural and non-structural performance and corresponding losses due to earthquake excitation. Recommendations for potential extended studies and selected future research are listed below:

- Future research could investigate dual system structures with varied design parameters (e.g., frame geometry, building height) to further demonstrate the improved seismic performance of the dual system in comparison to the conventional CBF system. Specially, 4-story and 8-story buildings are recommended to be considered to further generalize the results.
- Possible damage mitigation for non-structural components could be investigated further based on higher capacities. Better performance could be obtained by considering specific provisions to support acceleration-sensitive non-structural elements in the dual system.

- In this study, four non-structural elements are considered in cost analysis. It is recommended that all possible non-structural components in office buildings are considered in future research for more comprehensive results. In addition, corresponding costs due to maintenance, relocation, down-time and fatality are recommended for inclusion in future studies.
- Loss estimation using other software such as HAZUS or PACT with more refined and updated information could be conducted to assess vulnerability in terms of the expected loss.
- Hybrid simulation experimentation for scaled prototype structures could be conducted to validate the numerical analysis and results.

REFERENCES

- AISC (2010a), *ANSI/AISC 341-10: Seismic Provisions for Structural Steel Building*, American Institute of Steel Construction. Chicago, IL.
- AISC (2010b), *ANSI/AISC 360-10: Specification for Structural Steel Buildings*, American Institute of Steel Construction. Chicago, IL.
- ASCE (2007), *ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers. Reston, VA.
- ASCE (2010), *ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures*. American Society of Civil Engineers. Reston, VA.
- Backer, J.W (2007), “Probabilistic structural response assessment using vector-valued intensity measures”, *Earthquake Engineering and Structural Dynamics*, 36(5), 1861–83.
- Baker JW, Cornel CA. (2008), “Uncertainty propagation in probabilistic seismic loss estimation.” *Structural Safety*, (30), 236–252.
- Badpay A, Arbabi F. A. (2008), “comparison of seismic performance and vulnerability of buckling restrained and conventional steel braced frames.” In: *Proc. of the 14th world conference on earthquake engineering*, Beijing, China, 12-17 October 2008.
- Bai, J.W., Hueste, M.B.D. and Gardoni, P. (2009). “Probabilistic assessment of structural damage due to earthquakes for buildings in Mid-America.” *J Struct Eng*, 135(10), 1155-1163.
- Box, G.E.P. and Cox, D.R. (1964), “An analysis of transformations”, *Journal of the Royal Statistical Society*, 26(2), 211–246.
- BSSC (2003). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. FEMA 450. Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C.

- Cha E.J, Ellingwood B.R. (2014), "Attitudes towards acceptance of risk to buildings from extreme winds." *Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance*, 10(6), 697-707.
- Choe, D., Gardoni, P. and Rosowsky, D. (2007), "Closed-form Fragility Estimates, Parameter Sensitivity and Bayesian Updating for RC Columns", *Journal of Engineering Mechanics*, 133(7), 833-843.
- Christopoulos, C.; Tremblay, R.; Kim, H.-J.; and Lacerte, M. (2008). "Self-Centering Energy Dissipative Bracing System for the Seismic Resistance of Structures: Development and Validation," *ASCE Journal of Structural Engineering*, 134(1), pp 96-107
- Dyanati M, Huang Q, Roke D.(2014), Structural and nonstructural performance evaluation of self-centering, concentrically braced frames under seismic loading. In: *Proceedings of structures congress 2014*. Boston, Massachusetts, USA.
- Elshamy M. (1992), "Bivariate extreme value distributions." NASA Contractor Report No. 4444. NASA.
- Fahnestock, L.A., Sause, R. and Ricles, J.M. (2007), "Seismic Response and Performance of Buckling-Restrained Braced Frames", *Journal of Structural Engineering*, ASCE, 133(9), 1195-1204.
- FEMA, 2012. Next-Generation Methodology for Seismic Performance Assessment of Buildings, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Report No. FEMA P-58, Washington, D.C.
- Goda K, Lee C.S, Hong H.P. (2010), "Lifecycle cost-benefit analysis of isolated buildings." *Structural Safety*, 32, 52-63.
- Gumbel E.J, Mustafi C.K. (1967), "Some analytical properties of bivariate extremal distributions." *Journal of the American Statistical Association*, 62(318),569-588.
- Huang, Q., Gardoni, P. and Hurlbaush, S. (2010), "Probabilistic seismic demand models and fragility estimates for reinforced concrete highway bridges with one single-column bent", *Journal of Engineering Mechanics*, 136(11), 1340-1353.
- Kang Y.J, Wen Y.K. (2000), "Minimum life-cycle cost structural design against natural hazards," *Structural research series No.629*. University of Illinois at Urbana-Champaign, Urbana, Illinois, USA.
- Kinali, K. and Ellingwood, B.R. (2007), "Seismic fragility assessment of steel frames for consequence-based engineering: a case study for Memphis, TN", *Engineering Structure*, 29(6), 1115-1127.

- Krawinkler, H., Medina, R., and Alavi, B. (2003). "Seismic drift and ductility demands and their dependence on ground motions," *Engineering Structures*, 25, 637-653.
- Kutner, M., Nachtsheim, C., Neter, J. and Li, W. (2004), *Applied Linear Statistical Models*, 5th Edition, McGraw-Hill, Irvine.
- Lee, K. and Foutch, D.A. (2004), "Performance evaluation of damaged steel frame buildings subjected to seismic loads", *Journal of Structural Engineering*, 130(4), 588-599.
- Lin K.C, Lin C.C.J, Chen J.Y, Chang H.Y. (2010), "Seismic reliability of steel framed buildings". *Structural Safety*, 32(3),174-182.
- Mahin, S. and Uriz, P. (2004), "Seismic performance assessment of concentrically braced steel frames". *Proc. of 13th World Conf. on Earthquake Engineering*, Vancouver, Paper No. 1639. B.C., Canada: WCEE.
- Malley, J.O. and Popov, E.P. (1984), "Shear links in eccentrically braced frames", *Journal of Structural Engineering*, 110(9), 2275-95.
- Mayes, R.L., Goings, C.B., Naguib, W.I. and Harris, S.K. (2005), "Comparative seismic performance of four structural systems". *Proceedings of 74th Annual Convention of structural engineers association of California*, San Diego, California, USA.
- McKenna, F (1997). *Object Oriented Finite Element Programming: Frameworks for Analysis, Algorithms and Parallel Computing*, University of California, Berkeley, CA.
- Moehle, J. and Deierlein, G. G. (2004). "A Framework Methodology for Performance-Based Engineering," *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004.
- Nielson, B.G. and DesRoches, R. (2007), "Analytical seismic fragility curves for typical bridges in the central and southeastern United States", *Earthquake Spectra*, 23(3), 615-633.
- Padgett J.E, Dennemann K, Ghosh J. (2010), "Risk-based seismic life-cycle cost-benefit (LCC-B) analysis for bridge retrofit assessment." *Structural Safety*, 32, 165-173.
- Rao, C.R. and Toutenburg, H. (1997), *Linear models, Least squares and Alternatives*, Springer, New York.
- Roke, D., Sause, R., Ricles, J.M., Seo, C.Y. and Lee, K.S. (2006), "Self-centering seismic-resistant steel concentrically-braced frames", *Proceedings of 8th US national conference on earthquake engineering*, San Francisco, California, April 2006.

- Roke, D., Sause, R., Ricles, J.M. and Chancellor, N.B. (2010), “Damage-free seismic-resistant self-centering concentrically-braced frames”. ATLSS Report 10-09, Lehigh University, Bethlehem, PA, USA; 2010.
- Roke, D. Jeffers, B. (2012), “Analytical Parametric Study of Self-Centering Concentrically-Braced Frames with Friction-Based Energy Dissipation”. Proc. of 7th Intl. Conf. on Behavior of Steel Structures in Seismic Areas, Santiago, Chile, Paper No. 0087.
- Roke, D., Hasan, M.R. (2012), “The Effect of Frame Geometry on the Seismic Response of Self-Centering Concentrically-Braced Frames”. International Journal of Civil and Environmental Engineering, 6, 140-145.
- RS Means. (2013a), RS Means Building Construction Cost Data 2013. Kingston, Massachusetts: RS Means.
- RS Means. (2013b), RS Means Square Foot Costs 2013. Kingston, Massachusetts: RS Means.
- Seo, C.Y. and Sause, R. (2005), “Ductility demands on self-centering systems under earthquake loading”, ACI Structural Journal, 102 (2), 275–285.
- Sheather, S. (2009), Modern Approach to Regression with R. New York: Springer.
- Somerville, P., Smith, S., Punyamurthula, S. and Sun, J. (1997), Development of ground motion time histories for phase 2 of the FEMA/SAC steel project. Report No SAC/BD-97/04, SAC Joint Venture, Sacramento, California, USA.
- Song, J. and Ellingwood, B.R. (1999), “Seismic reliability of special moment steel frames with welded connections: I and II”, Journal of Structural Engineering, 125(4), 357–84.
- Tremblay R, Poirier L-P, Bouaanani N, Leclerc M, Rene V, Fronteddu L, Rivest (2008), S. Innovative viscously damped rocking braced steel frames. Proc., 14th World Conf. on Earthquake Eng., Beijing, China.
- Trifunac, M.D. and Brady, A.G. (1975), “A study on the duration of strong earthquake ground motion”, Bulletin of the Seismological Society of America, 65(3), 581-626.
- Uriz, P. and Mahin, S. (2004), “Seismic vulnerability of concentrically braced steel frames”, Steel Structures 4, 239-248.
- USACE. (2010), “Beyond the factor of safety: Developing Fragility Curves to Characterizing System Reliability”, Geotechnical and Structures Laboratory ERDC SR-10-1, United States Army Corps of Engineers, Washington DC.

- U.S. Geological Survey (USGS). (2014). "Database of U.S. Geological Survey for ground motion hazards". Available online at <http://www.usgs.gov/>
- Williams RJ, Gardoni P, Bracci JM. (2009), "Decision analysis for seismic retrofit of structures", *Structural Safety* 31(2):188–96.
- Wen Y.K, Ang A.H. (1991), "Reliability and cost-effectiveness of structures with active control. Intelligent Structures-2: Monitoring and Control." In: *Proceedings of the International Workshop on Intelligent Systems*, Perugia, Italy, 27-29, June 1991.
- Wen Y.K, Shinozuka M. (1994), "Cost-Effectiveness Issues in Active Structural Control," In: *Proceedings of First World Conference on Structural Control*, Pasadena, California, August 3-5.
- Wen Y.K, Shinozuka M. (1998), "Cost-effectiveness in active structural control." *Engineering Structures*, 20(3), 216–221.
- Wiebe L, Christopoulos C, Tremblay R, Leclerc M. (2013) "Mechanisms to limit higher mode effects in a controlled rocking steel frame. 1: Concept, modelling, and low-amplitude shake table testing". *Earthquake Engineering & Structural Dynamics*, 42:1053–1068.
- Yang T, Murphy M. (2012), Performance evaluation of three pre-qualified steel systems in Canada. In: *Proc. of the STESSA 2012 conference on behavior of steel structures in seismic areas*, Santiago, Chile.
- Zareian, F., and Krawinkler, H. (2009). "Simplified performance based earthquake engineering." Rep. No. TB 169, The John A. Blume Earthquake Engineering Research Center, Stanford Univ., Stanford, CA.

APPENDICES

APPENDIX A

PEAK DEMAND RESPONSES OF THE STRUCTURES

Table A.1 Peak Interstory drift ratio (%) for 6-story buildings

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
Imperial Valley, 1940, El Centro	LA	1.49	1.16	1.40	0.91
Imperial Valley, 1940, El Centro	LA	1.77	1.06	1.54	1.12
Imperial Valley, 1979, Array #05	LA	1.95	1.34	1.71	0.84
Imperial Valley, 1979, Array #05	LA	0.84	0.59	0.68	0.58
Imperial Valley, 1979, Array #06	LA	1.23	0.79	0.73	0.54
Imperial Valley, 1979, Array #06	LA	1.33	0.80	0.75	0.53
Landers, 1992, Barstow	LA	0.91	0.78	1.09	0.70
Landers, 1992, Barstow	LA	1.81	1.29	1.19	0.79
Landers, 1992, Yermo	LA	4.46	2.73	2.49	1.19
Landers, 1992, Yermo	LA	2.96	1.91	1.68	0.69
Loma Prieta, 1989, Gilroy	LA	3.55	1.80	1.45	0.82
Loma Prieta, 1989, Gilroy	LA	1.22	0.68	0.86	0.71
Northridge, 1994, Newhall	LA	2.09	1.75	2.48	0.97
Northridge, 1994, Newhall	LA	3.29	2.06	2.71	1.00
Northridge, 1994, Rinaldi RS	LA	3.11	1.90	1.83	1.05
Northridge, 1994, Rinaldi RS	LA	4.00	2.96	3.17	1.73
Northridge, 1994, Sylmar	LA	2.89	1.37	1.51	0.97
Northridge, 1994, Sylmar	LA	2.71	2.50	1.98	1.18
North Palm Springs, 1986	LA	1.42	1.15	1.06	0.80
North Palm Springs, 1986	LA	3.86	2.82	2.82	1.13
Long Beach, Vernon CMD Bldg.	SE	0.53	0.37	0.41	0.37
Long Beach, Vernon CMD Bldg.	SE	0.57	0.36	0.24	0.22
Morgan Hill, 1984, Gilroy	SE	1.16	0.74	0.80	0.59
Morgan Hill, 1984, Gilroy	SE	1.22	0.81	0.68	0.62

Table A.1 Peak Interstory drift ratio (%) for 6-story buildings (Continued)

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
West. Washington, Olympia, 1949	SE	0.81	0.63	0.85	0.42
West. Washington, Olympia, 1949	SE	0.66	0.58	0.47	0.35
West. Washington, Seattle Army B., 1949	SE	1.96	1.23	1.48	0.85
West. Washington, Seattle Army B., 1949	SE	1.73	1.14	1.50	0.94
North Palm Springs, 1986	SE	0.82	0.68	0.63	0.49
North Palm Springs, 1986	SE	2.15	1.28	0.90	0.54
Puget Sound, Wa., Olympia, 1949	SE	1.46	0.89	1.38	0.88
Puget Sound, Wa., Olympia, 1949	SE	1.18	0.89	1.02	0.62
Puget Sound, Wa., Federal OFC B., 1949	SE	1.22	0.89	1.15	0.56
Puget Sound, Wa., Federal OFC B., 1949	SE	0.94	0.66	0.60	0.52
Eastern Wa., Tacoma County, 1949	SE	0.69	0.62	0.62	0.46
Eastern Wa., Tacoma County, 1949	SE	0.82	0.90	0.99	0.79
Llolleo, Chile 1985	SE	1.29	1.15	1.37	0.82
Llolleo, Chile 1985	SE	1.27	1.44	0.89	0.71
Vinadel Mar, Chile, 1985	SE	2.29	1.75	2.44	1.26
Vinadel Mar, Chile, 1985	SE	1.37	0.92	1.1	0.69
Simulation, hanging wall	BO	0.29	0.15	0.13	0.11
Simulation, hanging wall	BO	0.19	0.11	0.11	0.09
Simulation, foot wall	BO	0.21	0.19	0.19	0.11
Simulation, foot wall	BO	0.40	0.38	0.28	0.25
New Hampshire, 1982	BO	0.26	0.21	0.15	0.12
New Hampshire, 1982	BO	0.28	0.18	0.14	0.10
Nahanni, 1985	BO	0.15	0.08	0.09	0.08
Nahanni, 1985	BO	0.14	0.08	0.08	0.06
Nahanni, 1985	BO	0.19	0.11	0.11	0.07
Nahanni, 1985	BO	0.10	0.14	0.14	0.10
Nahanni, 1985	BO	0.08	0.07	0.07	0.04
Nahanni, 1985	BO	0.09	0.08	0.09	0.06
Saguenay, 1988	BO	0.21	0.16	0.15	0.08
Saguenay, 1988	BO	0.29	0.18	0.17	0.11
Saguenay, 1988	BO	0.45	0.43	0.44	0.35
Saguenay, 1988	BO	0.12	0.15	0.09	0.08
Saguenay, 1988	BO	0.38	0.32	0.28	0.25
Saguenay, 1988	BO	0.32	0.22	0.20	0.15
Saguenay, 1988	BO	0.31	0.19	0.19	0.16
Saguenay, 1988	BO	0.35	0.25	0.25	0.21
1995 Kobe	LA	4.46	4.54	5.78	3.78
1995 Kobe	LA	3.16	2.70	3.00	1.60
1989 Loma Prieta	LA	2.77	2.00	1.66	0.84

Table A.1 Peak Interstory drift ratio (%) for 6-story buildings (Continued)

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
1989 Loma Prieta	LA	7.55	5.56	6.40	1.70
1994 Northridge	LA	3.98	3.60	4.13	2.17
1994 Northridge	LA	5.48	4.81	5.98	3.90
1994 Northridge	LA	8.27	5.26	4.04	1.52
1994 Northridge	LA	6.03			
1974 Tabas	LA	2.19	1.58	1.58	0.83
1974 Tabas	LA	4.60	2.50	1.80	1.16
Elysian Park (simulated)	LA	3.51	3.46	4.00	3.05
Elysian Park (simulated)	LA	4.16	3.24	4.72	3.08
Elysian Park (simulated)	LA	6.49	3.68	3.76	1.79
Elysian Park (simulated)	LA	4.02	4.38	3.69	1.26
Elysian Park (simulated)	LA	12.21			
Elysian Park (simulated)	LA	11.33			
Palos Verdes (simulated)	LA	8.18	5.60	5.59	2.83
Palos Verdes (simulated)	LA	11.65	8.57	7.06	2.31
Palos Verdes (simulated)	LA	3.53	2.39	1.91	0.82
Palos Verdes (simulated)	LA	9.86	7.81	4.84	1.1
1992 Mendocino	SE	6.27	3.65	2.74	1.21
1992 Mendocino	SE	1.54	1.03	1.29	0.79
1992 Erzincan	SE	2.47	2.09	2.42	1.14
1992 Erzincan	SE	5.46	3.68	2.68	1.00
1949 Olympia	SE	2.19	1.53	2.06	1.02
1949 Olympia	SE	1.87	1.46	1.77	0.67
1965 Seattle	SE	4.78	4.06	3.77	1.85
1965 Seattle	SE	2.87	2.27	2.44	1.58
1985 Valpariso	SE	2.43	2.13	2.63	1.57
1985 Valpariso	SE	3.49	3.51	4.10	2.42
1985 Valpariso	SE	3.88	3.84	3.74	2.50
1985 Valpariso	SE	1.91	1.84	2.32	1.32
Deep Interplate (simulation)	SE	4.11	4.55	5.80	1.82
Deep Interplate (simulation)	SE	1.72	1.79	2.52	1.01
1978 Miyagi-oki	SE	3.36	2.30	3.57	2.16
1978 Miyagi-oki	SE	6.59	4.89	6.33	3.23
Shallow Interplate (simulation)	SE	2.96	2.56	3.10	1.17
Shallow Interplate (simulation)	SE	3.59	2.82	2.65	1.46
Shallow Interplate (simulation)	SE	2.60	1.74	2.18	0.87
Shallow Interplate (simulation)	SE	2.61	2.85	3.18	1.38
simulation, foot wall	BO	0.69	0.43	0.30	0.25
simulation, foot wall	BO	0.63	0.46	0.41	0.36

Table A.1 Peak Interstory drift ratio (%) for 6-story buildings (Continued)

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
simulation, foot wall	BO	0.47	0.28	0.24	0.20
simulation, foot wall	BO	0.33	0.33	0.32	0.26
simulation, foot wall	BO	0.68	0.40	0.29	0.23
simulation, foot wall	BO	0.51	0.50	0.47	0.40
Nahanni, 1985 Station 1	BO	0.41	0.23	0.21	0.19
Nahanni, 1985 Station 1	BO	0.33	0.21	0.21	0.16
Nahanni, 1985 Station 2	BO	0.51	0.41	0.37	0.31
Nahanni, 1985 Station 2	BO	0.28	0.25	0.33	0.25
Nahanni, 1985 Station 3	BO	0.22	0.16	0.16	0.11
Nahanni, 1985 Station 3	BO	0.26	0.18	0.20	0.15
Saguenay, 1988	BO	0.56	0.32	0.32	0.20
Saguenay, 1988	BO	0.74	0.50	0.45	0.39
Saguenay, 1988	BO	0.96	0.97	1.04	0.88
Saguenay, 1988	BO	0.37	0.39	0.29	0.29
Saguenay, 1988	BO	0.87	0.87	0.81	0.71
Saguenay, 1988	BO	0.84	0.55	0.65	0.47
Saguenay, 1988	BO	1.03	0.78	0.77	0.54
Saguenay, 1988	BO	0.94	0.79	0.72	0.53
Coyote Lake, 1979	LA	1.94	1.44	1.39	0.92
Coyote Lake, 1979	LA	0.70	0.57	0.42	0.39
Imperial Valley, 1979	LA	0.39	0.32	0.36	0.30
Imperial Valley, 1979	LA	0.48	0.22	0.15	0.15
Kern, 1952	LA	0.55	0.65	0.42	0.36
Kern, 1952	LA	0.72	0.50	0.67	0.32
Landers, 1992	LA	0.83	0.63	0.53	0.37
Landers, 1992	LA	0.45	0.31	0.35	0.30
Morgan Hill, 1984	LA	0.66	0.56	0.66	0.53
Morgan Hill, 1984	LA	0.96	0.69	0.58	0.47
Parkfield, 1966, Cholame 5W	LA	0.87	0.56	0.61	0.48
Parkfield, 1966, Cholame 5W	LA	0.61	0.46	0.44	0.35
Parkfield, 1966, Cholame 8W	LA	0.95	0.63	0.75	0.57
Parkfield, 1966, Cholame 8W	LA	0.82	0.61	0.66	0.52
North Palm Springs, 1986	LA	1.09	0.84	0.84	0.68
North Palm Springs, 1986	LA	1.21	0.73	0.53	0.52
San Fernando, 1971	LA	0.62	0.45	0.42	0.30
San Fernando, 1971	LA	0.58	0.51	0.27	0.20
Whittier, 1987	LA	2.14	1.73	2.45	1.54
Whittier, 1987	LA	1.73	1.27	1.66	0.86

Table A.2 Peak Interstory drift ratio (%) for 10-story buildings

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Imperial Valley, 1940, El Centro	LA	1.00	1.58	1.55	1.92
Imperial Valley, 1940, El Centro	LA	1.03	0.84	0.83	2.20
Imperial Valley, 1979, Array #05	LA	0.85	2.72	0.88	0.88
Imperial Valley, 1979, Array #05	LA	0.47	0.60	0.45	0.44
Imperial Valley, 1979, Array #06	LA	0.81	3.19	0.79	0.78
Imperial Valley, 1979, Array #06	LA	0.37	0.61	0.38	0.27
Landers, 1992, Barstow	LA	0.84	1.14	0.98	1.24
Landers, 1992, Barstow	LA	0.96	0.95	0.97	1.12
Landers, 1992, Yermo	LA	2.84	1.58	2.40	2.41
Landers, 1992, Yermo	LA	1.32	1.05	1.45	1.57
Loma Prieta, 1989, Gilroy	LA	2.15	2.23	2.04	2.85
Loma Prieta, 1989, Gilroy	LA	0.58	0.64	0.45	1.31
Northridge, 1994, Newhall	LA	1.46	1.43	1.39	1.71
Northridge, 1994, Newhall	LA	1.43	1.35	1.59	2.07
Northridge, 1994, Rinaldi RS	LA	1.28	1.11	1.38	1.24
Northridge, 1994, Rinaldi RS	LA	1.81	1.72	2.68	2.11
Northridge, 1994, Sylmar	LA	1.04	2.41	1.44	1.92
Northridge, 1994, Sylmar	LA	2.44	2.41	2.59	2.62
North Palm Springs, 1986	LA	1.02	0.83	0.70	0.78
North Palm Springs, 1986	LA	1.96	1.77	2.25	1.97
Long Beach, Vernon CMD Bldg.	SE	0.40	0.55	0.43	0.27
Long Beach, Vernon CMD Bldg.	SE	0.21	0.26	0.27	0.16
Morgan Hill, 1984, Gilroy	SE	0.53	0.84	0.63	0.68
Morgan Hill, 1984, Gilroy	SE	0.57	0.99	0.56	0.94
West. Washington, Olympia, 1949	SE	0.58	0.73	0.59	0.47
West. Washington, Olympia, 1949	SE	0.62	0.56	0.66	0.97
West. Washington, Seattle Army B., 1949	SE	0.91	0.61	0.87	0.58
West. Washington, Seattle Army B., 1949	SE	0.89	0.60	0.72	0.68
North Palm Springs, 1986	SE	0.51	0.47	0.40	0.38
North Palm Springs, 1986	SE	0.89	1.08	1.19	1.34
Puget Sound, Wa., Olympia, 1949	SE	0.66	1.06	0.92	0.97
Puget Sound, Wa., Olympia, 1949	SE	0.68	0.83	0.70	0.95
Puget Sound, Wa., Federal OFC B., 1949	SE	0.80	0.62	0.75	0.74
Puget Sound, Wa., Federal OFC B., 1949	SE	0.46	0.34	0.44	1.34
Eastern Wa., Tacoma County, 1949	SE	0.70	0.48	0.70	0.50
Eastern Wa., Tacoma County, 1949	SE	0.76	0.86	0.75	0.57
Llolleo, Chile 1985	SE	0.94	0.86	0.76	0.88
Llolleo, Chile 1985	SE	0.87	0.74	1.18	1.12
Vinadel Mar, Chile, 1985	SE	1.33	1.05	1.17	1.04%

Table A.2 Peak Interstory drift ratio (%) for 10-story buildings (Continued)

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Vinadel Mar, Chile, 1985	SE	0.91	0.57	0.55	0.70
Simulation, hanging wall	BO	0.18	0.17	0.15	0.11
Simulation, hanging wall	BO	0.14	0.13	0.11	0.08
Simulation, foot wall	BO	0.16	0.16	0.12	0.13
Simulation, foot wall	BO	0.23	0.22	0.16	0.12
New Hampshire, 1982	BO	0.19	0.18	0.16	0.14
New Hampshire, 1982	BO	0.17	0.14	0.15	0.16
Nahanni, 1985	BO	0.05	0.08	0.06	0.04
Nahanni, 1985	BO	0.10	0.13	0.10	0.06
Nahanni, 1985	BO	0.09	0.11	0.09	0.07
Nahanni, 1985	BO	0.08	0.07	0.06	0.05
Nahanni, 1985	BO	0.10	0.06	0.06	0.05
Nahanni, 1985	BO	0.06	0.06	0.06	0.05
Saguenay, 1988	BO	0.10	0.15	0.11	0.11
Saguenay, 1988	BO	0.10	0.11	0.12	0.13
Saguenay, 1988	BO	0.28	0.23	0.21	0.16
Saguenay, 1988	BO	0.07	0.09	0.08	0.06
Saguenay, 1988	BO	0.19	0.20	0.17	0.12
Saguenay, 1988	BO	0.14	0.15	0.12	0.12
Saguenay, 1988	BO	0.20	0.20	0.16	0.11
Saguenay, 1988	BO	0.25	0.22	0.18	0.16
1995 Kobe	LA	2.97	3.35	3.65	4.02
1995 Kobe	LA	1.90	2.41	1.97	3.27
1989 Loma Prieta	LA	1.15	1.75	1.55	1.43
1989 Loma Prieta	LA	4.14	5.83	4.87	3.98
1994 Northridge	LA	2.22	1.94	2.29	2.51
1994 Northridge	LA	3.27	2.43	3.66	4.48
1994 Northridge	LA	3.91	4.53	5.13	3.79
1994 Northridge	LA	3.56	3.65	5.22	5.84
1974 Tabas	LA	1.32	1.73	1.52	1.68
1974 Tabas	LA	2.63	3.33	2.60	2.55
Elysian Park (simulated)	LA	2.80	1.75	3.21	3.61
Elysian Park (simulated)	LA	3.36	2.31	2.71	3.57
Elysian Park (simulated)	LA	3.77	3.95	3.80	3.96
Elysian Park (simulated)	LA	3.26	3.91	3.93	4.04
Elysian Park (simulated)	LA	9.43	5.13	9.02	9.10
Elysian Park (simulated)	LA	8.93	6.03	8.28	11.04
Palos Verdes (simulated)	LA	5.00	4.75	5.19	4.78
Palos Verdes (simulated)	LA	6.43	6.92	6.34	8.34

Table A.2 Peak Interstory drift ratio (%) for 10-story buildings (Continued)

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Palos Verdes (simulated)	LA	2.15	1.83	2.07	2.33
Palos Verdes (simulated)	LA	7.39	5.52	5.04	5.93
1992 Mendocino	SE	3.52	2.95	3.21	3.43
1992 Mendocino	SE	1.05	1.68	0.92	1.97
1992 Erzincan	SE	1.61	2.51	2.19	1.65
1992 Erzincan	SE	3.27	2.88	3.33	2.90
1949 Olympia	SE	1.56	2.66	1.69	2.03
1949 Olympia	SE	1.18	1.62	1.48	1.92
1965 Seattle	SE	2.87	2.63	3.02	3.72
1965 Seattle	SE	1.83	1.78	2.01	1.90
1985 Valpariso	SE	2.20	1.92	1.72	1.88
1985 Valpariso	SE	2.06	1.60	1.91	3.14
1985 Valpariso	SE	3.26	2.28	2.31	2.49
1985 Valpariso	SE	1.40	1.41	1.16	1.20
Deep Interplate (simulation)	SE	3.43	1.85	3.15	1.80
Deep Interplate (simulation)	SE	1.32	0.98	1.35	1.47
1978 Miyagi-oki	SE	1.87	1.24	1.46	1.03
1978 Miyagi-oki	SE	3.72	2.31	3.38	2.48
Shallow Interplate (simulation)	SE	2.25	1.61	2.12	1.98
Shallow Interplate (simulation)	SE	1.84	1.90	2.66	2.74
Shallow Interplate (simulation)	SE	1.20	1.11	1.74	1.84
Shallow Interplate (simulation)	SE	1.65	1.79	2.66	2.87
simulation, foot wall	BO	0.48	0.72	0.74	0.41
simulation, foot wall	BO	0.34	0.49	0.40	0.26
simulation, foot wall	BO	0.30	0.35	0.24	0.15
simulation, foot wall	BO	0.36	0.40	0.39	0.30
simulation, foot wall	BO	0.39	0.39	0.39	0.23
simulation, foot wall	BO	0.40	0.43	0.34	0.21
Nahanni, 1985 Station 1	BO	0.14	0.19	0.18	0.10
Nahanni, 1985 Station 1	BO	0.29	0.35	0.25	0.13
Nahanni, 1985 Station 2	BO	0.25	0.30	0.29	0.18
Nahanni, 1985 Station 2	BO	0.24	0.17	0.15	0.10
Nahanni, 1985 Station 3	BO	0.27	0.19	0.14	0.11
Nahanni, 1985 Station 3	BO	0.18	0.15	0.13	0.11
Saguenay, 1988	BO	0.27	0.30	0.26	0.25
Saguenay, 1988	BO	0.28	0.32	0.29	0.30
Saguenay, 1988	BO	0.90	0.67	0.62	0.71
Saguenay, 1988	BO	0.19	0.23	0.18	0.14
Saguenay, 1988	BO	0.56	0.60	0.49	0.35

Table A.2 Peak Interstory drift ratio (%) for 10-story buildings (Continued)

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Saguenay, 1988	BO	0.39	0.37	0.48	0.28
Saguenay, 1988	BO	0.57	0.44	0.47	0.31
Saguenay, 1988	BO	0.77	0.56	0.61	0.61
Coyote Lake, 1979	LA	0.91	1.27	1.24	1.07
Coyote Lake, 1979	LA	0.36	0.52	0.32	0.32
Imperial Valley, 1979	LA	0.33	0.33	0.28	0.18
Imperial Valley, 1979	LA	0.18	0.25	0.21	0.13
Kern, 1952	LA	0.42	0.42	0.41	1.01
Kern, 1952	LA	0.35	0.40	0.44	1.10
Landers, 1992	LA	0.39	0.32	0.28	0.22
Landers, 1992	LA	0.23	0.31	0.30	0.31
Morgan Hill, 1984	LA	0.43	0.72	0.58	0.64
Morgan Hill, 1984	LA	0.46	0.76	0.48	0.51
Parkfield, 1966, Cholame 5W	LA	0.63	0.41	0.53	0.53
Parkfield, 1966, Cholame 5W	LA	0.43	0.39	0.42	0.47
Parkfield, 1966, Cholame 8W	LA	0.67	0.48	0.53	0.48
Parkfield, 1966, Cholame 8W	LA	0.52	0.64	0.47	0.45
North Palm Springs, 1986	LA	0.60	0.64	0.59	0.68
North Palm Springs, 1986	LA	0.61	0.57	0.62	0.54
San Fernando, 1971	LA	0.34	0.32	0.25	0.70
San Fernando, 1971	LA	0.40	0.67	0.85	1.35
Whittier, 1987	LA	1.39	0.83	1.25	1.11
Whittier, 1987	LA	1.19	0.95	0.90	0.90

Table A.3 Peak floor acceleration (g) for 6-story buildings

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
Imperial Valley, 1940, El Centro	LA	0.7592	1.1430	1.2951	-
Imperial Valley, 1940, El Centro	LA	1.1735	1.3900	1.7970	-
Imperial Valley, 1979, Array #05	LA	0.7373	0.8817	1.0345	-
Imperial Valley, 1979, Array #05	LA	0.8653	0.8600	1.0716	-
Imperial Valley, 1979, Array #06	LA	0.5426	0.6495	0.8348	-
Imperial Valley, 1979, Array #06	LA	0.5711	0.8481	0.9425	-
Landers, 1992, Barstow	LA	0.7532	0.8409	1.2174	-
Landers, 1992, Barstow	LA	0.6389	1.0881	1.4409	-
Landers, 1992, Yermo	LA	0.8544	1.0066	1.2539	-
Landers, 1992, Yermo	LA	0.7396	1.0647	1.2648	-
Loma Prieta, 1989, Gilroy	LA	0.8968	1.3425	1.5522	-
Loma Prieta, 1989, Gilroy	LA	1.7853	1.6387	1.8998	-
Northridge, 1994, Newhall	LA	1.0226	1.6611	1.4655	-
Northridge, 1994, Newhall	LA	0.7827	1.5136	1.5513	-
Northridge, 1994, Rinaldi RS	LA	1.0533	1.1577	1.5196	-
Northridge, 1994, Rinaldi RS	LA	0.8251	1.8584	1.1845	-
Northridge, 1994, Sylmar	LA	0.6105	0.9705	1.1292	-
Northridge, 1994, Sylmar	LA	1.3205	1.2380	1.4126	-
North Palm Springs, 1986	LA	1.3723	2.0920	1.9088	-
North Palm Springs, 1986	LA	1.4366	2.2016	1.8068	-
Long Beach, Vernon CMD Bldg.	SE	0.4025	0.4358	0.7627	-
Long Beach, Vernon CMD Bldg.	SE	0.4903	0.5413	0.8641	-
Morgan Hill, 1984, Gilroy	SE	0.7479	1.1386	1.0580	-
Morgan Hill, 1984, Gilroy	SE	0.9487	1.0891	1.4975	-
West. Washington, Olympia, 1949	SE	0.7899	0.7347	1.0641	-
West. Washington, Olympia, 1949	SE	0.7642	0.9916	1.1718	-
West. Washington, Seattle Army B., 1949	SE	0.6160	1.0996	1.0460	-
West. Washington, Seattle Army B., 1949	SE	0.6415	1.2395	1.2294	-
North Palm Springs, 1986	SE	0.9952	1.2875	1.0861	-
North Palm Springs, 1986	SE	0.8257	1.3083	1.2741	-
Puget Sound, Wa., Olympia, 1949	SE	1.1691	1.8598	1.5485	-
Puget Sound, Wa., Olympia, 1949	SE	1.0704	1.2593	1.3144	-
Puget Sound, Wa., Federal OFC B., 1949	SE	0.9599	1.0465	1.0694	-
Puget Sound, Wa., Federal OFC B., 1949	SE	0.6487	1.0116	1.0840	-
Eastern Wa., Tacoma County, 1949	SE	0.6721	0.7112	1.0614	-
Eastern Wa., Tacoma County, 1949	SE	0.8240	1.4963	1.1575	-
Llolleo, Chile 1985	SE	1.2264	1.8013	1.3758	-
Llolleo, Chile 1985	SE	1.4405	1.4597	1.6551	-
Vinadel Mar, Chile, 1985	SE	0.9915	1.3642	1.2409	-

Table A.3 Peak floor acceleration (g) for 6-story buildings (Continued.)

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
Vinadel Mar, Chile, 1985	SE	0.7103	0.9356	1.2227	-
Simulation, hanging wall	BO	0.4240	0.4417	0.3228	-
Simulation, hanging wall	BO	0.3240	0.4174	0.3937	-
Simulation, foot wall	BO	0.3636	0.4691	0.5312	-
Simulation, foot wall	BO	0.4159	0.5370	0.8419	-
New Hampshire, 1982	BO	0.6389	0.9018	1.0650	-
New Hampshire, 1982	BO	0.5079	0.5776	0.6005	-
Nahanni, 1985	BO	0.2018	0.3291	0.3680	-
Nahanni, 1985	BO	0.1715	0.2950	0.2496	-
Nahanni, 1985	BO	0.2067	0.2512	0.3053	-
Nahanni, 1985	BO	0.1225	0.3800	0.4178	-
Nahanni, 1985	BO	0.2011	0.2618	0.2269	-
Nahanni, 1985	BO	0.1644	0.2879	0.2696	-
Saguenay, 1988	BO	0.4098	0.4970	0.6341	-
Saguenay, 1988	BO	0.5741	0.6868	0.7846	-
Saguenay, 1988	BO	0.8373	0.8791	1.0442	-
Saguenay, 1988	BO	0.3163	0.5149	0.5743	-
Saguenay, 1988	BO	0.4952	0.6987	0.9002	-
Saguenay, 1988	BO	0.6481	0.5673	0.7727	-
Saguenay, 1988	BO	0.3971	0.5567	0.7312	-
Saguenay, 1988	BO	0.5317	0.5049	0.7971	-
1995 Kobe	LA	1.4502	1.9211	1.5786	-
1995 Kobe	LA	1.3746	1.2067	1.5453	-
1989 Loma Prieta	LA	0.7258	0.8211	1.1935	-
1989 Loma Prieta	LA	1.0656	1.7122	1.2622	-
1994 Northridge	LA	0.9358	1.7397	1.6295	-
1994 Northridge	LA	1.1146	2.0334	0.0000	-
1994 Northridge	LA	0.8937	1.0991	1.5832	-
1994 Northridge	LA	1.5997	-	-	-
1974 Tabas	LA	1.2912	1.9402	1.5505	-
1974 Tabas	LA	1.5928	2.4932	1.7281	-
Elysian Park (simulated)	LA	1.1646	8.6041	1.7177	-
Elysian Park (simulated)	LA	1.6129	2.5287	1.7343	-
Elysian Park (simulated)	LA	1.0556	2.0768	1.5557	-
Elysian Park (simulated)	LA	1.0463	1.7452	1.5129	-
Elysian Park (simulated)	LA	1.2682	-	-	-
Elysian Park (simulated)	LA	1.1550	-	-	-
Palos Verdes (simulated)	LA	0.8925	1.0578	1.1694	-
Palos Verdes (simulated)	LA	0.7886	1.2732	1.0984	-

Table A.3 Peak floor acceleration (g) for 6-story buildings (Continued.)

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
Palos Verdes (simulated)	LA	0.8029	1.0498	1.1290	-
Palos Verdes (simulated)	LA	1.7054	1.1457	1.0041	-
1992 Mendocino	SE	1.3535	1.1200	1.2029	-
1992 Mendocino	SE	0.9074	0.9026	1.0199	-
1992 Erzincan	SE	0.6207	1.2115	1.1352	-
1992 Erzincan	SE	0.8463	0.9643	1.3991	-
1949 Olympia	SE	0.9952	1.9127	1.7727	-
1949 Olympia	SE	1.2928	1.7975	1.9009	-
1965 Seattle	SE	2.2970	3.3515	3.4522	-
1965 Seattle	SE	1.8103	2.9488	3.0048	-
1985 Valpariso	SE	2.0087	3.7653	3.1820	-
1985 Valpariso	SE	2.3728	3.2848	3.2976	-
1985 Valpariso	SE	2.3847	2.0422	1.7470	-
1985 Valpariso	SE	1.1022	1.4714	1.4360	-
Deep Interplate (simulation)	SE	1.1890	1.8618	1.7458	-
Deep Interplate (simulation)	SE	1.4220	1.5816	1.6610	-
1978 Miyagi-oki	SE	1.0714	1.5242	1.4178	-
1978 Miyagi-oki	SE	0.7811	1.5877	1.6765	-
Shallow Interplate (simulation)	SE	1.5275	1.2205	1.3509	-
Shallow Interplate (simulation)	SE	0.8912	1.4885	1.7316	-
Shallow Interplate (simulation)	SE	0.6990	1.4106	1.6381	-
Shallow Interplate (simulation)	SE	1.0365	1.5130	1.8331	-
simulation, foot wall	BO	0.5169	0.7271	0.7006	-
simulation, foot wall	BO	0.7351	0.7347	0.8470	-
simulation, foot wall	BO	0.6045	0.6395	0.6733	-
simulation, foot wall	BO	0.4974	0.5936	0.8521	-
simulation, foot wall	BO	0.5823	0.6115	0.7442	-
simulation, foot wall	BO	0.6670	0.7390	1.0080	-
Nahanni, 1985 Station 1	BO	0.5073	0.5425	0.7442	-
Nahanni, 1985 Station 1	BO	0.4176	0.5068	0.6902	-
Nahanni, 1985 Station 2	BO	0.4625	0.6221	0.8237	-
Nahanni, 1985 Station 2	BO	0.3366	0.5206	0.7918	-
Nahanni, 1985 Station 3	BO	0.5699	0.5716	0.6395	-
Nahanni, 1985 Station 3	BO	0.4730	0.6377	0.8003	-
Saguenay, 1988	BO	1.1153	1.1414	1.1812	-
Saguenay, 1988	BO	1.0167	1.5997	1.5722	-
Saguenay, 1988	BO	1.6474	2.3053	2.4913	-
Saguenay, 1988	BO	0.8610	1.4554	1.5088	-
Saguenay, 1988	BO	0.9015	1.1710	1.7368	-

Table A.3 Peak floor acceleration (g) for 6-story buildings (Continued)

Record	Site	CBF	Dual4-2	Dual3-3	Dual2-4
Saguenay, 1988	BO	1.0873	1.4531	1.4875	-
Saguenay, 1988	BO	0.9623	1.1962	1.0694	-
Saguenay, 1988	BO	0.8837	1.4681	1.1532	-
Coyote Lake, 1979	LA	1.1361	1.4812	1.3970	-
Coyote Lake, 1979	LA	0.6542	0.9191	0.9719	-
Imperial Valley, 1979	LA	0.3597	0.4138	0.6608	-
Imperial Valley, 1979	LA	0.4327	0.5898	0.5692	-
Kern, 1952	LA	0.5786	0.6925	0.7929	-
Kern, 1952	LA	0.5133	0.7029	0.9972	-
Landers, 1992	LA	0.6361	0.7579	1.0449	-
Landers, 1992	LA	0.7614	0.9270	1.4124	-
Morgan Hill, 1984	LA	0.6472	0.8643	1.1263	-
Morgan Hill, 1984	LA	0.8159	0.9664	1.2450	-
Parkfield, 1966, Cholame 5W	LA	1.1549	1.3879	1.3714	-
Parkfield, 1966, Cholame 5W	LA	0.9539	1.0146	1.3282	-
Parkfield, 1966, Cholame 8W	LA	0.8745	1.6940	1.7911	-
Parkfield, 1966, Cholame 8W	LA	1.3278	1.4686	1.4641	-
North Palm Springs, 1986	LA	0.8310	0.9625	1.1110	-
North Palm Springs, 1986	LA	0.8068	0.9571	1.1152	-
San Fernando, 1971	LA	0.5756	0.8577	0.9008	-
San Fernando, 1971	LA	0.5917	0.8176	1.0410	-
Whittier, 1987	LA	1.1955	1.4786	1.6175	-
Whittier, 1987	LA	1.1044	1.5182	1.7335	-

Table A.4 Peak floor acceleration (g) for 10-story buildings

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Imperial Valley, 1940, El Centro	LA	0.7816	1.2827	1.0568	-
Imperial Valley, 1940, El Centro	LA	1.1873	1.2767	1.4339	-
Imperial Valley, 1979, Array #05	LA	0.5905	0.7813	0.8773	-
Imperial Valley, 1979, Array #05	LA	0.5996	0.9291	0.9792	-
Imperial Valley, 1979, Array #06	LA	0.5845	0.7829	0.7456	-
Imperial Valley, 1979, Array #06	LA	0.4556	0.7619	0.5780	-
Landers, 1992, Barstow	LA	0.6618	0.9326	0.8983	-
Landers, 1992, Barstow	LA	0.7455	0.9221	0.9489	-
Landers, 1992, Yermo	LA	0.8463	0.8611	1.2664	-
Landers, 1992, Yermo	LA	0.8121	0.9348	1.0380	-
Loma Prieta, 1989, Gilroy	LA	1.0102	1.5490	1.4858	-
Loma Prieta, 1989, Gilroy	LA	1.3682	2.2720	2.2142	-
Northridge, 1994, Newhall	LA	1.0486	1.7744	1.1856	-
Northridge, 1994, Newhall	LA	0.8201	1.1969	1.0096	-
Northridge, 1994, Rinaldi RS	LA	0.9158	1.0119	1.3479	-
Northridge, 1994, Rinaldi RS	LA	0.8685	1.0319	1.2508	-
Northridge, 1994, Sylmar	LA	0.7696	1.0987	0.8900	-
Northridge, 1994, Sylmar	LA	1.3349	1.6239	1.7591	-
North Palm Springs, 1986	LA	1.3185	2.2944	2.1926	-
North Palm Springs, 1986	LA	1.2383	2.0186	2.3600	-
Long Beach, Vernon CMD Bldg.	SE	0.4183	0.6676	0.5249	-
Long Beach, Vernon CMD Bldg.	SE	0.2641	0.4291	0.5093	-
Morgan Hill, 1984, Gilroy	SE	0.5484	0.8875	0.9465	-
Morgan Hill, 1984, Gilroy	SE	0.8027	1.0188	1.2117	-
West. Washington, Olympia, 1949	SE	0.5816	0.8542	0.9311	-
West. Washington, Olympia, 1949	SE	0.5793	1.0748	0.8710	-
West. Washington, Seattle Army B., 1949	SE	0.6867	0.8598	0.9355	-
West. Washington, Seattle Army B., 1949	SE	0.7486	0.9220	0.9466	-
North Palm Springs, 1986	SE	0.9102	1.4493	1.6141	-
North Palm Springs, 1986	SE	0.9174	1.5566	1.4989	-
Puget Sound, Wa., Olympia, 1949	SE	0.9725	1.5109	1.6963	-
Puget Sound, Wa., Olympia, 1949	SE	0.8778	1.3829	1.1341	-
Puget Sound, Wa., Federal OFC B., 1949	SE	0.7134	1.1806	0.8696	-
Puget Sound, Wa., Federal OFC B., 1949	SE	0.5980	0.9034	1.0605	-
Eastern Wa., Tacoma County, 1949	SE	0.6371	0.9941	0.8655	-
Eastern Wa., Tacoma County, 1949	SE	0.7126	1.0281	1.1805	-
Llolleo, Chile 1985	SE	1.0191	1.7858	1.4231	-
Llolleo, Chile 1985	SE	0.8162	1.2615	1.1373	-
Vinadel Mar, Chile, 1985	SE	1.0228	1.1852	1.1698	-

Table A.4 Peak floor acceleration (g) for 10-story buildings (Continued)

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Vinadel Mar, Chile, 1985	SE	0.7258	0.9026	0.8742	-
Simulation, hanging wall	BO	0.2118	0.3280	0.3822	-
Simulation, hanging wall	BO	0.1661	0.3297	0.3515	-
Simulation, foot wall	BO	0.2086	0.4481	0.5242	-
Simulation, foot wall	BO	0.2580	0.4848	0.4584	-
New Hampshire, 1982	BO	0.5829	0.8643	0.9367	-
New Hampshire, 1982	BO	0.3675	0.5725	0.8641	-
Nahanni, 1985	BO	0.1055	0.2631	0.3069	-
Nahanni, 1985	BO	0.1190	0.2776	0.3442	-
Nahanni, 1985	BO	0.1146	0.2462	0.2076	-
Nahanni, 1985	BO	0.1072	0.2389	0.2007	-
Nahanni, 1985	BO	0.1440	0.2846	0.2518	-
Nahanni, 1985	BO	0.1163	0.2339	0.2523	-
Saguenay, 1988	BO	0.2168	0.4939	0.5282	-
Saguenay, 1988	BO	0.2681	0.6185	0.7117	-
Saguenay, 1988	BO	0.6003	1.0316	1.0811	-
Saguenay, 1988	BO	0.2655	0.4593	0.4895	-
Saguenay, 1988	BO	0.2420	0.6232	0.5010	-
Saguenay, 1988	BO	0.2682	0.6658	0.7236	-
Saguenay, 1988	BO	0.2631	0.4476	0.5141	-
Saguenay, 1988	BO	0.3503	0.6343	0.5514	-
1995 Kobe	LA	1.0139	1.9910	1.6663	-
1995 Kobe	LA	1.0556	1.5181	1.2557	-
1989 Loma Prieta	LA	0.7666	0.9827	0.9440	-
1989 Loma Prieta	LA	0.8505	0.9620	1.1655	-
1994 Northridge	LA	1.1830	1.8239	1.8121	-
1994 Northridge	LA	1.2081	1.6922	1.9383	-
1994 Northridge	LA	0.8581	1.2991	1.3300	-
1994 Northridge	LA	1.9019	-	-	-
1974 Tabas	LA	1.2710	1.9630	1.6926	-
1974 Tabas	LA	1.6632	2.1078	2.0663	-
Elysian Park (simulated)	LA	1.3724	2.0982	1.7055	-
Elysian Park (simulated)	LA	1.2346	1.6561	1.8203	-
Elysian Park (simulated)	LA	0.8576	1.5564	1.4615	-
Elysian Park (simulated)	LA	0.9266	1.5046	1.9169	-
Elysian Park (simulated)	LA	1.6252	1.8155	-	-
Elysian Park (simulated)	LA	0.9823	1.4190	-	-
Palos Verdes (simulated)	LA	1.0033	1.1839	1.2539	-
Palos Verdes (simulated)	LA	1.0687	1.3148	-	-

Table A.4 Peak floor acceleration (g) for 10-story buildings (Continued)

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Palos Verdes (simulated)	LA	0.8288	0.9644	1.0676	-
Palos Verdes (simulated)	LA	0.8308	1.0265	1.4213	-
1992 Mendocino	SE	1.0454	1.1127	1.3265	-
1992 Mendocino	SE	0.7947	0.8595	0.8772	-
1992 Erzincan	SE	0.7649	0.9141	0.9144	-
1992 Erzincan	SE	0.8747	0.8946	0.9872	-
1949 Olympia	SE	0.9787	1.7302	1.6822	-
1949 Olympia	SE	1.2444	1.7488	1.5781	-
1965 Seattle	SE	1.9852	3.3526	3.4501	-
1965 Seattle	SE	1.6838	2.4414	2.3350	-
1985 Valpariso	SE	1.7486	3.3686	2.4516	-
1985 Valpariso	SE	1.7697	3.0543	2.5947	-
1985 Valpariso	SE	1.5372	1.9906	1.7187	-
1985 Valpariso	SE	1.1698	1.3954	1.5187	-
Deep Interplate (simulation)	SE	0.9685	1.8292	1.5181	-
Deep Interplate (simulation)	SE	1.2648	1.4227	1.6287	-
1978 Miyagi-oki	SE	0.9676	1.1257	1.1840	-
1978 Miyagi-oki	SE	1.2391	1.0862	1.0879	-
Shallow Interplate (simulation)	SE	0.7603	1.0974	1.2829	-
Shallow Interplate (simulation)	SE	0.9557	1.5069	1.1938	-
Shallow Interplate (simulation)	SE	0.9132	1.2900	1.2798	-
Shallow Interplate (simulation)	SE	0.9346	1.4702	1.4258	-
simulation, foot wall	BO	0.5268	0.7873	0.8695	-
simulation, foot wall	BO	0.5508	0.7842	0.8639	-
simulation, foot wall	BO	0.4492	0.8013	0.6770	-
simulation, foot wall	BO	0.4482	0.6036	0.8104	-
simulation, foot wall	BO	0.5151	0.7329	0.6713	-
simulation, foot wall	BO	0.4982	0.7236	0.7332	-
Nahanni, 1985 Station 1	BO	0.3018	0.6006	0.6528	-
Nahanni, 1985 Station 1	BO	0.3371	0.6920	0.5354	-
Nahanni, 1985 Station 2	BO	0.3275	0.4855	0.6497	-
Nahanni, 1985 Station 2	BO	0.3060	0.3938	0.4653	-
Nahanni, 1985 Station 3	BO	0.4110	0.5583	0.6451	-
Nahanni, 1985 Station 3	BO	0.3326	0.6099	0.5476	-
Saguenay, 1988	BO	0.6213	1.1247	1.1414	-
Saguenay, 1988	BO	0.7706	1.4076	1.4144	-
Saguenay, 1988	BO	1.6499	2.4636	2.7363	-
Saguenay, 1988	BO	0.7625	1.2283	1.0340	-
Saguenay, 1988	BO	0.6909	1.1415	1.0319	-

Table A.4 Peak floor acceleration (g) for 10-story buildings (Continued)

Record	Site	CBF	Dual7-3	Dual5-5	Dual4-6
Saguenay, 1988	BO	0.7669	1.5144	1.5810	-
Saguenay, 1988	BO	0.7334	1.1025	1.0738	-
Saguenay, 1988	BO	0.9333	1.3843	1.2541	-
Coyote Lake, 1979	LA	0.8479	1.1324	1.5059	-
Coyote Lake, 1979	LA	0.5051	1.0357	0.9565	-
Imperial Valley, 1979	LA	0.3482	0.3672	0.4725	-
Imperial Valley, 1979	LA	0.2180	0.4023	0.5432	-
Kern, 1952	LA	0.4360	0.5429	0.6110	-
Kern, 1952	LA	0.3766	0.6406	0.8257	-
Landers, 1992	LA	0.5096	0.8016	0.8458	-
Landers, 1992	LA	0.4968	0.7715	0.8707	-
Morgan Hill, 1984	LA	0.4620	0.7167	0.8253	-
Morgan Hill, 1984	LA	0.6612	1.1312	1.0479	-
Parkfield, 1966, Cholame 5W	LA	1.0817	1.4451	1.3836	-
Parkfield, 1966, Cholame 5W	LA	0.7999	1.0577	1.1380	-
Parkfield, 1966, Cholame 8W	LA	0.9631	1.3190	1.5888	-
Parkfield, 1966, Cholame 8W	LA	1.0885	1.8197	1.4800	-
North Palm Springs, 1986	LA	0.8772	1.0914	1.4033	-
North Palm Springs, 1986	LA	0.6477	0.8972	0.8654	-
San Fernando, 1971	LA	0.4341	0.6334	0.6628	-
San Fernando, 1971	LA	0.5079	0.5858	0.7853	-
Whittier, 1987	LA	1.0331	1.8969	1.1580	-
Whittier, 1987	LA	0.9175	1.0457	1.2477	-

APPENDIX B

WEIGHT CALCULATION OF THE STRUCTURES

A-1 Introduction

One of the most important and time consuming steps in the cost analysis in this study is finding the initial cost of the building. To have a good estimation of the initial cost, structural members' weight should be calculated. To calculate the structural weight, beams, columns and braces are considered in the CBF system and beams, columns, adjacent gravity columns, and PT bar are considered in the dual system. The details of calculations are presented as follows for 6-story and 10-story buildings.

A-2 6-story buildings

A-2-1 6-story CBF

Total weight of the 6-story CBF steel structure:

$$W_{Tot} = 1271.865 \text{ kips} \equiv 576918 \text{ kg} \quad (A-1)$$

$$Area_{Tot} = 180 \times 180 \times 6 = 194400 \text{ ft}^2 \quad (A-2)$$

$$W_{Tot} / Area_{Tot} = 6.5425 \frac{lb}{ft^2} \Rightarrow \begin{cases} w_{gr} = 4.761 \frac{lb}{ft^2} \\ w_{CBF} = 1.782 \frac{lb}{ft^2} \end{cases} \quad (A-3)$$

Table A-1 Separated weights of CBF6			
	Total Weight (Kips)	Gravity weight (Kips)	CBF weight (Kips)
W_1	267.24	166.646	100.594
W_2	236.956	159.88	77.076
W_3	206.187	151.511	55.445
W_4	201.828	151.51	50.37
W_5	179.829	147.948	31.881
W_6	179.825	147.95	31.875
W_{Tot}	1271.865	925.445	347.241

A-2-1 Dual4-2

$$W_{D42} \text{ (From excel file for one frame)} = 101.136 \text{ Kips} \quad (A-4)$$

$$W_{D42\_adj\_gr} = \left(\frac{(145 * 30) + (145 * 25) + (132 * 25) + (132 * 25)}{1000} \right) = 14.725 \text{ Kips} \quad (A-5)$$

$$W_{D42\_PT\_bar} = \left(\left(\left((23.7 / 144) * (52.5 + 1) \right) * 490 \right) / 1000 \right) = 4.315 \text{ Kips} \quad (A-6)$$

$$W_{D42\_eq} \text{ (For one frame)} = 120.176 \text{ Kips} \quad (A-7)$$

$$W_{D42\_eq} \text{ (For four frames)} = 480.704 \text{ Kips} \rightarrow 2.473 \text{ Kips / ft}^2 \quad (A-8)$$

$$W_{D42\_gr} \text{ (same as CBF6)} = 925.445 \text{ Kips} \rightarrow 4.761 \text{ Kips / ft}^2 \quad (A-9)$$

$$W_{D42\_tot} = 1406.149 \text{Kips} \rightarrow 7.233 \text{ Kips / ft}^2 \quad (\text{A-10})$$

Table A-2 Separated weights of Dual4-2			
	Total Weight (Kips)	Gravity weight (Kips)	D4-2 weight (Kips)
W_1	287.761	166.646	121.119*
W_2	250.775	159.88	90.895
W_3	243.346	151.511	91.835
W_4	255.745	151.51	104.235
W_5	185.228	147.948	37.28
W_6	182.71	147.95	34.76
W_{Tot}	1406	925.445	480.124

$$\begin{aligned}
 121.119^* &= 4 \times (24.851 + 4.35 + (4.315 / 4)) \\
 &= 4 * (Frame + adj\_Gravity + (PTbar / 4))
 \end{aligned}$$

A-2-2 Dual3-3

$$W_{D33} (\text{From excel file for one frame}) = 99.286 \text{Kips} \quad (\text{A-11})$$

$$W_{D33\_adj\_gr} = \left(\frac{(145 * 30) + (145 * 25)}{+ (132 * 25)} \right) / 1000 = 11.275 \text{Kips} \quad (\text{A-12})$$

$$W_{D33\_PT\_bar} = (((25.28 / 144) * (40 + 1)) * 490) / 1000 = 3.523 \text{Kips} \quad (\text{A-13})$$

$$W_{D33\_eq} (\text{For one frame}) = 114.084 \text{Kips} \quad (\text{A-14})$$

$$W_{D33\_eq} (\text{For four frames}) = 456.336 \text{Kips} \rightarrow 2.347 \text{ Kips / ft}^2 \quad (\text{A-15})$$

$$W_{D33\_gr} (\text{same as CBF6}) = 925.445 \text{Kips} \rightarrow 4.761 \text{ Kips / ft}^2 \quad (\text{A-16})$$

$$W_{D33\_tot} = 1381.781Kips \rightarrow 7.108 Kips / ft^2 \quad (A-17)$$

Table A-3 Separated weights of Dual3-3			
	Total Weight (Kips)	Gravity weight (Kips)	D3-3 weight (Kips)
W_1	287.761	166.646	129.177*
W_2	250.775	159.88	106.685
W_3	243.346	151.511	104.161
W_4	255.745	151.51	45.44
W_5	185.228	147.948	37.32
W_6	182.71	147.95	33.52
W_{Tot}	1382	925.445	456.303

$$\begin{aligned}
 129.177* &= 4 \times (26.77 + 4.35 + (3.523 / 3)) \\
 &= 4 * (Frame + adj\_Gravity + (PTbar / 3))
 \end{aligned}$$

A-2-3 Dual2-4

$$W_{D24} (\text{From excel file for one frame}) = 91.063Kips \quad (A-18)$$

$$W_{D24\_adj\_gr} = ((132 * 30) + (132 * 25)) / 1000 = 7.26Kips \quad (A-19)$$

$$W_{D24\_PT\_bar} = (((23.58 / 144) * (27.5 + 1)) * 490) / 1000 = 2.287Kips \quad (A-20)$$

$$W_{D24\_eq} (\text{For one frame}) = 100.61Kips \quad (A-21)$$

$$W_{D24\_eq} (\text{For four frames}) = 402.44Kips \rightarrow 2.07 Kips / ft^2 \quad (A-22)$$

$$W_{D24\_gr} (\text{same as CBF6}) = 925.445Kips \rightarrow 4.761 Kips / ft^2 \quad (A-23)$$

$$W_{D24\_tot} = 1327.885 \text{ Kips} \rightarrow 6.831 \text{ Kips} / \text{ft}^2 \quad (\text{A-24})$$

Table A-4 Separated weights of Dual2-4			
	Total Weight (Kips)	Gravity weight (Kips)	D2-4 weight (Kips)
W_1	303.26	166.646	136.614*
W_2	279.214	159.88	119.334
W_3	195.791	151.511	44.28
W_4	192.35	151.51	40.84
W_5	180.508	147.948	32.56
W_6	176.75	147.95	28.8
W_{Tot}	1328	925.445	402.428

$$136.614^* = 4 \times (29.05 + 3.96 + (2.287 / 2))$$

$$= 4 * (Frame + adj\_Gravity + (PTbar / 2))$$

A-3 10-story buildings

A-3-1 10-story CBF

Total weight of the 10-story CBF steel structure:

$$W_{Tot} = 3211.639 \text{ kips} \equiv 1456800 \text{ kg} \quad (\text{A-25})$$

$$Area_{Tot} = 180 \times 180 \times 10 = 324000 \text{ ft}^2 \quad (\text{A-26})$$

$$W_{Tot} / Area_{Tot} = 9.9125 \frac{\text{lb}}{\text{ft}^2} \Rightarrow \begin{cases} w_{gr} = 5.354 \frac{\text{lb}}{\text{ft}^2} \\ w_{CBF} = 4.558 \frac{\text{lb}}{\text{ft}^2} \end{cases} \quad (\text{A-27})$$

Table A-5 Separated weights of CBF10			
	Total Weight (Kips)	Gravity weight (Kips)	CBF weight (Kips)
W_1	471.91	199.974	271.936
W_2	412.299	190.04	222.259
W_3	357.902	181.649	176.253
W_4	353.512	181.645	171.867
W_5	308.487	170.191	138.296
W_6	308.492	170.204	138.288
W_7	264.995	163.567	101.428
W_8	265.01	163.574	101.436
W_9	234.514	156.969	77.545
W_{10}	234.518	156.972	77.546
W_{Tot}	3211.639	1734.785	1476.854

A-3-2 Dual4-6

$$W_{D46} \text{ (From excel file for one frame)} = 202.75 Kips \quad (A-28)$$

$$W_{D46\_adj\_gr} = \left(\frac{(176*30) + (176*25) + (176*25) + (176*25)}{(176*25) + (176*25)} \right) / 1000 = 18.48 Kips \quad (A-29)$$

$$W_{D46\_PT\_bar} = (((39.3/144)*(52.5+1)) * 490) / 1000 = 7.155 Kips \quad (A-30)$$

$$W_{D46\_eq} \text{ (For one frame)} = 228.385 Kips \quad (A-31)$$

$$W_{D46\_eq} \text{ (For six frames)} = 1370.31 Kips \rightarrow 4.229 Kips / ft^2 \quad (A-32)$$

$$W_{D46\_tot} = 3105.095 Kips \rightarrow 9.584 Kips / ft^2 \quad (A-33)$$

$$W_{D42\_tot} = 1406.149 Kips \rightarrow 7.233 Kips / ft^2 \quad (A-34)$$

Table A-6 Separated weights of Dua4-6			
	Total Weight (Kips)	Gravity weight (Kips)	D4-6 weight (Kips)
W_1	475.304	199.974	275.33*
W_2	425.77	190.04	235.73
W_3	403.399	181.649	221.75
W_4	432.555	181.645	250.91
W_5	242.131	170.191	71.94
W_6	244.484	170.204	74.28
W_7	231.127	163.567	67.56
W_8	231.134	163.574	67.56
W_9	211.329	156.969	54.36
W_{10}	207.912	156.972	50.94
W_{Tot}	3105.145	1734.785	1370.36

$$275.33^* = 6 \times (38.82 + 5.28 + (7.155 / 4))$$

$$= 6 * (Frame + adj\_Gravity + (PTbar / 4))$$

A-3-3 Dual5-5

$$W_{D55} (\text{From excel file for one frame}) = 195.891 Kips \quad (A-35)$$

$$W_{D55\_adj\_gr} = \left(\frac{(176*30) + (176*25) + (159*25)}{+(159*25) + (159*25)} \right) / 1000 = 21.605 Kips \quad (A-36)$$

$$W_{D55\_PT\_bar} = (((36.68 / 144) * (65 + 1)) * 490) / 1000 = 8.238 Kips \quad (A-37)$$

$$W_{D55\_eq} (\text{For one frame}) = 225.734 Kips \quad (A-38)$$

$$W_{D55\_eq} (\text{For six frames}) = 1354.404 Kips \rightarrow 4.180 Kips / ft^2 \quad (A-39)$$

$$W_{D55\_gr} \text{ (same as CBF10)} = 1734.785 \text{ Kips} \rightarrow 5.354 \text{ Kips / ft}^2 \quad (\text{A-40})$$

$$W_{D55\_tot} = 3089.189 \text{ Kips} \rightarrow 9.535 \text{ Kips / ft}^2 \quad (\text{A-41})$$

Table A-7 Separated weights of Dual5-5			
	Total Weight (Kips)	Gravity weight (Kips)	D5-5 weight (Kips)
W_1	464.924	199.974	264.95*
W_2	384.43	190.040	194.39
W_3	360.949	181.649	179.30
W_4	368.445	181.645	186.80
W_5	379.971	170.191	209.78
W_6	239.864	170.204	69.66
W_7	233.227	163.567	69.66
W_8	229.154	163.574	65.58
W_9	222.549	156.969	65.58
W_{10}	205.692	156.972	48.72
W_{Tot}	3089.205	1734.785	1354.42

$$\begin{aligned}
 264.95^* &= 6 \times (37.23 + 5.28 + (8.238 / 5)) \\
 &= 6 * (Frame + adj\_Gravity + (PTbar / 5))
 \end{aligned}$$

A-3-4 Dual5-5

$$W_{D73} \text{ (From excel file for one frame)} = 206.65 \text{ Kips} \quad (\text{A-42})$$

$$W_{D73\_adj\_gr} = \left(\frac{(176 * 55) + (159 * 50) + (145 * 50) + (132 * 25)}{1000} \right) = 28.18 \text{ Kips} \quad (\text{A-43})$$

$$W_{D73\_PT\_bar} = \left(\left(\left((31.44 / 144) * (90 + 1) \right) * 490 \right) / 1000 \right) = 9.74 \text{ Kips} \quad (\text{A-44})$$

$$W_{D73\_eq} \text{ (For one frame) } = 244.57 \text{ Kips} \quad (\text{A-45})$$

$$W_{D73\_eq} \text{ (For six frames) } = 1467.42 \text{ Kips} \rightarrow 4.529 \text{ Kips / ft}^2 \quad (\text{A-46})$$

$$W_{D73\_gr} \text{ (same as CBF10) } = 1734.785 \text{ Kips} \rightarrow 5.354 \text{ Kips / ft}^2 \quad (\text{A-47})$$

$$W_{D73\_tot} = 3202.205 \text{ Kips} \rightarrow 9.883 \text{ Kips / ft}^2 \quad (\text{A-48})$$

Table A-8 Separated weights of Dual7-3			
	Total Weight (Kips)	Gravity weight (Kips)	D7-3 weight (Kips)
W_1	433.804	199.974	233.83*
W_2	368.61	190.040	178.57
W_3	353.349	181.649	171.7
W_4	347.645	181.645	166
W_5	325.691	170.191	155.5
W_6	344.964	170.204	174.76
W_7	360.797	163.567	197.23
W_8	229.154	163.574	65.58
W_9	222.549	156.969	65.58
W_{10}	215.532	156.972	58.56
W_{Tot}	3202.095	1734.785	1467.31

$$\begin{aligned}
 233.83^* &= 6 \times (32.3 + 5.28 + (9.74 / 7)) \\
 &= 6 * (Frame + adj\_Gravity + (PTbar / 7))
 \end{aligned}$$