**SEISMIC REHABILITATION OF STEEL MOMENT FRAMES VULNERABLE
TO SOFT-STORY FAILURES THOUGH IMPLEMENTATION OF ROCKING
CORES**

A Thesis

presented to

the Faculty of California Polytechnic State University,

San Luis Obispo

In Partial Fulfillment

of the Requirements for the Degree of

Master of Science in Civil and Environmental Engineering

by

Juan Carlos Sanchez

June 2013

© 2013

Juan Carlos Sanchez

ALL RIGHTS RESERVED

COMMITTEE MEMBERSHIP

TITLE:	Seismic Rehabilitation of Steel Moment Frames Vulnerable to Soft-Story Failures Through Implementation of Rocking Cores
AUTHOR:	Juan Carlos Sanchez
DATE SUBMITTED:	June 2013
COMMITTEE CHAIR:	Bing Qu, Ph.D., Assistant Professor of Civil Engineering
COMMITTEE MEMBER:	Charles Chadwell, Ph.D., Associate Professor of Civil Engineering
COMMITTEE MEMBER:	Rakesh Goel, Ph.D., Professor, Associate Dean of College of Engineering

ABSTRACT

Seismic Rehabilitation of Steel Moment Frames Vulnerable to Soft-Story Failures Through Implementation of Rocking Cores

Juan Carlos Sanchez

During seismic events, inefficient steel moment frame building systems may exhibit soft-story failures. This thesis focuses on development and validation of a seismic retrofit strategy for avoiding soft-story failures in low-rise and mid-rise steel moment frame buildings. The considered retrofit strategy consists of a sufficiently stiff Rocking Core (RC) pinned to the foundation and pin connected to the existing frame. For demonstration purposes, two representative benchmark steel moment frames, which are modified from the three- and nine-story pre-Northridge steel moment frames designed for Los Angeles in the SAC Steel Project, are considered. Finite Element (FE) models of the benchmark buildings are developed with consideration of member yielding, connection rupture, and P-Delta effect, and validated using published results. Eigenvalue analyses are conducted to investigate the effect of the RC on system modal properties. It is found that in general the added RC with practical stiffness value does not significantly change the fundamental period and therefore does not attract excessive earthquake force to the system. In addition, nonlinear static pushover analyses are performed to address the beneficial contribution of the RC to the system under the performance objectives including immediate occupancy, life safety, and collapse prevention. The Monte-Carlo simulation technique is used to generate the random lateral force distribution required in the nonlinear static pushover analysis. It is found that RC works as expected in all considered scenarios and creates more uniform inter-story distribution along the vertical direction

when it is sufficiently stiff. Furthermore, nonlinear dynamic analyses are conducted using three different ground motion suites (including two suites with ground motions having probabilities of exceedance of 2% and 10% in 50 years, and one suite with near-fault ground motions). It is shown that the systems with properly selected RC can achieve the Best Safety Objective defined in FEMA 356 and exhibit collapse prevention performance under near-fault earthquakes.

ACKNOWLEDGMENTS

I would like to sincerely thank Prof. Bing Qu for his time and dedication in this thesis project. It was a pleasure working with him and an absolutely remarkable learning experience.

I would also like to thank Dr. Charles Chadwell and Dr. Rakesh Goel for taking their time in serving as my defense committee.

I am very grateful for the funding provided by the U.S. National Science Foundation under Award Number CMMI-1134953. In addition I would like to thank Maria Manzano and Jackie Duerr for their support, allowing me to work for the MEP tutoring lab, and for the scholarships they granted me.

Finally, I would like to thank my family for their moral support during the many stressful days in my college career and completion of this thesis.

TABLE OF CONTENTS

List of Tables	ix
List of Figures.....	xi
1.0 Introduction.....	1
1.1 Background	1
1.2 Scope/Objective	2
1.3 Thesis Organization.....	2
2.0 Literature Review.....	4
2.1 Gunay et al. (2009).....	4
2.2 Alavi and Krawinkler (2004a and 2004b).....	5
2.3 Wada et al. (2011) and Qu et al. (2012)	5
3.0 Description of Retrofit Strategy and Benchmark Buildings	7
3.1 Retrofit Strategy	7
3.2 Selection of Benchmark Buildings.....	9
4.0 Finite Element Modeling and Validation.....	14
4.1 Modeling of Frame Components.....	18
4.1.1 Selection of Material Model	18
4.1.2 Selection of Elements	19
4.1.3 Consideration of Low-Cycle Fatigue	21
4.2 Modeling of RC.....	23
4.3 Modeling of Links	25
4.4 Consideration of P-Delta Effect	26
4.5 Other Modeling Considerations	28
4.6 Model Validation.....	29
5.0 Seismic Performance Evaluation of Three-Story Building	33
5.1 Impact of RC on Modal Properties	33
5.2 Nonlinear Static Pushover Analysis	37
5.2.1 Random Lateral Force Distribution	38
5.2.2 Target Inter-story Drifts.....	39
5.2.3 Response Quantities of Interest	40
5.2.4 Result Discussion.....	41
5.2.4.1 Discussion of DCF.....	41
5.2.4.2 Discussion of Demands on RC	43
5.2.4.3 Discussion of Demands on RC Links	49
5.3 Nonlinear Dynamic Analysis	51
5.3.1 Selection of Ground Motions	51
5.3.2 Result Discussion	55
5.3.2.1 Discussion on Inter-story Drift	56
5.3.2.2 Demands on RC	60
5.3.2.3 Link Demands	63
5.3.2.4 Impact of Links with Energy Dissipation Capacity	65
5.3.2.5 Residual Drift.....	76
5.4 Demonstration Retrofit using Reinforced Concrete RC	77

6.0 Seismic Performance Evaluation of Nine-Story Building	81
6.1 Impact of RC on Modal Properties	81
6.2 Nonlinear Static Pushover Analysis	85
6.2.1 Result Discussion	86
6.2.1.1 Discussion of DCF.....	86
6.2.1.2 Discussion of Demands on RC	88
6.2.1.3 Discussion of Demands on RC Links	94
6.3 Nonlinear Dynamic Analysis	96
6.3.1 Result Discussion	96
6.3.1.1 Discussion on Inter-story Drift	97
6.3.1.2 Demands on RC	101
6.3.1.3 Link Demands	104
6.3.1.4 Impact of Links with Energy Dissipation Capacity	106
6.3.1.5 Residual Drift.....	116
6.4 Demonstration Retrofit using Reinforced Concrete RC	117
7. Summary of Conclusions and Recommendations for Future Research.....	120
7.1 Conclusion.....	120
7.2 Recommendations for Future Research	121
List of References	123
Appendices.....	125
Appendix A: Response Spectra	125
Appendix B: Maximum Inter-Story Drift Results from Analysis	127

LIST OF TABLES

Table 4-1: Maximum percent drift with and without gravity wall	28
Table 4-2: Vibration period comparison for validation purposes	30
Table 5-1: Period comparison for three levels of stiffness of RC in LA3-RA	34
Table 5-2: Period comparison for three levels of stiffness of RC in LA3-RB	36
Table 5-3: Los Angeles 10% in 50 years ground motions (adapted from SAC Steel Project website).....	53
Table 5-4: Los Angeles 2% in 50 years ground motions (adapted from SAC Steel Project website).....	54
Table 5-5: Near-Fault ground motions (adapted from SAC Steel Project website)	55
Table 5-6: LA3-RB-Comparison of simplified RC and reinforced concrete RC for 10% in 50 years suite	80
Table 5-7: LA3-RB-Comparison of simplified RC and reinforced concrete RC for 2% in 50 years suite	80
Table 5-8: LA3-RB-Comparison of simplified RC and reinforced concrete RC for Near-Fault suite.....	80
Table 6-1: Period comparison for three levels of stiffness of RC in LA9-RA	82
Table 6-2: Period comparison for three levels of stiffness of RC in LA9-RB	84
Table 6-3: LA9-RA-Comparison of simplified RC and reinforced concrete RC for 10% in 50 years suite	119
Table 6-4: LA9-RA-Comparison of simplified RC and reinforced concrete RC for 2% in 50 years suite	119
Table 6-5: LA9-RA-Comparison of simplified RC and reinforced concrete RC for Near-Fault suite.....	119
Table B-1: Maximum Inter-Story Drift (%) for LA3-RA (10% in 50 years suite)	128
Table B-2: Maximum Inter-Story Drift (%) for LA3-RA (2% in 50 years suite)	129
Table B-3: Maximum Inter-Story Drift (%) for LA3-RA (Near-Fault ground motions)	130
Table B-4: Maximum Inter-Story Drift (%) for LA3-RB (10% in 50 years suite).....	131
Table B-5: Maximum Inter-Story Drift (%) for LA3-RB (2% in 50 years suite).....	132
Table B-6: Maximum Inter-Story Drift (%) for LA3-RB (Near-Fault ground motions)	133
Table B-7: Maximum Inter-Story Drift (%) for LA9-RA (10% in 50 years suite)	134
Table B-8: Maximum Inter-Story Drift (%) for LA9-RA (2% in 50 years suite)	135

Table B-9: Maximum Inter-Story Drift (%) for LA9-RA (Near-Fault ground motions)	136
Table B-10: Maximum Inter-Story Drift (%) for LA9-RB (10% in 50 years suite).....	137
Table B-11: Maximum Inter-Story Drift (%) for LA9-RB (2% in 50 years suite).....	138
Table B-12: Maximum Inter-Story Drift (%) for LA9-RB (Near-Fault ground motions)	139

LIST OF FIGURES

Figure 2.1: Mean inter-story drift profiles for the as-built and retrofitted frames. (Gunay et al. 2009)	4
Figure 2.2: Typical elastic deflected shape of dual systems: (a) fixed wall; and (b) hinged wall.....	5
(Alavi and Krawinkler 2004a and 2004b)	5
Figure 2.3: Basic components in a rocking wall system (Wada et al. 2011)	6
Figure 2.4: Installed steel damper with a length of 1500mm (Wada et al. 2011).....	6
Figure 3.1A: Soft-story failure in a moment frame	8
Figure 3.1B: Moment frame retrofitted with a RC	9
Figure 3.2: Plan view for 3- and 9-story SAC buildings for Los Angeles (Gupta and Krawinkler, 1999)	12
Figure 3.3: Elevation of 3- and 9-story SAC buildings for Los Angeles (Gupta and Krawinkler, 1999)	12
Figure 3.4: Moment frame member sizes for buildings being analyzed (Gupta and Krawinkler, 1999)	13
Figure 4.1: Three- Story FE Model.....	16
Figure 4.2: Nine-story FE model	17
Figure 4.3: Giuffre-Menegotto-Pinto steel model with isotropic strain hardening (OpenSees Wiki)	18
Figure 4.4: FBE VS. DBE (Williamson, 2012)	20
Figure 4.5: FBE vs. DBE (Terzic, 2011)	20
Figure 4.6: Accuracy of Low-cycle Fatigue Model: OpenSees vs. Experimental Hysteresis Results (Uriz and Mahin, 2008)	22
Figure 4.7: Cyclic loading protocol used in analysis	23
Figure 4.8: Strength degradation effect on beam-to-column connection due to Low- cycle Fatigue	23
Figure 4.9: Identification of first story stiffness for 3-story building	25
Figure 4.10: Identification of first story stiffness for 9-story building	25
Figure 4.11: P-Delta effect in LA3-R.	27
Figure 4.12: Beam axial deformation difference with and without Ghost Members	29
Figure 4.13: 9- Story mode shapes before modification	30

Figure 4.14: 9-Story mode shapes after modification.....	31
Figure 4.15: 3-Story mode shapes before modification.....	31
Figure 4.16: 3-Story mode shapes after modification.....	32
Figure 5.1: 1 st mode shapes comparison of LA3-RA with different RC stiffness values.	34
Figure 5.2: 2 nd mode shapes comparison of LA3-RA with different RC stiffness values	35
Figure 5.3: 3 rd mode shapes comparison of LA3-RA with different RC stiffness values	35
Figure 5.4: 1 st mode shapes comparison of LA3-RB with different RC stiffness values.	36
Figure 5.5: 2 nd mode shapes comparison of LA3-RB with different RC stiffness values	37
Figure 5.6: 3 rd mode shapes comparison of LA3-RB with different RC stiffness values	37
Figure 5.7: DCF for collapse prevention category.....	42
Figure 5.8: DCF for life safety category	42
Figure 5.9: DCF for immediate occupancy category	43
Figure 5.10: Comparison of maximum DCF for the three performance categories	43
Figure 5.11: Result of pushover analysis of the bottom story of LA3-R.....	45
Figure 5.12: Change in slope of pushover curve	45
Figure 5.13: RC shear demand (Performance category: collapse prevention)	46
Figure 5.14: RC shear demand (Performance category: life safety).....	47
Figure 5.15: RC shear demand (Performance category: immediate occupancy).....	47
Figure 5.16: RC moment demand (Performance category: collapse prevention).....	48
Figure 5.17: RC moment demand (Performance category: life safety)	48
Figure 5.18: RC moment demand (Performance category: immediate occupancy)	49
Figure 5.19: RC axial demand (Performance category: collapse prevention)	50
Figure 5.20: RC axial demand (Performance category: life safety)	50
Figure 5.21: Link axial demand (Performance category: immediate occupancy)	51
Figure 5.22: Median of the maximum inter-story drifts from all ground motions (10% in 50 years)	57
Figure 5.23: Median of the maximum inter-story drifts from critical ground motions (10% in 50 years)	57
Figure 5.24: Median of the maximum inter-story drifts from all ground motions (2% in 50 years)	58
Figure 5.25: Median of the maximum inter-story drifts from all ground motions (near-fault earthquakes)	59
Figure 5.26: Median of the maximum inter-story drifts from critical ground motions (near-fault earthquakes)	59

Figure 5.27: Normalized median shear demand in RC (Earthquakes corresponding to 10% in 50 years)	61
Figure 5.28: Normalized median bending moment demand in RC (Earthquakes corresponding to 10% in 50 years)	61
Figure 5.29: Normalized median shear demand in RC (Earthquakes corresponding to 2% in 50 years)	62
Figure 5.30: Normalized median bending moment demand in RC (Earthquakes corresponding to 2% in 50 years)	62
Figure 5.31: Normalized median shear demand in RC (Near-Fault Earthquakes)	63
Figure 5.32: Normalized median bending moment demand in RC (Near-Fault Earthquakes)	63
Figure 5.33: Normalized median axial demand in links (Earthquakes corresponding to 10% in 50 years)	64
Figure 5.34: Normalized median axial demand in links (Earthquakes corresponding to 2% in 50 years)	65
Figure 5.35: Normalized median axial demand in links (Near-Fault Earthquakes)	65
Figure 5.36: Drift comparison of LA3-RA and LA3-RB (Earthquakes corresponding to 10% in 50 years)	66
Figure 5.37: Drift comparison of LA3-RA and LA3-RB (Earthquakes corresponding to 2% in 50 years)	67
Figure 5.38: Drift comparison of LA3-RA and LA3-RB (Near-Fault Earthquakes)	67
Figure 5.39: Normalized median shear demand in RC of LA3-RB (Earthquakes corresponding to 10% in 50 years)	68
Figure 5.40: Normalized median bending moment demand in RC of LA3-RB (Earthquakes corresponding to 10% in 50 years)	69
Figure 5.41: Normalized median shear demand in RC of LA3-RB (Earthquakes corresponding to 2% in 50 years)	69
Figure 5.42: Normalized median bending moment demand in RC of LA3-RB (Earthquakes corresponding to 2% in 50 years)	70
Figure 5.43: Normalized median shear demand in RC of LA3-RB (Near-Fault Earthquakes)	70
Figure 5.44: Normalized median bending moment demand in RC of LA3-RB (Near-Fault Earthquakes)	71
Figure 5.45: Normalized median axial demand in links of LA3-RB (Earthquakes corresponding to 10% in 50 years)	72
Figure 5.46: Normalized median shear demand in links of LA3-RB (Earthquakes corresponding to 10% in 50 years)	72

Figure 5.47: Normalized median bending moment demands in links of LA3-RB (Earthquakes corresponding to 10% in 50 years)	73
Figure 5.48: Normalized median axial demand in links of LA3-RB (Earthquakes corresponding to 2% in 50 years)	73
Figure 5.49: Normalized median shear demand in links of LA3-RB (Earthquakes corresponding to 2% in 50 years)	74
Figure 5.50: Normalized median bending moment demands in links of LA3-RB (Earthquakes corresponding to 2% in 50 years)	74
Figure 5.51: Normalized median axial demand in links of LA3-RB (Near-Fault Earthquakes)	75
Figure 5.52: Normalized median shear demand in links of LA3-RB (Near-Fault Earthquakes)	75
Figure 5.53: Normalized median bending moment demands in links of LA3-RB (Near-Fault Earthquakes).....	76
Figure 5.54: Median residual drift in LA3-RB	77
Figure 5.55: Median residual drift in LA3-RA	77
Figure 5.56: Reinforced concrete RC cross-section design for LA3-RB	79
Figure 6.1: 1 st mode shapes comparison of LA9-RA with different RC stiffness values.	82
Figure 6.2: 2 nd mode shapes comparison of LA9-RA with different RC stiffness values	83
Figure 6.3: 3 rd mode shapes comparison of LA9-RA with different RC stiffness values	83
Figure 6.4: 1 st mode shapes comparison of LA9-RB with different RC stiffness values.	84
Figure 6.5: 2 nd mode shapes comparison of LA9-RB with different RC stiffness values	85
Figure 6.6: 3 rd mode Shapes comparison of LA9-RB with different RC stiffness values	85
Figure 6.7: DCF for collapse prevention category.....	87
Figure 6.8: DCF for life safety category	87
Figure 6.9: DCF for immediate occupancy category	88
Figure 6.10: Comparison of maximum DCF for the three performance categories	88
Figure 6.11: Result of pushover analysis of the bottom story of LA9-RA.....	90
Figure 6.12: Change in slope of pushover curve	90
Figure 6.13: RC shear demand (Performance category: collapse prevention)	91
Figure 6.14: RC shear demand (Performance category: life safety).....	92
Figure 6.15: RC shear demand (Performance category: immediate occupancy).....	92
Figure 6.16: RC moment demand (Performance category: collapse prevention).....	93
Figure 6.17: RC moment demand (Performance category: life safety)	93

Figure 6.18: RC moment demand (Performance category: immediate occupancy)	94
Figure 6.19: RC axial demand (Performance category: collapse prevention)	95
Figure 6.20: RC axial demand (Performance category: life safety)	95
Figure 6.21: Link axial demand (Performance category: immediate Occupancy)	96
Figure 6.22: Median of the maximum inter-story drifts from all ground motions (10% in 50 years)	98
Figure 6.23: Median of the maximum inter-story drifts from critical ground motions (10% in 50 years)	98
Figure 6.24: Median of the maximum inter-story drifts from all ground motions (2% in 50 years)	99
Figure 6.25: Median of the maximum inter-story drifts from all ground motions (Near-Fault Earthquakes)	100
Figure 6.26: Median of the maximum inter-story drifts from critical ground motions (Near-Fault Earthquakes)	100
Figure 5.27: Normalized median shear demand in RC (Earthquakes corresponding to 10% in 50 years)	102
Figure 6.28: Normalized median bending moment demand in RC (Earthquakes corresponding to 10% in 50 years)	102
Figure 6.29: Normalized median shear demand in RC (Earthquakes corresponding to 2% in 50 years)	103
Figure 6.30: Normalized median bending moment demand in RC (Earthquakes corresponding to 2% in 50 years)	103
Figure 6.31: Normalized median shear demand in RC (Near-Fault Earthquakes)	104
Figure 6.32: Normalized median bending moment demand in RC (Near-Fault Earthquakes)	104
Figure 6.33: Normalized median axial demand in links (Earthquakes corresponding to 10% in 50 years)	105
Figure 6.34: Normalized median axial demand in links (Earthquakes corresponding to 2% in 50 years)	106
Figure 6.35: Normalized median axial demand in links (Near-Fault Earthquakes)	106
Figure 6.36: Drift comparison of LA9-RA and LA9-RB (Earthquakes corresponding to 10% in 50 years)	107
Figure 6.37: Drift comparison of LA9-RA and LA9-RB (Earthquakes corresponding to 2% in 50 years)	108
Figure 6.38: Drift comparison of LA9-RA and LA9-RB (Near-Fault Earthquakes)	108
Figure 6.39: Normalized median shear demand in RC of LA9-RB (Earthquakes corresponding to 10% in 50 years)	109

Figure 6.40: Normalized median bending moment demand in RC of LA9-RB (Earthquakes corresponding to 10% in 50 years)	109
Figure 6.41: Normalized median shear demand in RC of LA9-RB (Earthquakes corresponding to 2% in 50 years)	110
Figure 6.42: Normalized median bending moment demand in RC of LA9-RB (Earthquakes corresponding to 2% in 50 years)	110
Figure 6.43: Normalized median shear demand in RC of LA9-RB (Near-Fault Earthquakes)	111
Figure 6.44: Normalized median bending moment demand in RC of LA9-RB (Near-Fault Earthquakes).....	111
Figure 6.45: Normalized median axial demand in links of LA9-RB (Earthquakes corresponding to 10% in 50 years)	112
Figure 6.46: Normalized median shear demand in links of LA9-RB (Earthquakes corresponding to 10% in 50 years)	112
Figure 6.47: Normalized median bending moment demands in links of LA9-RB (Earthquakes corresponding to 10% in 50 years)	113
Figure 6.48: Normalized median axial demand in links of LA9-RB (Earthquakes corresponding to 2% in 50 years)	113
Figure 6.49: Normalized median shear demand in links of LA9-RB (Earthquakes corresponding to 2% in 50 years)	114
Figure 6.50: Normalized median bending moment demands in links of LA9-RB (Earthquakes corresponding to 2% in 50 years)	114
Figure 6.51: Normalized median axial demand in links of LA9-RB (Near-Fault Earthquakes)	115
Figure 6.52: Normalized median shear demand in links of LA9-RB (Near-Fault Earthquakes)	115
Figure 6.53: Normalized median bending moment demands in links of LA9-RB (Near-Fault Earthquakes).....	116
Figure 6.54: Median residual drift in LA9-RB	117
Figure 6.55: Median residual drift in LA9-RA	117
Figure 6.56: Reinforced concrete RC cross-section design for LA9-RA	118
Figure A.1: DBE Design Spectrum and response spectra for 10% in 50 years ground motions.....	125
Figure A.2: MCE Design Spectrum and response spectra for 2% in 50 years ground motions.....	125
Figure A.3: DBE Design Spectrum and response spectra for Near-Fault ground motions.....	126

Figure B.1: Number of earthquakes under each performance objective for LA3-RA (10% in 50 years)	140
Figure B.2: Number of earthquakes under each performance objective for LA3-RA (2% in 50 years)	140
Figure B.3: Number of earthquakes under each performance objective for LA3-RA (Near-Fault Earthquakes)	141
Figure B.4: Number of earthquakes under each performance objective for LA3-RB (10% in 50 years)	141
Figure B.5: Number of earthquakes under each performance objective for LA3-RB (2% in 50 years)	142
Figure B.6: Number of earthquakes under each performance objective for LA3-RB (Near-Fault Earthquakes)	142
Figure B.7: Number of earthquakes under each performance objective for LA9-RA (10% in 50 years)	143
Figure B.8: Number of earthquakes under each performance objective for LA9-RA (2% in 50 years)	143
Figure B.9: Number of earthquakes under each performance objective for LA9-RA (Near-Fault Earthquakes)	144
Figure B.10: Number of earthquakes under each performance objective for LA9-RB (10% in 50 years)	144
Figure B.11: Number of earthquakes under each performance objective for LA9-RB (2% in 50 years)	145
Figure B.12: Number of earthquakes under each performance objective for LA9-RB (Near-Fault Earthquakes)	145

1.0 Introduction

1.1 Background

Modern building codes consider elastic analysis techniques to determine the design force distribution at each level of the building. These forces are then reduced due to the presence of system ductility. The demand and capacity are expected to have a similar distribution along the height of a building structure to achieve the desirable sway mechanism when a strong earthquake occurs. However, there are some factors that may contribute significantly to variation between demand and capacity. These factors include the actual mass distribution different from the one considered in the original design, non-uniform overstrength along the height of the building, and uncertainty of magnitude and frequency contents of ground motions. The variation of demand and capacity may result in concentrated ductility demand in a certain story. The concentration of ductility demands in a certain level may result in a soft-story failure mechanism.

Soft-story failure mechanisms can lead to very severe structural damage or collapse of the building. There are abundant structures in seismic regions that possess seismic deficiencies that may lead to soft-story failures. In addition, some of the buildings exist near faults and thus are subjected to near-fault ground motion pulses. These strong pulses may lead to concentrated ductility demand at the bottom story of multi-story building frames, leading to formation of the detrimental soft-story mechanism (Alavi and Krawinkler, 2004). Therefore, there is a research need to develop effective, practical, and economical retrofit strategies for steel building frames vulnerable to soft-story failures to reduce the probability of severe structural damage.

1.2 Scope/Objective

This thesis focuses on investigating adequacy of a retrofit strategy to mitigate drift concentration and thus soft-story failure in multi-story steel moment frames. The considered strategy includes a sufficiently stiff Rocking Core (RC) pinned to ground and connected to an existing steel moment frame to create relatively uniform inter-story drifts along the height of the building. Two representative steel moment frames, including one low-rise and one mid-rise steel moment frames that originally exhibit soft-story failures are retrofitted using the considered strategy. Finite Element (FE) models of the retrofitted systems that take into account member yielding and rupture behavior are developed. An analytical research program including nonlinear static pushover analysis and nonlinear dynamic analysis is conducted to address key issues regarding seismic performance evaluation and design demand estimates in the retrofitted system.

1.3 Thesis Organization

This thesis includes a total of 7 chapters. The organization provides the reader with a sequential process of how the research was conducted.

Chapter 2 presents an overview of past research related to soft-story failures and retrofit strategies for preventing soft-story failures and their limitations.

Chapter 3 describes the retrofit strategy being considered and selection of low-rise and mid-rise moment frame benchmark buildings.

Chapter 4 presents development and validation of FE models of the original and retrofitted benchmark buildings.

Chapter 5 presents the nonlinear static and dynamic analyses for the benchmark low-rise moment frame building.

Chapter 6 presents the nonlinear static and dynamic analyses for the benchmark mid-rise moment frame building.

Chapter 7 summarizes the results from this investigation and provides recommendations for future research.

2.0 Literature Review

This chapter briefly describes some past research and projects related to seismic retrofit of building frames vulnerable to soft-story failure that have led to the foundation of this thesis.

2.1 Gunay et al. (2009)

Gunay et al. (2009) studied the effect of using rocking walls as a retrofit system on a non-ductile reinforced concrete moment frame with unreinforced masonry infill walls to reduce the tendency of soft-story failure. The analysis consisted of nonlinear static and dynamic analyses. As shown in Figure 2.1, it was concluded that inter-story drift concentrations and ductility demands were reduced along the height of an example 9-story building. The results obtained were very promising in significantly improving the seismic performance of non-ductile reinforced concrete frames.

Figure 2.1: Mean inter-story drift profiles for the as-built and retrofitted frames. (Gunay et al. 2009)

2.2 Alavi and Krawinkler (2004a and 2004b)

Alavi and Krawinkler (2004a and 2004b) study the near-fault impulses and a retrofit strategy for enhancing building performance against soft-story failures. In this research it was found that structures, including those that are code compliant, subjected to near-fault ground motions can experience large ductility demands in the first story from the large pulse-like motions. The retrofit strategy consisted of introducing a wall that is either fixed or hinged at the base. Figure 2.2 provides an elevation view of the retrofit considerations. In both cases the wall significantly reduced concentrated ductility demands; however a hinged wall at the base was recommended to be more effective due to the fact that it does not introduce large demands at the foundation.

Figure 2.2: Typical elastic deflected shape of dual systems: (a) fixed wall; and (b) hinged wall.
(Alavi and Krawinkler 2004a and 2004b)

2.3 Wada et al. (2011) and Qu et al. (2012)

Wada et al. (2011) and Qu et al. (2012) studied the implementation of prestressed rocking walls and steel dampers in an attempt to increase strength and ductility of an 11-story reinforced concrete frame. The retrofit strategy was investigated through nonlinear response history analysis and the results showed that the inter-story drift was significantly

reduced and distributed more uniformly along the building's height. Figure 2.3 illustrates a basic schematic of the retrofit system and Figure 2.4 shows the steel damper used in the system as an energy dissipation device.

Figure 2.3: Basic components in a rocking wall system (Wada et al. 2011)

Figure 2.4: Installed steel damper with a length of 1500mm (Wada et al. 2011)

3.0 Description of Retrofit Strategy and Benchmark Buildings

3.1 Retrofit Strategy

Previous earthquakes have repeatedly shown the detrimental outcome from the soft-story failure of steel moment frame buildings in which large inelastic deformations concentrated at one single story. Investigated in this thesis is an attempt to reduce the probability of formation of soft-story failure mechanisms for sub-standard steel moment frame buildings (i.e., those vulnerable to soft-story failure). The proposed retrofit strategy is to introduce a rocking core (RC) in the system to redistribute the seismic forces between stories and provide more uniform inter-story drift distributions in the system.

Implementation of RC into an existing steel moment frame building is illustrated in Figure 3.1B, in which links connect the RC to the existing framing. It is noted that the RC in practice can be a rigid reinforced concrete wall or a relatively stiff truss frame. For simplicity, the RC was idealized as a line element in this investigation. Such an assumption might underestimate the rotation demands at the ends of the connecting links depending on the width of RC; but will highlight the beneficial contribution of RC regardless of whatever system (reinforce concrete wall or steel truss) is adopted. Moreover, RC is selected to be pinned to the ground to reduce the seismic demands on the foundation and minimize the extra seismic force due to the addition of RC. The link element may consist of a structural member with simple connections such that only the interacting axial force is transferred between the existing framing and RC.

Additionally, for existing structures that do not have adequate ductility and/or strength to resist seismic demands, fixed link connections or other passive energy dissipation devices (steel yielding, viscous damping, etc.) can be implemented between the RC and the existing framing to reduce the ductility demands. In addition to the pin-pin links, this thesis also investigates the case in which the link-to-RC connections are fixed whereas the link-to-existing-frame connections are pinned. It is recognized that applying pinned link-to-existing-frame connections may help reduce the flexural demands on the columns of existing frames.

Figure 3.1A: Soft-story failure in a moment frame

Figure 3.1B: Moment frame retrofitted with a RC

3.2 Selection of Benchmark Buildings

Benchmark steel moment frame buildings vulnerable to soft-story failure are selected. Seismic performances of the selected buildings before and after implementation of RC are compared to demonstrate the efficiency of the retrofit strategy discussed in Section 3.1. The following parameters were taken into consideration when selecting the demonstration buildings: 1) buildings should be designed for a region of high seismicity; 2) building heights should cover both low-rise and mid-rise, in which the RC are intuitively believed to be practical; 3) design of structural components should be consistent with the recent seismic design practice; and 4) properties of the buildings should be available in published papers, which can be used to validate the building numerical models developed in this research.

As a result, the 3-story low-rise and 9-story mid-rise steel moment frame buildings designed for Los Angeles as part of the SAC steel project (Gupta and Krawinkler, 1999) were selected in this project. It is noted that similar buildings were also designed for

Seattle and Boston in the SAC project; however these buildings were not considered in this research. Moreover, both pre-and post- Northridge building designs are presented in (Gupta and Krawinkler, 1999). Considering the buildings with pre-Northridge design may exhibit less desirable seismic performance, which better suits the need of this research, this thesis only focuses on the pre- Northridge buildings.

As indicated in (Gupta and Krawinkler, 1999), the selected pre-Northridge buildings were designed in accordance with UBC 1994 and without consideration of the recommendations of FEMA 267 (1995). Figures 3.2 and 3.3 show the plan view and elevation view of the selected buildings. The bay width for both buildings is 30 feet. Figure 3.4 presents the sizes of the members designed in both buildings. The following summarizes the load/mass information used in design of the selected buildings. More detailed information about these buildings is available from (Gupta and Krawinkler, 1999).

Loads on the considered buildings (for both three-story and nine-story examples);

Steel Framing:	as designed
Floors and Roof:	3 inch metal decking with 2.5 inches of normal weight concrete fill and fireproofing
Roofing:	7 psf average
Ceilings/Flooring:	3 psf average, including fireproofing
Mechanical/Electrical:	7 psf average for all floors, additionally 40 psf over penthouse area for equipment
Partitions:	as per code requirements (10 psf for seismic load, 20 psf for gravity design)
Exterior Wall:	25 psf of wall surface average, including any penthouses. Assume 2 feet from perimeter column lines to edge of building envelope. Include 42 inch parapet at main roof level, none at penthouse roof.
Live Load:	typical code values for office occupancy (50 psf everywhere)

Wind Load	as per code requirements, assuming congested area (exposure B as per UBC '94 definition)
Seismic Load:	as per code requirements

Resulting floor load distribution (Gupta and Krawinkler, 1999):

Floor dead load for weight calculations:	96 psf
Floor dead load for mass calculations:	86 psf
Roof dead load excluding penthouse:	83 psf
Penthouse dead load:	116 psf
Reduced live load per floor and for roof:	20 psf

Based on the floor load distribution, the published seismic mass for the two buildings being considered here are (the values are for the entire structure):

3-story Structure:

Roof:	70.90 kips-sec ² /ft
Floor 3 and Floor 2:	65.53 kips-sec ² /ft

9-story Structure:

Roof:	73.10 kips-sec ² /ft
Floor 9 to Floor 3:	67.86 kips-sec ² /ft
Floor 2:	69.04 kips-sec ² /ft

For simplicity, 2D models rather than 3D models were considered in this thesis. Moreover, due to symmetry, only the moment frames along axis A were considered for both buildings. Note that both considered 2D moment frames having columns bending about their strong axes.

Figure 3.2: Plan view for 3- and 9-story SAC buildings for Los Angeles (Gupta and Krawinkler, 1999)

Figure 3.3: Elevation of 3- and 9-story SAC buildings for Los Angeles (Gupta and Krawinkler, 1999)

PRE-NORTHRIDGE DESIGNS

NS Moment Resisting Frame

NS Gravity Frames

3-story Building

Story/Floor	COLUMNS		DOUBLER PLATES (in)	GIRDER	COLUMNS		BEAMS
	Exterior	Interior			Below penthouse	Others	
1/2	W14X257	W14X311	0.0	W33X118	W14X82	W14X68	W18X35
2/3	W14X257	W14X311	0.0	W30X116	W14X82	W14X68	W18X35
3/Roof	W14X257	W14X311	0.0	W24X68	W14X82	W14X68	W16X26

9-story Building

Story/Floor	COLUMNS		DOUBLER PLATES (in)	GIRDER	COLUMNS		BEAMS
	Exterior	Interior			Below penthouse	Others	
-1/1	W14X370	W14X500	0.0	W36X160	W14X211	W14X193	W21X44
1/2	W14X370	W14X500	0.0	W36X160	W14X211	W14X193	W18X35
2/3	W14X370, W14X370	W14X500, W14X455	0.0	W36X160	W14X211, W14X159	W14X193, W14X145	W18X35
3/4	W14X370	W14X455	0.0	W36X135	W14X159	W14X145	W18X35
4/5	W14X370, W14X283	W14X455, W14X370	0.0	W36X135	W14X159, W14X120	W14X145, W14X109	W18X35
5/6	W14X283	W14X370	0.0	W36X135	W14X120	W14X109	W18X35
6/7	W14X283, W14X257	W14X370, W14X283	0.0	W36X135	W14X120, W14X90	W14X109, W14X82	W18X35
7/8	W14X257	W14X283	0.0	W30X99	W14X90	W14X82	W18X35
8/9	W14X257, W14X233	W14X283, W14X257	0.0	W27X84	W14X90, W14X61	W14X82, W14X48	W18X35
9/Roof	W14X233	W14X257	0.0	W24X68	W14X61	W14X48	W16X26

Figure 3.4: Moment frame member sizes for buildings being analyzed (Gupta and Krawinkler, 1999)

4.0 Finite Element Modeling and Validation

As discussed in Chapter 3, the three- and nine-story moment frames designed for the SAC project were selected in this thesis for the following reasons: 1) properties of these buildings are available in literature, which can be used for validating the developed models; 2) these buildings were designed for Los Angeles, CA according to the current seismic design practice and thus are deemed to be representative in region of high seismicity; and 3) these buildings have been investigated by other researchers for other system seismic performance aspects (such as demonstrating the adequacy of other seismic retrofit technologies), which possibly offers the opportunities to compare the advantage of the proposed seismic retrofit strategy over others. However, it should be noted that the original SAC buildings were designed to meet code standards and thus the buildings did not have the potential of forming a soft-story mechanism under seismic loading. Because this research aims at evaluating the adequacy of the RC retrofit strategy for sub-standard steel moment frames, it is better to focus on the systems vulnerable to soft-story failures. To better suit the purpose of this research, modifications were made to the original buildings. Specifically, the bottom-story columns in the three-story building, which are originally fixed to ground, were modified to be pinned in this research. For the nine-story building, the basement was neglected and all columns in the bottom story were assumed to be pinned to ground. As shown by the results to be presented in detail in Chapters 5 and 6, the above modifications in column base boundary conditions triggered formation of soft-story failures at the bottom levels in both considered buildings. In the following sections of this chapter, the original three- and nine-story SAC buildings are

denoted as LA3 and LA9; the three- and nine-story buildings modified to exhibit soft-story behavior are denoted as LA3-R and LA9-R.

This chapter describes the development of the Finite Element (FE) models for the considered buildings. The FE models were developed using the Open System for Earthquake Engineering Simulation (OpenSees). Figures 4.1 and 4.2 schematically illustrate the FE models for the three- and nine story buildings with RC. As shown, the models include the original moment frame, RC, a gravity column to capture the P-Delta effect, and the links connecting the gravity column and RC to the existing frame. The following sections present the assumptions and details in developing the models.

To consider P-Delta effects, a gravity column was added to the models. The links connecting the gravity column to the moment frame were modeled to have pin connections on both ends. Half of each story weight minus the tributary weight on the moment frame was lumped on the corresponding joint of the gravity column. The tributary weight of the moment frame was distributed on the moment frame joints. The seismic mass of the building was also distributed equally on the moment frame joints.

Legend

—	Nonlinear Force Based (FB) Element	■	Fixed connection for LA3-RB Pin connection for LA3-RA
—	Truss element for LA3-RA, nonlinear FB element for LA3-RB	////	Fixed connection
—	Truss element	△	Pin connection
—	Elastic element	○	Hinge connection
—	Elastic element		

Figure 4.1: Three- Story FE Model

Figure 4.2: Nine-story FE model

Figure 4.1 and 4.2 shows the FE models that were used in the analysis for this research.

All columns of the model are assumed to be pinned to the foundation.

4.1 Modeling of Frame Components

4.1.1 Selection of Material Model

The material used for all beams and columns of the three- and nine- story moment frames was the Giuffre-Menegotto-Pinto steel model with isotropic strain hardening. Figure 4.3 shows a parametric study of a parameter R for the stress strain curve of this material model. The parameter R controls the transition from the elastic portion to inelastic portion of the curve. An R value of 20 as recommended in the OpenSees Wiki was used. The elastic modulus, E , was taken to be that of ordinary steel with a value of 29,000 ksi. Strain hardening ratio of 2% (.02) was used in the simulations.

Figure 4.3: Giuffre-Menegotto-Pinto steel model with isotropic strain hardening (OpenSees Wiki)

The beam yield strength reported in (Gupta and Krawinkler, 1999) is 36ksi for both the three- and nine-story buildings. However, recent research has reported that significant

overstrength may exist in this type of steel. Given that material overstrength in beams may trigger the formation of inelastic behavior in columns and hence soft-story failure, an over strength factor of 1.5 was applied to the steel in beams, resulting in a yield strength of 54 ksi. The column yield strength of 50 ksi reported in Gupta and Krawinkler (1999) was used for all columns of both buildings.

4.1.2 Selection of Elements

There are two types of fiber elements in OpenSees, i.e., force-based elements (FBEs) and displacement-based elements (DBEs) that can be used for the flexural members in the models. In a previous investigation performed at Cal Poly (Williamson 2012), 3D models of the SAC buildings were developed using both DBEs and FBEs. Figure 4.4 compares the results from nonlinear response history analyses of the 3D three-story SAC moment frames developed using DBEs and FBEs, respectively. As shown, both elements provide very similar results. Williamson (2012) also suggests that FBEs require more integration points along each element and DBE requires more elements along the member to converge to the accurate results. If convergence could be achieved, FBE and DEB typically both provide very similar and reliable results. This point was further confirmed by the results (see Figure 4.5) from the OpenSees training document -“Force-based Element vs. Displacement-based Element”, which is available at OpenSees Wiki (Terzic 2011).

In this investigation, FBEs were selected for the FE models. As a result, each frame member (i.e., columns and beams) was considered by a single FBE with 7 integration points along the member. Moreover, the same fiber arrangements were assigned to each

wide-flange member including, 16 fibers along the web depth; 2 fibers through web thickness; 16 fibers along flange width; and 4 fibers through flange thickness.

Figure 4.4: FBE VS. DBE (Williamson, 2012)

Figure 4.5: FBE vs. DBE (Terzic, 2011)

4.1.3 Consideration of Low-Cycle Fatigue

Modeling material damage caused by low-cycle fatigue is a new feature in the latest version of OpenSees. Such a new function was based on the latest research outcome by Uriz and Mahin (2008). While the low-cycle fatigue model has been experimentally validated and successfully used in recent research of steel concentrically braced frames (see Figure 4.6), it has not been widely used for steel moment frames. In the event of strong earthquakes, the beam-to-column connections of steel moment frames may be subjected to reversal of large inelastic deformation, possibly leading to low-cycle fatigue induced fractures in beam-to-column connections. As such, this thesis incorporates the low-cycle fatigue model into the beams of the considered SAC frames. It is noted that the parameters required in the low-cycle fatigue model were determined based on the cyclic testing results of wide flange members (Uriz and Mahin, 2008).

Figure 4.6: Accuracy of Low-cycle Fatigue Model: OpenSees vs. Experimental Hysteresis Results (Uriz and Mahin, 2008)

To ensure consideration of the low-cycle fatigue, the developed three-story model was subjected to a roof cyclic loading protocol shown in Figure 4.7. Figure 4.8 shows the strength degradation of the right end of the beam at the bottom story of the center bay of LA3. As shown, the flexural resistance degrades beyond a certain level of displacement, indicating formation of the low-cycle fatigue induced ruptures at the beam-to-column connections.

Figure 4.5: Cyclic loading protocol used in analysis

Figure 4.8: Strength degradation effect on beam-to-column connection due to Low-cycle Fatigue

4.2 Modeling of RC

The RC was modeled using the elasticBeamColumn element in OpenSees to ensure purely elastic behavior in it. The elastic modulus of RC was taken to be that of ordinary steel (29,000 ksi). A large cross sectional area of 1,000 in² was assigned to the RC to

ensure axial rigidity. Moreover, proper parameters were assigned to the RC element such that the RC shear deformation is negligible. To better understand the flexural stiffness of the RC relative to the existing frame, the following stiffness ratio, α , was defined:

$$\alpha = \frac{I_{RC}E}{k_1 h_{s1}^3} \quad \text{Eq. 4.1}$$

where: I_{RC} is the RC moment of inertia

E is the RC elastic modulus

k_1 is the structure first story stiffness and

h_{s1} is the structure's first story height

Solving for I_{RC} gives the following equation:

$$I_{RC} = \frac{\alpha k_1 h_{s1}^3}{E} \quad \text{Eq. 4.2}$$

In this thesis, the value of α was varied in the analysis to achieve different levels of RC stiffness. Moreover, k_1 was determined through pushover analysis on the bottom-story frames as part of the considered SAC buildings. Figures 4.9 and 4.10 show the results from pushover analyses of the bottom story of the three- and nine- story buildings. The stiffness values identified for the three- and nine- story buildings were 229 kips/inch and 236 kips/inch, respectively.

Figure 4.9: Identification of first story stiffness for 3-story building

Figure 4.10: Identification of first story stiffness for 9-story building

4.3 Modeling of Links

The links connecting the gravity column to existing frame (see Figure 4.1) were considered by truss elements. A large area of 1000 in² was assigned to these truss elements to ensure they were axially rigid. The links connecting the RC to existing

frames were modeled in two different manners. In one case the links only transfer axial loads, they were modeled using elastic elements with large cross sectional areas and moments of inertia, and pin-pin boundary conditions at the ends. In the other case the links are used as energy dissipation elements, they were modeled using FBEs with areas and moments of inertia selected from specific link designs. In such cases, the end connecting the link to RC is fixed and the end connecting the link to existing frame is pinned. More detailed information about link design is presented in Chapters 5 and 6 for the 3-and 9-story buildings, respectively.

4.4 Consideration of P-Delta Effect

To consider the P-Delta effect, a gravity column was introduced to each considered building as shown in Figure 4.1. The gravity column was modeled using an elastic element (i.e., `elasticBeamColumn` in OpenSees). Moreover, it was expected that the gravity column remain axially rigid but with no flexural strength. Accordingly, its cross sectional area and moment of inertia were taken to be 100,000 in² and 1 in⁴, respectively. The gravity loads tributary to each modeled moment frame was applied to the gravity column at the floor levels. Based on Gupta and Krawinkler (1999), the gravity loads corresponding to the gravity columns are provided below (these values are tributary to the considered frame):

<u>3-Story Model:</u>	
2 nd and 3 rd Levels-	1002.8 kips
Roof-	1095.1 kips
<u>9-Story Model:</u>	
2 nd Level-	1074.4 kips
3 rd to 9 th Levels-	1056.1 kips
Roof-	1137.7 kips

To better illustrate the P-Delta effect on system seismic performance, Figure 4.11 shows the response history analysis results of LA3-R with and without consideration of the P-Delta effect. It is noted that RC was not considered in this example and the earthquake excitation is El Centro earthquake (recorded from Imperial Valley 1940 and scaled to have a probability of exceedance of 10% in 50 years). As shown, neglecting the P-Delta effect leads to unconservative predictions for maximum inter-story drift and residual drift. Table 4.1 further compares the maximum drifts at each story from the models with and without consideration of the P-Delta effect.

Figure 4.11: P-Delta effect in LA3-R.

Table 4-1: Maximum percent drift with and without gravity wall

Story Level	% Drift Without Gravity Wall	% Drift With Gravity Wall
1 st Story Level	4.50	5.80
2 nd Story Level	1.20	1.20
3 rd Story Level	.80	.80

4.5 Other Modeling Considerations

As discussed in Section 4.1.3, low-cycle fatigue was considered in the developed models to capture member fractures caused by reversal of large inelastic deformations. When such a failure occurs, the structural continuity will be compromised, leading to numerical difficulty in achieving convergence. In this investigation, a rigid truss element is used as a “ghost element” in parallel with each beam element. It is noted that the ghost element does not provide any lateral strength and stiffness to the system; but will help achieve convergence when ruptures occur at beam ends. Moreover, the ghost elements also capture the axial stiffness provided by concrete slabs. Figure 4.12 shows the axial strain in the first story beams of LA3 with and without the abovementioned ghost elements. The earthquake excitation is El Centro earthquake (recorded from Imperial Valley 1940 and scaled to have a probability of exceedance of 10% in 50 years). As shown, the ghost elements reduce the axial deformation in beams due to the presence of concrete slabs.

Figure 4.12: Beam axial deformation difference with and without Ghost Members

4.6 Model Validation

The vibration periods of the three- and nine- story Pre-Northridge SAC buildings in Los Angeles are presented in Gupta and Krawinkler (1999) and were used to validate the FE models developed for the buildings without RC in this research. As modifications were made in both SAC buildings to trigger the soft-story failure, the FE models were developed and validated in two steps: Step 1, the FE models consistent with the original SAC buildings were developed and validated using the vibration periods available in Gupta and Krawinkler (1999); and Step 2) the models validated in Step 1 were modified to introduce pinned column-to-ground connections as described previously. Table 4.2 compares the periods of the first three vibration modes from Gupta and Krawinkler (1999) and the models developed in this research. As shown, the models without boundary condition modifications and developed in this thesis provide periods very

similar to those from Gupta and Krawinkler (1999), indicating adequacy of the models. The models with boundary condition modifications provide periods longer than those without boundary condition modifications. This is consistent with the fact that replacing fixed connections with pinned connections at column bases tends to reduce stiffness of the system, which leads to longer periods. The mode shapes before and after modifications are shown in Figures 4.13 through 4.16 for both considered buildings.

Table 4-2: Vibration period comparison for validation purposes

Mode	Periods of 3-story SAC building			Periods of 9-story SAC building		
	Gupta and Krawinkler. (1999)	Developed model without modification	Developed model with modification	Gupta and Krawinkler. (1999)	Developed model without modification	Developed model without modification
1	1.03	1.02	1.49	2.34	2.30	2.84
2	0.33	0.33	0.42	0.88	0.86	0.98
3	0.17	0.17	0.19	0.50	0.50	0.53

Figure 4.13: 9- Story mode shapes before modification

Figure 4.14: 9-Story mode shapes after modification

Figure 4.15: 3-Story mode shapes before modification

Figure 4.16: 3-Story mode shapes after modification

5.0 Seismic Performance Evaluation of Three-Story Building

5.1 Impact of RC on Modal Properties

Eigenvalue analyses were first conducted on the FE models of the considered 3-story building to evaluate the effect of RC on modal properties of the retrofitted system. Two cases were considered here: LA3-RA in which the links connecting moment frame to RC are pinned on both ends; and LA3-RB in which the links are pinned to moment frame and fixed to RC, i.e., links are used as force transfer members as well as hysteretic energy dissipation devices. In both cases, three different levels of RC stiffness are considered, modeling RC to be extremely flexible, moderately rigid, and ideally stiff. The three levels of stiffness were determined by setting α equal to 0.003, 0.096 and 5.0 in Equation 4.2. As a result, the corresponding moments of inertia for RC are 90 in⁴, 2,900 in⁴, and 150,000 in⁴ for the low, medium, and high stiffness levels, respectively.

For LA3-RA, periods for the first three modes are summarized in Table 5.1. As shown, addition of RC, while shortens the higher mode periods to a higher degree when RC is extremely rigid, does not significantly shorten the fundamental periods. This trend indicates that addition of RC does not attract significant extra seismic force on the system. Figures 5.1 through 5.3 compare the mode shapes from the models with different RC stiffness levels. As shown, the impact of RC stiffness on mode shapes is also negligible.

Table 5-1: Period comparison for three levels of stiffness of RC in LA3-RA

Mode	Period (sec) for Three Levels of RC Stiffness		
	Low Stiffness ($\alpha=0.003$)	Median Stiffness ($\alpha=0.096$)	High Stiffness ($\alpha=5.0$)
1 st	1.49	1.47	1.32
2 nd	0.42	0.40	0.21
3 rd	0.19	0.18	0.10

Figure 5.1: 1st mode shapes comparison of LA3-RA with different RC stiffness values

Figure 5.2: 2nd mode shapes comparison of LA3-RA with different RC stiffness values

Figure 5.3: 3rd mode shapes comparison of LA3-RA with different RC stiffness values

For LA3-RB, periods for the first three modes are summarized in Table 5.2. As shown, stiffer RC tends to shorten the periods associated with all modes and to a higher degree for the fundamental periods. Comparing the results shown in Table 5.2 with those in Table 5.1, it is found that the corresponding fundamental periods are shorter when the

links are used with energy dissipation capacity; however the differences in higher-mode periods are negligible. This is due to the extra stiffness provided by the fixed link ends. Figures 5.4 to 5.6 further compare the mode shapes of LA3-RB with different RC stiffness levels. As shown, the impact of RC stiffness on mode shapes is also negligible.

Table 5-2: Period comparison for three levels of stiffness of RC in LA3-RB

Mode	Period (sec) for Three Levels of RC Stiffness		
	Low Stiffness ($\alpha=0.003$)	Median Stiffness ($\alpha=0.096$)	High Stiffness ($\alpha=5.0$)
1 st	1.48	1.30	0.97
2 nd	0.41	0.36	0.20
3 rd	0.19	0.17	0.10

Figure 5.4: 1st mode shapes comparison of LA3-RB with different RC stiffness values

Figure 5.5: 2nd mode shapes comparison of LA3-RB with different RC stiffness values

Figure 5.6: 3rd mode shapes comparison of LA3-RB with different RC stiffness values

5.2 Nonlinear Static Pushover Analysis

Nonlinear static pushover analyses are conducted in this section to investigate the contribution of RC on mitigation of soft-story failure in the 3-story building. As nonlinear static pushover analysis does not take into account the hysteretic energy dissipation by

the links with one end fixed to RC, only LA3-RA is included in the analysis. Throughout the analyses, the RC stiffness was varied from 0.003 to 5.0 to explore the most efficient range of RC stiffness in reducing drift concentration. For each considered RC stiffness ratio, the Monte-Carlo simulation technique was used to generate random lateral force distributions required in the nonlinear static analyses. The following sections present generation of random lateral force distributions, determination of target displacements, response quantities of interest, and discussion of the analysis results.

5.2.1 Random Lateral Force Distribution

The lateral force distribution is an important parameter affecting the yielding progression and drift distribution along the height of a structure. However, when an earthquake occurs, the lateral force distribution may not be the same as that assumed in design due to many reasons such as 1) redistribution of seismic masses; 2) progressively developed nonlinear behavior of the system that cause strength and stiffness deteriorations; and 3) unexpected and non-uniform overstrength distribution in the system. As such, it is necessary to evaluate if a RC can successfully prevent drift concentration along the height of the considered building when the lateral force distribution is varying within a practical range. The Monte-Carlo technique was used to account for the uncertainties in lateral force distributions.

Specifically, the seismic mass at each floor was randomly sampled around its nominal value such that the selected value is not greater than 150% of the masses at its adjacent floors. It is recognized that this sampling criterion will produce random reactive mass distributions under a reasonable range, i.e., it will not introduce any mass irregularities as defined in ASCE 7-05 (ASCE 2005). The generated mass distributions are converted to

lateral force distributions based on the following equation provided in ASCE 7-05 (ASCE 2005):

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad \text{Eq. 5.1}$$

where: C_{vx} = vertical distribution factor,

w_i and w_x = the portion of the total effective seismic weight (W) located or assigned to Level i or x the level of interest

h_i and h_x = the height from the base to Level i or x

k = exponent related to the period of the structure

Based on the above procedure, a total of 500 lateral force distributions were generated for the nonlinear static analyses at each considered RC stiffness ratio.

5.2.2 Target Inter-story Drifts

During each nonlinear static analysis, the building model was monotonically pushed until maximum inter-story drift in the system reached a target value. In this investigation, the target inter-story drifts were determined to be 0.7, 2.5 and 5%, which are associated with immediate occupancy, life safety, and collapse prevention performance targets according to FEMA 356 (FEMA, 2000). It is noted that the above target inter-story drifts are used as damage indexes under the framework of performance-based earthquake engineering. Based on FEMA 356 (FEMA 2000), “immediate occupancy” corresponds to “Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members”; “life safety” represents to “Hinges form. Local buckling of some

beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture”; and “collapse prevention” represents “Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact”.

5.2.3 Response Quantities of Interest

The response quantities of interest include uniformity of inter-story drift distribution in the system; moment and shear demands on RC and axial load demand on the links connecting RC to existing frames. To quantify uniformity of the inter-story drift distributions, the following Drift Concentration Factor (DCF), is defined for each story:

$$DCF_i = \frac{\delta_i / h_{si}}{\Delta_{\text{roof}} / H} \Bigg|_{\max(\delta_1/h_{s1} \dots \delta_n/h_{sn}) = \text{target inter-story drift}} \quad \text{Eq. 5.2}$$

where DCF_i = drift concentration factor of the i^{th} story;

δ_i = inter-story displacement of the i^{th} story

h_{si} = height of the i^{th} story

Δ_{roof} = roof displacement relative to ground

H = height of the building

Based on the definition, DCF of a soft-story will be greater than 1.0 whereas DCF of a strong story will be smaller than 1.0. Moreover, if a building develops an ideally uniform drift distribution, DCF values for all stories are equal to 1.0.

5.2.4 Result Discussion

During the nonlinear static pushover analysis, the stiffness ratio, α , was varied from 0.003 to 5.0 to create RC from extremely flexible to ideally rigid. A total of 31 values of α were considered. At each considered value of α , a total of 500 sampled lateral force distributions were used in the nonlinear static analyses. The response quantities of interest were output when the maximum inter-story drift reached the expected levels associated with immediate occupancy, life safety, and collapse prevention performance targets. As a result, a total of 15,500 nonlinear static analyses were conducted for LA3-RA.

5.2.4.1 Discussion of DCF

Figures 5.7 to 5.9 show the DCF values associated with collapse prevention, life safety, and immediate occupancy, respectively. As shown, when α increases, DCF values approach 1.0 in all the considered cases, indicating that increasing stiffness of RC is an effective option to create more uniform inter-story drift distributions in the system. It is also important to note that DCF values are not very sensitive to α when α is smaller than 0.1; however when α is increased beyond 0.1, particularly in the range of 0.1 to 1.0, increasing α becomes very effective in reducing inter-story drift concentration in the system (as evidenced by the DCF values quickly converging to 1.0). Figure 5.10 further compares the maximum DCF values associated with the considered performance categories. As shown, among all the three considered performance categories, increasing stiffness of RC works most effectively in reducing drift concentration under the category of collapse prevention, which is evidenced by the faster drop of DCF values to 1.0.

Figure 5.7: DCF for collapse prevention category

Figure 5.8: DCF for life safety category

Figure 5.9: DCF for immediate occupancy category

Figure 5.10: Comparison of maximum DCF for the three performance categories

5.2.4.2 Discussion of Demands on RC

The results shown in Section 5.2.4.1 indicate that RC can be very promising in mitigating soft-story failure in low-rise steel moment frames. Successful implementation of RC

requires that it remains elastic during earthquake events. This section discusses the shear and bending moment demands on RC based on the results from nonlinear static analyses. It is noted that RC may also be subjected to axial load due to its self-weight and the shear force in the links when they are fixed to RC; however it was determined that axial force will not govern RC design.

The shear demand on RC depends on strength distribution along the vertical direction of the building since it redistributes story shears. For comparison, the shear demand on RC discussed in this section was normalized by shear yielding strength of the bottom story of the original structure (LA3-R), denoted as V_{1y} . Accordingly, the moment demand on RC was normalized by the overturning moment associated with the shear yielding strength of the bottom story of LA3-R.

The yield strength, V_{1y} , was determined by a pushover analysis on the bottom story of LA3-R. Figure 5.11 shows the result from the pushover analysis. The yield strength, V_{1y} , was taken to be at approximately 1% change in slope in the pushover curve. Figure 5.12 shows that at about 1% change in slope the base shear is approximately 550 kips. With the bottom story height of 13 feet, the corresponding overturning moment, $V_{1y}h_{s1}$, is 7,150 k-ft.

Figure 5.16: Result of pushover analysis of the bottom story of LA3-R

Figure 5.12: Change in slope of pushover curve

As discussed earlier, 500 nonlinear static analyses were conducted for each considered α value. In each of the 500 analyses, the shear and bending moment in each story of the RC were output as design demands. Then, the maximum values of the 500 design

demands were output for each story at each considered α value. It is recommended to design the RC based on the maximum values to ensure the RC remains elastic.

Figures 5.13 to 5.15 show the normalized maximum RC shear demands under the three performance categories. As shown, for the same α value, shear demand is the largest when the system is pushed to the inter-story drift associated with collapse prevention. Under all performance categories, the maximum shear demand always occurs at the soft-story (i.e., 1st story of LA3-RA) and it varies from 20% to 75% for collapse prevention (10% to 70% for life safety and 5% to 30% for immediate occupancy) over the specific range of α from 0.1 to 5.

Figure 5.13: RC shear demand (Performance category: collapse prevention)

Figure 5.17: RC shear demand (Performance category: life safety)

Figure 5.15: RC shear demand (Performance category: immediate occupancy)

Figures 5.16 to 5.18 show the maximum normalized RC bending moment demands under the three performance categories. As shown, the maximum bending moment demand always occurs at the soft-story and its adjacent story regardless of the performance category considered. In fact, the maximum bending moment occurs at the intersection of

the first-story and second-story RC segments. This is because the RC is loaded like a continuous beam and a large concentrated force is applied to RC by the link at the first floor level. Over the specific range of α , from 0.1 to 5, the maximum normalized bending moment varies from 20% to 75% for collapse prevention (10% to 70% for life safety and 5% to 30% for immediate occupancy).

Figure 5.16: RC moment demand (Performance category: collapse prevention)

Figure 5.17: RC moment demand (Performance category: life safety)

Figure 5.18: RC moment demand (Performance category: immediate occupancy)

5.2.4.3 Discussion of Demands on RC Links

The link members are expected to be subjected to large axial loads when transferring the interaction forces between RC and existing frame. In each nonlinear static analysis, the axial demand in the link at each floor level was output for comparison purposes. Similar to the shear and bending moment demands on the RC, the demands on links are also normalized by V_{1y} . For each considered α value, the maximum link axial demands from the 500 nonlinear static analyses were recorded. Figures 5.19 to 5.21 present the results of link axial demands under the three considered performance categories. As shown, the axial demand in the links connecting the soft-story (i.e., the first story in this case) is 2 to 3 times larger than those in the links at the other levels regardless of the performance categories considered. The link axial demands are comparable under collapse prevention and life safety performance targets; and larger than those under immediate occupancy

performance target. Over the specific range of α from 0.1 to 5, the maximum link axial demand varies from 40% to 100% for collapse prevention (20% to 100% for life safety and 5% to 45% for immediate occupancy).

Figure 5.19: RC axial demand (Performance category: collapse prevention)

Figure 5.20: RC axial demand (Performance category: life safety)

Figure 5.21: Link axial demand (Performance category: immediate occupancy)

5.3 Nonlinear Dynamic Analysis

In addition to the nonlinear static analyses conducted in Section 5.2, this thesis further investigated performance of LA3-RA through nonlinear dynamic analyses. In the analyses, different stiffness values were assigned to the RC to investigate effects of RC stiffness on system seismic performance. Moreover, to compare performance of the retrofitted system with and without extra energy dissipation capacity, LA3-RB is also investigated. The following describes the ground motions selected for the nonlinear dynamic analysis; response quantities of interest, and discussion of the results.

5.3.1 Selection of Ground Motions

The ground motions used in this research were those used for Los Angeles in the SAC Steel Project (available at www.sacsteel.org). Three different suites of horizontal earthquake ground motions were considered; each suite consists of 20 earthquake records. The first suite of ground motions are scaled to have a probability of exceedance of 10% in 50 years, i.e. to a level similar to a design basis earthquake (DBE); the second

suite of ground motions are scaled to have a probability of exceedance of 2% in 50 years, i.e., to a level similar to a Maximum Considered Earthquake (MCE); and the last suite of ground motions include near-fault records. The first two suites of ground motions were considered to evaluate if the retrofitted buildings could successfully satisfy the Best Safety Objective (BSO) according to FEMA 356; i.e., achieving collapse prevention under earthquakes associated with 2% in 50 years and life safety under earthquakes associated with 10% in 50 years. The near-fault ground motions were also included in this investigation since prior research reported that moment frame building structures may be more likely to fail due to soft-story failure under near-fault earthquakes (Alavi and Krawinkler 2004). Results associated with near-fault earthquakes help confirm if RC is also effective to reduce inter-story drift concentration in buildings under near-fault earthquakes. It is noted that FEMA 356 does not have any recommendations on the inter-story drift limit associated with near-fault ground motions; but in this thesis the inter-story drift limit associated with 2% in 50 years earthquake (i.e., 5.0%) is used for near-fault earthquakes. It is also recognized that all the earthquake records from each suite do not cause the same level of damage in the system. In this thesis, the specific earthquake records from each suite that cause inter-story drifts beyond the corresponding inter-story drift limit (e.g. causing more than 5% inter-story drift in earthquakes corresponding to 2% in 50 years) in LA3-R are defined as critical earthquake records. Tables 5.3 to 5.5 below provide details of the three sets of ground motions described above. The response spectra of the ground motions are provided in Appendix A.

Table 5-3: Los Angeles 10% in 50 years ground motions (adapted from SAC Steel Project website)

**Details of Los Angeles Ground Motions Having a
Probability of Exceedence of 10% in 50 Years**

SAC Name	Record	Earthquake Magnitude	Distance (km)	Scale Factor	Number of Points	DT (sec)	Duration (sec)	PGA (cm/sec²)
<u>LA01</u>	Imperial Valley, 1940, El Centro	6.9	10	2.01	2674	0.02	39.38	452.03
<u>LA02</u>	Imperial Valley, 1940, El Centro	6.9	10	2.01	2674	0.02	39.38	662.88
<u>LA03</u>	Imperial Valley, 1979, Array #05	6.5	4.1	1.01	3939	0.01	39.38	386.04
<u>LA04</u>	Imperial Valley, 1979, Array #05	6.5	4.1	1.01	3939	0.01	39.38	478.65
<u>LA05</u>	Imperial Valley, 1979, Array #06	6.5	1.2	0.84	3909	0.01	39.08	295.69
<u>LA06</u>	Imperial Valley, 1979, Array #06	6.5	1.2	0.84	3909	0.01	39.08	230.08
<u>LA07</u>	Landers, 1992, Barstow	7.3	36	3.2	4000	0.02	79.98	412.98
<u>LA08</u>	Landers, 1992, Barstow	7.3	36	3.2	4000	0.02	79.98	417.49
<u>LA09</u>	Landers, 1992, Yermo	7.3	25	2.17	4000	0.02	79.98	509.70
<u>LA10</u>	Landers, 1992, Yermo	7.3	25	2.17	4000	0.02	79.98	353.35
<u>LA11</u>	Loma Prieta, 1989, Gilroy	7	12	1.79	2000	0.02	39.98	652.49
<u>LA12</u>	Loma Prieta, 1989, Gilroy	7	12	1.79	2000	0.02	39.98	950.93
<u>LA13</u>	Northridge, 1994, Newhall	6.7	6.7	1.03	3000	0.02	59.98	664.93
<u>LA14</u>	Northridge, 1994, Newhall	6.7	6.7	1.03	3000	0.02	59.98	644.49
<u>LA15</u>	Northridge, 1994, Rinaldi RS	6.7	7.5	0.79	2990	0.005	14.945	523.30
<u>LA16</u>	Northridge, 1994, Rinaldi RS	6.7	7.5	0.79	2990	0.005	14.945	568.58
<u>LA17</u>	Northridge, 1994, Sylmar	6.7	6.4	0.99	3000	0.02	59.98	558.43
<u>LA18</u>	Northridge, 1994, Sylmar	6.7	6.4	0.99	3000	0.02	59.98	801.44
<u>LA19</u>	North Palm Springs, 1986	6	6.7	2.97	3000	0.02	59.98	999.43
<u>LA20</u>	North Palm Springs, 1986	6	6.7	2.97	3000	0.02	59.98	967.61

Table 5-4: Los Angeles 2% in 50 years ground motions (adapted from SAC Steel Project website)

**Details of Los Angeles Ground Motions Having a
Probability of Exceedence of 2% in 50 Years**

SAC Name	Record	Earthquake Magnitude	Distance (km)	Scale Factor	Number of Points	DT (sec)	Duration (sec)	PGA (cm/sec²)
<u>LA21</u>	1995 Kobe	6.9	3.4	1.15	3000	0.02	59.98	1258.00
<u>LA22</u>	1995 Kobe	6.9	3.4	1.15	3000	0.02	59.98	902.75
<u>LA23</u>	1989 Loma Prieta	7	3.5	0.82	2500	0.01	24.99	409.95
<u>LA24</u>	1989 Loma Prieta	7	3.5	0.82	2500	0.01	24.99	463.76
<u>LA25</u>	1994 Northridge	6.7	7.5	1.29	2990	0.005	14.945	851.62
<u>LA26</u>	1994 Northridge	6.7	7.5	1.29	2990	0.005	14.945	925.29
<u>LA27</u>	1994 Northridge	6.7	6.4	1.61	3000	0.02	59.98	908.70
<u>LA28</u>	1994 Northridge	6.7	6.4	1.61	3000	0.02	59.98	1304.10
<u>LA29</u>	1974 Tabas	7.4	1.2	1.08	2500	0.02	49.98	793.45
<u>LA30</u>	1974 Tabas	7.4	1.2	1.08	2500	0.02	49.98	972.58
<u>LA31</u>	Elysian Park (simulated)	7.1	17.5	1.43	3000	0.01	29.99	1271.20
<u>LA32</u>	Elysian Park (simulated)	7.1	17.5	1.43	3000	0.01	29.99	1163.50
<u>LA33</u>	Elysian Park (simulated)	7.1	10.7	0.97	3000	0.01	29.99	767.26
<u>LA34</u>	Elysian Park (simulated)	7.1	10.7	0.97	3000	0.01	29.99	667.59
<u>LA35</u>	Elysian Park (simulated)	7.1	11.2	1.1	3000	0.01	29.99	973.16
<u>LA36</u>	Elysian Park (simulated)	7.1	11.2	1.1	3000	0.01	29.99	1079.30
<u>LA37</u>	Palos Verdes (simulated)	7.1	1.5	0.9	3000	0.02	59.98	697.84
<u>LA38</u>	Palos Verdes (simulated)	7.1	1.5	0.9	3000	0.02	59.98	761.31
<u>LA39</u>	Palos Verdes (simulated)	7.1	1.5	0.88	3000	0.02	59.98	490.58
<u>LA40</u>	Palos Verdes (simulated)	7.1	1.5	0.88	3000	0.02	59.98	613.28

Table 5-5: Near-Fault ground motions (adapted from SAC Steel Project website)

**Details of Near-Field Ground Motions Derived
from Historical Acceleration Records**

SAC Name	Record	Earthquake Magnitude	Distance (km)
<u>NF01</u>	Tabas, 1978	7.4	1.2
<u>NF02</u>	Tabas, 1978	7.4	1.2
<u>NF03</u>	Loma Prieta, 1989, Los Gatos	7	3.5
<u>NF04</u>	Loma Prieta, 1989, Los Gatos	7	3.5
<u>NF05</u>	Loma Prieta, 1989, Lex. Dam	7	6.3
<u>NF06</u>	Loma Prieta, 1989, Lex. Dam	7	6.3
<u>NF07</u>	C. Mendocino, 1992, Petrolia	7.1	8.5
<u>NF08</u>	C. Mendocino, 1992, Petrolia	7.1	8.5
<u>NF09</u>	Erzincan, 1992	6.7	2
<u>NF10</u>	Erzincan, 1992	6.7	2
<u>NF11</u>	Landers, 1992	7.3	1.1
<u>NF12</u>	Landers, 1992	7.3	1.1
<u>NF13</u>	Nothridge, 1994, Rinaldi	6.7	7.5
<u>NF14</u>	Nothridge, 1994, Rinaldi	6.7	7.5
<u>NF15</u>	Nothridge, 1994, Olive View	6.7	6.4
<u>NF16</u>	Nothridge, 1994, Olive View	6.7	6.4
<u>NF17</u>	Kobe, 1995	6.9	3.4
<u>NF18</u>	Kobe, 1995	6.9	3.4
<u>NF19</u>	Kobe, 1995, Takatori	6.9	4.3
<u>NF20</u>	Kobe, 1995, Takatori	6.9	4.3

5.3.2 Result Discussion

During the nonlinear dynamic analysis for each earthquake, the maximum inter-story drift in the system was output. It is noted that complete results of maximum inter-story drift from each earthquake under each considered RC stiffness are provided in Appendix

B. The median of the maximum inter-story drifts from all earthquakes in the considered suite is selected as a response quantity of interest and discussed in detail in the following section. Moreover, demands on the RC and links and residual drifts are discussed. At the end, the results from LA3-RA are compared to those from LA3-RB to discuss the effect of links with energy dissipation characteristic.

5.3.2.1 Discussion on Inter-story Drift

The median of the maximum drifts obtained from all the ground motions under the 10% in 50 years suite are presented in Figure 5.22. As shown, the median of the maximum drifts reduces from 5.1% to 2.5% (limit associated with life safety) when α is increase from 0.003 to 5. It is noted that among the 20 ground motions under this category, 18 ground motions are critical earthquake records; and 10 out of the 18 ground motions cause inter-story drift in LA3-R even greater than 5.0%, i.e., the limit associated with collapse prevention. In all the critical cases, LA3-RA experiences a soft-story failure at the bottom story. To better understand the contribution of RC, the median of the maximum inter-story drifts resulting from the 18 critical earthquake records is shown in Figure 5.23. As shown, the median of the maximum drifts from the critical ground motions reduces from 5.6% to 2.8% which is still beyond the limit associated with life safety, indicating either a stiffer RC or extra energy dissipation device is required in the system to achieve the life safety objective.

Figure 5.22: Median of the maximum inter-story drifts from all ground motions (10% in 50 years)

Figure 5.23: Median of the maximum inter-story drifts from critical ground motions (10% in 50 years)

The median of the maximum drifts obtained from all the ground motions under the 2% in 50 years suite are presented in Figures 5.24. In this case, the maximum drift of LA3-RA exceeds the limit associated with collapse prevention in all ground motions when α is small. As shown, the median of the maximum drifts reduces significantly from 13% to

5.8% when α increased over the considered range. However, either a stiffer RC or extra energy dissipation device is required in the system to achieve the collapse prevention objective

Figure 5.24: Median of the maximum inter-story drifts from all ground motions (2% in 50 years)

The median of the maximum drifts obtained from all the near-fault ground motions are presented in Figure 5.25. The median of the maximum inter-story drifts resulting from the 16 critical earthquake records under this suite is shown in Figure 5.26. As shown, stiffer RC successfully reduces the inter-story drift when the system is subjected to near-fault ground motions. Specifically, the median of the maximum inter-story drifts reduces from approximately 8.7% to 4.4% (below the limit associated with collapse prevention) when the critical ground motions in this suite are considered.

Figure 5.25: Median of the maximum inter-story drifts from all ground motions (near-fault earthquakes)

Figure 5.26: Median of the maximum inter-story drifts from critical ground motions (near-fault earthquakes)

In summary, stiffer RC does enhance the performance of LA3-RA under all the three suites of ground motions. However, only implementing RC with considered stiffness may not be sufficient to control performance of the system to meet the requirement of some performance objectives (e.g. under earthquakes associated with a probability of

exceedance of 2% in 50 years); stiffer RC or extra energy dissipation device may be needed to further enhance system seismic performance.

5.3.2.2 Demands on RC

This section presents the median values of the maximum shear and bending moment demands from each ground motion of the three sets of ground motions. For comparison purposes, the shear and bending moment demands are normalized by V_{ly} and $V_{ly}h_{s1}$, respectively. Figures 5.27, 5.29, and 5.31 present shear demands from each earthquake suite; Figures 5.28, 5.30 and 5.32 present bending moment demands from each earthquake suite.

Compared with the normalized RC shear demands from nonlinear pushover analyses (see Figures 5.13 to 5.15), the RC shear demands from nonlinear dynamic analyses are higher in every story, particularly, when α is greater than 0.1. When α is equal to 5.0, the nonlinear dynamic analysis results are 30% higher than those from nonlinear static analysis. Similarly, the normalized RC bending moment demands from nonlinear dynamic analyses are also higher than those from nonlinear pushover analysis (see Figures 5.16 to 5.18). Larger RC demands are observed from the results from nonlinear dynamic analysis because the FE models used in nonlinear dynamic analysis can capture the rupture behavior of beam-to-column connections and P-Delta effect. When connection rupture failures occur, sudden strength and stiffness degradations are expected in the system, resulting in excessive demands on RC. The observation that the demands from the earthquakes corresponding to 2% in 50 years are highest is consistent with the fact that beam-to-column connections are more likely to rupture in a stronger earthquake. Although the normalized RC bending moment demands from nonlinear dynamic analyses

are higher, both nonlinear static analysis and nonlinear dynamic analysis results show the same trend, i.e., results for the RC segments over the bottom two stories overlap with each other.

Figure 5.27: Normalized median shear demand in RC (Earthquakes corresponding to 10% in 50 years)

Figure 5.28: Normalized median bending moment demand in RC (Earthquakes corresponding to 10% in 50 years)

Figure 5.29: Normalized median shear demand in RC (Earthquakes corresponding to 2% in 50 years)

Figure 5.30: Normalized median bending moment demand in RC (Earthquakes corresponding to 2% in 50 years)

Figure 5.31: Normalized median shear demand in RC (Near-Fault Earthquakes)

Figure 5.32: Normalized median bending moment demand in RC (Near-Fault Earthquakes)

5.3.2.3 Link Demands

The axial demand on the links was also of interest. Similarly, it was normalized by V_{1y} .

Figures 5.33 to 5.35 show the median normalized axial demand for the links in the three suites of ground motions. As shown, under all three earthquake suites, larger axial force

is expected in all links when a stiffer RC is used. The link demands predicted from the ground motions corresponding to 10% in 50 years and from the near-fault ground motions are very similar but they are higher than the nonlinear pushover analysis results (see Figures 5.20 to 5.21). The link demands from the earthquakes corresponding to 2% in 50 years are the largest since the beam-to-column connections are more likely to rupture in this level of earthquakes and the links will be further loaded when the ruptures result in strength and stiffness degradations.

Figure 5.33: Normalized median axial demand in links (Earthquakes corresponding to 10% in 50 years)

Figure 5.34: Normalized median axial demand in links (Earthquakes corresponding to 2% in 50 years)

Figure 5.35: Normalized median axial demand in links (Near-Fault Earthquakes)

5.3.2.4 Impact of Links with Energy Dissipation Capacity

In an attempt to further enhance the system seismic performance, the links are used with energy dissipation capacity. This section conducts the nonlinear dynamic analyses in LA3-RB. The links were designed based on the maximum axial demand from all three

stories obtained from the nonlinear pushover analysis. Based on the interaction equation considering axial load and bending moment in wide-flange cross-section presented in (Bruneau et al., 2011), the links were designed such that they can provide full moment capacity when transferring axial loads. As a result, two W30X132 sections spanning 5 feet in parallel are selected as the link component at each floor level.

Figures 5.36 to 5.38 compare median of the maximum drifts from LA3-RA and LA3-RB under critical ground motions in the three suites. It was found that, although LA3-RA and LA3-RB generally tend to provide comparable drift results when α is relatively small, the results from LA3-RB are lower than those from LA3-RA when α is greater than 1.5.

Figure 5.36: Drift comparison of LA3-RA and LA3-RB (Earthquakes corresponding to 10% in 50 years)

Figure 5.37: Drift comparison of LA3-RA and LA3-RB (Earthquakes corresponding to 2% in 50 years)

Figure 5.38: Drift comparison of LA3-RA and LA3-RB (Near-Fault Earthquakes)

Figures 5.39 to 5.44 present the normalized median of the maximum shear and bending moment demands in RC in LA3-RB under each considered earthquake suite. Compared with the results shown in Figures 5.27 and 5.29 for LA3-RA, it is found under earthquakes associated with 2% and 10% in 50 years that LA3-RB have RC shear

demands in the second and third stories similar to those of LA3-RA; however the RC shear demands in the first story is larger in LA3-RB. Based on the results shown in Figures 5.31 and 5.43, it is found that in near-field ground motions LA3-RB has a larger demand in the first story and smaller demands in the second and third stories than LA3-RA. Based on the results shown in Figure 5.40, 5.42 and 5.44 for LA3-RB and Figures 5.28, 5.30 and 5.32 for LA3-RA, it is found that moment demand at the third story is similar in both models under all three suites of earthquakes; however, LA3-RB has higher moment demands on the other two stories.

Figure 5.39: Normalized median shear demand in RC of LA3-RB (Earthquakes corresponding to 10% in 50 years)

Figure 5.40: Normalized median bending moment demand in RC of LA3-RB (Earthquakes corresponding to 10% in 50 years)

Figure 5.41: Normalized median shear demand in RC of LA3-RB (Earthquakes corresponding to 2% in 50 years)

Figure 5.42: Normalized median bending moment demand in RC of LA3-RB (Earthquakes corresponding to 2% in 50 years)

Figure 5.43: Normalized median shear demand in RC of LA3-RB (Near-Fault Earthquakes)

Figure 5.44: Normalized median bending moment demand in RC of LA3-RB (Near-Fault Earthquakes)

The normalized median values of the maximum axial load demands, maximum shear demands and maximum bending moment demands for the links in LA3-RB are shown in Figures 5.45 to 5.53 for each considered suite of ground motions. Compared with the axial demands shown in Figures 5.33, 5.34 and 5.35 for LA3-RA, it is consistently observed that the axial demands in LA3-RB are similar.

Results shown in Figures 5.47, 5.49 and 5.51 indicate that the bending moment demands converge to the same level under the different suites of earthquakes. This is due to formation of plastic hinges at the fixed end of each link. The fact that the moment demands in the links at different levels converge to the same level indicate that formation of a sway mechanism in the system is achieved. A similar observation can be obtained from shear force demands (see Figures 5.46, 5.48 and 5.50).

Figure 5.45: Normalized median axial demand in links of LA3-RB (Earthquakes corresponding to 10% in 50 years)

Figure 5.46: Normalized median shear demand in links of LA3-RB (Earthquakes corresponding to 10% in 50 years)

Figure 5.47: Normalized median bending moment demands in links of LA3-RB (Earthquakes corresponding to 10% in 50 years)

Figure 5.48: Normalized median axial demand in links of LA3-RB (Earthquakes corresponding to 2% in 50 years)

Figure 5.49: Normalized median shear demand in links of LA3-RB (Earthquakes corresponding to 2% in 50 years)

Figure 5.50: Normalized median bending moment demands in links of LA3-RB (Earthquakes corresponding to 2% in 50 years)

Figure 5.51: Normalized median axial demand in links of LA3-RB (Near-Fault Earthquakes)

Figure 5.52: Normalized median shear demand in links of LA3-RB (Near-Fault Earthquakes)

Figure 5.53: Normalized median bending moment demands in links of LA3-RB (Near-Fault Earthquakes)

Overall, the system performance is enhanced by introducing the links with energy dissipation capacity. In all three ground motion sets the inter-story drift targets are all achieved in LA3-RB, this is not the case in LA3-RA.

5.3.2.5 Residual Drift

The median residual drifts from all ground motions in the same suite are discussed in this section. Figures 5.54 and 5.55 show the median residual drifts of LA3-RA and LA3-RB under the three earthquake suites. It is shown that stiffer RC tends to reduce the permanent deformation in all cases. Moreover, the median residual drifts from LA3-RB are generally slightly lower than those from LA3-RA, indicating energy dissipation helps reduce the permanent deformation caused by earthquakes.

Figure 5.54: Median residual drift in LA3-RB

Figure 5.55: Median residual drift in LA3-RA

5.4 Demonstration Retrofit using Reinforced Concrete RC

Results shown in Sections 5.2 and 5.3 demonstrate the adequacy of RC in retrofit of low-rise steel moment frame buildings. This section implements a reinforced concrete RC in LA3-RB. The retrofitted system was analyzed using nonlinear dynamic analysis to

confirm the performance predicted in Section 5.3 is achievable with the designed reinforced concrete RC. The model was subjected to the three ground motion suites used in the previous analysis. The design of the reinforced concrete wall was based on the smallest α value required to meet the three performance objectives of each of the ground motion suites and also based on the corresponding loading demands. The governing α value was 5.0 and the largest normalized median moment and shear demands were 227%. It is noted that two RC walls were designed and placed on both sides of the moment frame. The reason for designing two walls instead of one is because there are two links being used in parallel and using two walls on both sides of the building will allow for practical connections. It was found during the designing of the wall that the moment demand controlled the dimensions required.

The required dimensions of the RC, shown in Figure 5.56, are 68" by 36" with 26 #10 bars on both ends of the wall. The required shear reinforcement was calculated for the maximum demand; the resulting spacing of #4 ties and stirrups was calculated to be 5.5" on center. The materials used for the modeling of the confined, unconfined, and longitudinal steel are Mander model, Todeshini Parabolic model, and the Giuffr-Menegotto-Pinto model respectively with f'_c of 4ksi and f_y of 60 ksi. In the OpenSees model, the reinforced concrete RC was considered using FBE.

Figure 5.56: Reinforced concrete RC cross-section design for LA3-RB

The results obtained for the nonlinear dynamic analysis of LA3-RB with the reinforced concrete RC are compared with the results obtained for LA3-RB with elastic elements used for the RC in Tables 5.6 to 5.8 for each of the ground motion suites. It is noted that the results presented for the reinforced concrete RC are for both RC combined. As shown in the tables, the results match very closely for both the RC and link demands when the RC is modeled with elastic elements or reinforced concrete elements. This suggests that satisfactory results from the simplified RC model are achievable in the system with reinforced concrete RC.

Table 5-6: LA3-RB-Comparison of simplified RC and reinforced concrete RC for 10% in 50 years suite

Comparison of Drifts and Loading Demands for 10% in 50 years suite		
$\alpha = 5.0$	Elastic RC	Reinforced Concrete RC
Median Drifts from Critical Earthquakes (%)	2.3	2.1
Median Drifts from all Earthquakes (%)	2.2	2.0
Normalized Median Moment on RC (%)	190	191
Normalized Median Shear on RC (%)	190	191
Normalized Median Moment on Links (%)	33	31
Normalized Median Shear on Links (%)	85	82
Normalized Median Axial on Links (%)	90	93

Table 5-7: LA3-RB-Comparison of simplified RC and reinforced concrete RC for 2% in 50 years suite

Comparison of Drifts and Loading Demands for 2% in 50 years suite		
$\alpha = 5.0$	Elastic RC	Reinforced Concrete RC
Median Drifts from Critical Earthquakes (%)	4.3	4.0
Median Drifts from all Earthquakes (%)	4.3	4.0
Normalized Median Moment on RC (%)	227	221
Normalized Median Shear on RC (%)	227	221
Normalized Median Moment on Links (%)	36	34
Normalized Median Shear on Links (%)	94	92
Normalized Median Axial on Links (%)	117	116

Table 5-8: LA3-RB-Comparison of simplified RC and reinforced concrete RC for Near-Fault suite

Comparison of Drifts and Loading Demands for Near-Fault suite		
$\alpha = 5.0$	Elastic RC	Reinforced Concrete RC
Median Drifts from Critical Earthquakes (%)	3.5	3.2
Median Drifts from all Earthquakes (%)	2.8	2.4
Normalized Median Moment on RC (%)	195	188
Normalized Median Shear on RC (%)	195	188
Normalized Median Moment on Links (%)	33	32
Normalized Median Shear on Links (%)	86	85
Normalized Median Axial on Links (%)	91	94

6.0 Seismic Performance Evaluation of Nine-Story Building

6.1 Impact of RC on Modal Properties

To consider the effect of RC on modal properties of the nine-story building, eigenvalue analyses were performed. For the nine-story structure two cases were considered: LA9-RA in which the links connecting the moment frame to the RC are pinned on both ends; and LA9-RB in which the links are pinned on the moment frame and fixed on the RC. In LA9-RB the fixed connections on the RC serve to dissipate energy. Three different levels of RC stiffness (extremely flexible, moderately rigid, and rigid) are considered in evaluating the RC effects on modal properties. The three moments of inertia corresponding to the three RC stiffness levels were calculated using Equation 4.2 with the following α values: 0.02, 1.29, and 150. The resulting moments of inertia were 1600 in⁴, 105,000 in⁴, and 12.3 million in⁴ for the low, median, and high RC stiffness levels respectively.

A summary of the first three mode periods for LA9-RA is shown in Table 6.1. Addition of the RC, as shown, lowers the period from low RC stiffness to high RC stiffness but not to a significant degree. This suggests that the RC does not elevate the seismic forces on the system. Figures 6.1 through 6.3 compare the first three mode shapes for the three different RC stiffness levels. It is evident that that there is no significant impact on mode shapes due to the variation of RC stiffness.

Table 6-1: Period comparison for three levels of stiffness of RC in LA9-RA

Mode	Period (sec) for Three Level of Stiffness		
	Low Stiffness ($\alpha=0.02$)	Median Stiffness ($\alpha=1.29$)	High Stiffness ($\alpha=150$)
1 st	2.84	2.83	2.61
2 nd	0.99	0.99	0.86
3 rd	0.54	0.54	0.40

Figure 6.1: 1st mode shapes comparison of LA9-RA with different RC stiffness values

Figure 6.2: 2nd mode shapes comparison of LA9-RA with different RC stiffness values

Figure 6.3: 3rd mode shapes comparison of LA9-RA with different RC stiffness values

Table 6.2 summarizes the periods of LA9-RB with different RC stiffness values. As shown, stiffer RC tends to reduce the periods in all three modes and to a higher degree for the fundamental periods. Comparing the results from Table 6.1 to the results in Table 6.2 it is found that the periods get shorter when the links are used with energy dissipation

capacity. The reason that LA9-RB periods get shorter more than that of LA9-RA is due to the extra stiffness provided by the fixed link end on the RC. Figures 6.4 through 6.6 compare the effect of RC stiffness level on mode shapes of LA9-RB. As shown, the impact of RC stiffness on mode shape is also negligible.

Table 6-2: Period comparison for three levels of stiffness of RC in LA9-RB

Mode	Period (sec) for Three Level of Stiffness		
	Low Stiffness ($\alpha=0.02$)	Median Stiffness ($\alpha=1.29$)	High Stiffness ($\alpha=150$)
1 st	2.82	2.61	2.27
2 nd	0.97	0.90	0.67
3 rd	0.53	0.46	0.29

Figure 6.4: 1st mode shapes comparison of LA9-RB with different RC stiffness values

Figure 6.5: 2nd mode shapes comparison of LA9-RB with different RC stiffness values

Figure 6.6: 3rd mode Shapes comparison of LA9-RB with different RC stiffness values

6.2 Nonlinear Static Pushover Analysis

Similar to LA3-RA, nonlinear static pushover analyses were also conducted on LA9-RA to evaluate the performance of the building. A total of 31 values of α ranging from 0.02 to 150, were used. The lateral load distribution used in this analysis was obtained using

the same method presented in Section 5.2.1. However, only 250 Monte Carlo simulations were performed and analyzed due to the extensive computational time. The target inter-story drifts of interest are also the same as those presented in Section 5.2.2.

6.2.1 Result Discussion

The response quantities of interest were output when the maximum inter-story drift reached the expected levels associated with immediate occupancy, life safety, and collapse prevention performance targets. In the following sections the results from the nonlinear static pushover analyses are provided among these are DCF, demands on RC, and demands on RC links.

6.2.1.1 Discussion of DCF

Based on the nonlinear static pushover analyses on LA9-RA it was found that at low RC stiffness the first story always failed in a soft-story mechanism. For a soft-story failure on the first floor the maximum DCF corresponding to a perfect soft-story failure is 6.77, i.e., all deformation demands concentrate on the first story and the upper stories remain completely elastic. A perfect sway mechanism correlates to a DCF of 1 at each story.

Considering collapse prevention as the performance objective, Figure 6.7 presents DCF, of each story of LA9-RA at each RC stiffness level being considered. It is evident that at low RC stiffness the drift concentrates mainly on the first story causing a soft-story failure mechanism. At higher RC stiffness the deformation demands are more evenly distributed along the height of the building causing a sway-failure mechanism. It is important to note that at α values greater than 10, DCF values do not change much with higher RC stiffness. Figures 6.8 and 6.9 have the same general trend, the DCF values converge to a value of 1 and the lateral load distribution is much less significant at higher

RC stiffness. Figure 6.10 further compares the maximum DCF values of the three considered performance categories. Under the three performance objectives, higher RC stiffness consistently reduces drift concentration at the bottom story.

Figure 6.7: DCF for collapse prevention category

Figure 6.8: DCF for life safety category

Figure 6.9: DCF for immediate occupancy category

Figure 6.10: Comparison of maximum DCF for the three performance categories

6.2.1.2 Discussion of Demands on RC

Based on the results of the previous section the addition of a RC can be very promising in mitigating soft-story failure in mid-rise steel moment frames. The success of RC in

mitigating soft-story failures required however that the RC remains elastic during seismic events. This section discusses the shear and bending moment demands on RC based on the nonlinear static pushover analysis. It is noted that RC may also be subjected to axial loads due to its self-weight and the shear force in the links when they are fixed to RC; however it was determined that axial force will not govern RC design.

The shear demand of the RC is dependent on the strength distribution along the height of the building since it redistributes story shear. Similar to the nonlinear static pushover analyses of LA3-RA, the shear demands on the RC of LA9-RA were presented in this section are normalized by the shear yielding strength of the bottom story of the original structure (LA9-R), denoted as V_{1y} . The moment demand on the RC was normalized by the overturning moment associated with the shear yielding strength of the bottom story of LA9-R.

The yield strength, V_{1y} , was determined by a pushover analysis on the bottom story of LA9-R. Figure 6.11 shows the result from the pushover analysis. The yield strength, V_{1y} , was taken to be at approximately 1% change in slope in the pushover curve. Figure 6.12 shows that at about 1% change in slope the base shear is approximately 920 kips. With the bottom story height of 18 feet, the corresponding overturning moment, $V_{1y}h_{s1}$, is 16,560 k-ft.

Figure 6.11: Result of pushover analysis of the bottom story of LA9-RA

Figure 6.18: Change in slope of pushover curve

As discussed earlier, 250 nonlinear static analyses were conducted for each considered α value. In each of the 250 analyses, the shear and bending moment in each stories of the RC were output as design demands. Then, the maximum values of the 250 design demands were output for each story at each considered α value. It is recommended to design the RC based on the maximum values to ensure the RC remains elastic.

Figures 6.13 to 6.15 show the normalized maximum RC shear demands under the three performance categories. As shown, for the same α value, shear demand is the largest when the system is pushed to the inter-story drift associated with collapse prevention. Under all performance categories, the maximum shear demand always occurs at the soft-story (i.e., 1st story of LA9-RA) and it varies from 25% to 175% for collapse prevention (20% to 150% for life safety and 5% to 55% for immediate occupancy) over the specific range of α from 0.1 to 10.

Figure 6.19: RC shear demand (Performance category: collapse prevention)

Figure 6.110: RC shear demand (Performance category: life safety)

Figure 6.111: RC shear demand (Performance category: immediate occupancy)

Figures 6.16 to 6.18 show the maximum normalized RC bending moment demands under the three performance categories. As shown, the maximum bending moment demand always occurs at the soft-story and its adjacent story regardless of the performance category considered. In fact, the maximum bending moment occurs at the intersection of

the first-story and second-story RC segments. This is because the RC is loaded like a continuous beam and a large concentrated force is applied to RC by the link at the first floor level. Over the specific range of α from 0.1 to 10, the maximum normalized bending moment varies from 25% to 175% for collapse prevention (20% to 150% for life safety and 5% to 55% for immediate occupancy).

Figure 6.112: RC moment demand (Performance category: collapse prevention)

Figure 6.113: RC moment demand (Performance category: life safety)

Figure 6.114: RC moment demand (Performance category: immediate occupancy)

6.2.1.3 Discussion of Demands on RC Links

The link members are expected to be subjected to large axial loads when transferring the interaction forces between RC and existing frame. In each nonlinear static analysis, the axial demand in the link at each floor level was output for comparison purposes. Similar to the shear and bending moment demands on the RC, the demands on links are also normalized by V_{1y} . For each considered α value, the maximum link axial demands from the 250 nonlinear static analyses were recorded. Figures 6.19 to 6.21 present the results of link axial demands under the three considered performance categories. As shown, the axial demand in the links connecting the soft story (i.e., the first-story in this case) is 3 to 4 times larger than those in the links at the other levels regardless of the performance categories considered. The link axial demands are comparable under collapse prevention and life safety performance targets; and larger than those under immediate occupancy performance target. Over the specific range of α from 0.1 to 10, the maximum link axial demand varies from 50% to 200% for collapse prevention (30% to 190% for life safety and 10% to 80% for immediate occupancy).

Figure 6.115: RC axial demand (Performance category: collapse prevention)

Figure 6.20: RC axial demand (Performance category: life safety)

Figure 6.21: Link axial demand (Performance category: immediate Occupancy)

6.3 Nonlinear Dynamic Analysis

In addition to the nonlinear static analyses conducted in Section 6.2, this thesis further investigated performance of LA9-RA through nonlinear dynamic analyses. In the analyses, different stiffness values were assigned to the RC to investigate effects of RC stiffness on system seismic performance. Moreover, to compare performance of the retrofitted system with and without extra energy dissipation capacity, LA9-RB is also investigated. The same three ground motion suites described in Section 5.3.1 were used in this analysis. The following sections describe the response quantities of interest and discussion of the results.

6.3.1 Result Discussion

The maximum inter-story drift in the system was output for the nonlinear dynamic analysis for each earthquake. It is noted that complete results of maximum inter-story

drift from each earthquake under each considered RC stiffness are provided in Appendix B. The median of the maximum inter-story drifts from all earthquakes in the considered suite is selected as a response quantity of interest and discussed in detail in the following section. Moreover, demands on the RC and links and residual drifts are discussed. At the end, the results from LA9-RA are compared to those from LA9-RB to discuss the effect of links with energy dissipation characteristic.

6.3.1.1 Discussion on Inter-story Drift

The median of the maximum drifts obtained from all the ground motions under the 10% in 50 years suite are presented in Figure 6.22. As shown, the median of the maximum drifts from all ground motions reduce from 5.3% to 1.8% (limit associated with life safety) when α is increase from 0.02 to 150. It is noted that among the 20 ground motions under this category, 19 ground motions are critical earthquake records; and 10 out of the 19 ground motions cause inter-story drift in LA9-R even greater than 5.0%, i.e., the limit associated with collapse prevention. In all the critical cases, LA9-RA experiences a soft-story failure at the bottom story. To better understand the contribution of RC, the median of the maximum inter-story drifts resulting from the 19 critical earthquake records is shown in Figure 6.23. As shown, the median of the maximum drifts from the critical ground motions reduce from 5.3% to 1.8%, which is below the limit associated with life safety, indicating that that the RC significantly enhanced the performance of the building. Life safety target drift was met at a α value of about 3.0.

Figure 6.22: Median of the maximum inter-story drifts from all ground motions (10% in 50 years)

Figure 6.23: Median of the maximum inter-story drifts from critical ground motions (10% in 50 years)

The median of the maximum drifts obtained from all the ground motions under the 2% in 50 years suite are presented in Figures 6.24. In this case, the maximum drift of LA9-RA exceeds the limit associated with collapse prevention in all ground motions when α is small. As shown, the median of the maximum drifts reduces significantly from 13.5% to

3.0% when α increased over the considered range. In this suite of ground motions the target drift associated with collapse prevention is met at a α value of about 1.0.

Figure 6.24: Median of the maximum inter-story drifts from all ground motions (2% in 50 years)

The median of the maximum drifts obtained from all the near-fault ground motions are presented in Figure 6.25. The median of the maximum inter-story drifts resulting from the 14 critical earthquake records under this suite is shown in Figure 6.26. As shown, stiffer RC successfully reduces the inter-story drift when the system is subjected to near-fault ground motions. Specifically, the median of the maximum inter-story drifts reduces from approximately 12.0% to 2.5% (below the limit associated with collapse prevention) when the critical ground motions in this suite are considered.

Figure 6.25: Median of the maximum inter-story drifts from all ground motions (Near-Fault Earthquakes)

Figure 6.26: Median of the maximum inter-story drifts from critical ground motions (Near-Fault Earthquakes)

In summary, stiffer RC does enhance the performance of LA9-RA under all the three suites of ground motions. In all three cases implementation of RC was enough to reduce the drift demands below the considered three target drifts. In comparison to LA3-RA the required α value in LA9-RA to drop below the target drifts is much lower.

6.3.1.2 Demands on RC

This section presents the median values of the maximum shear and bending moment demands from each ground motion of the three sets of ground motions. For comparison purposes, the shear and bending moment demands are normalized by V_{ly} and $V_{ly}h_{sl}$, respectively. Figures 6.27, 6.29, and 6.31 present shear demands from each earthquake suite; Figures 6.28, 6.30 and 6.32 present bending moment demands from each earthquake suite.

Compared with the normalized RC shear demands from nonlinear pushover analyses (see Figures 6.13 to 6.15), the RC shear demands from nonlinear dynamic analyses are higher in every story, particularly, when α is greater than 0.1. When α is equal to 10.0, the nonlinear dynamic analysis results are 50% higher than those from nonlinear static analysis. Similarly, the normalized RC bending moment demands from nonlinear dynamic analyses are also higher than those from nonlinear pushover analysis (see Figures 6.16 to 6.18). Larger RC demands are observed from the results from nonlinear dynamic analysis because the FE models used in nonlinear dynamic analysis can capture the rupture behavior of beam-to-column connections and P-Delta effect. When connection rupture failures occur, sudden strength and stiffness degradations are expected in the system, resulting in excessive demands on RC. The observation that the demands from the earthquakes corresponding to 2% in 50 years are highest is consistent with the fact that beam-to-column connections are more likely to rupture in a stronger earthquake. Although the normalized RC bending moment demands from nonlinear dynamic analyses are higher, both nonlinear static analysis and nonlinear dynamic analysis results show the same trend.

Figure 5.27: Normalized median shear demand in RC (Earthquakes corresponding to 10% in 50 years)

Figure 6.28: Normalized median bending moment demand in RC (Earthquakes corresponding to 10% in 50 years)

Figure 6.29: Normalized median shear demand in RC (Earthquakes corresponding to 2% in 50 years)

Figure 6.30: Normalized median bending moment demand in RC (Earthquakes corresponding to 2% in 50 years)

Figure 6.31: Normalized median shear demand in RC (Near-Fault Earthquakes)

Figure 6.32: Normalized median bending moment demand in RC (Near-Fault Earthquakes)

6.3.1.3 Link Demands

The axial demand on the links was also of interest. Similarly, it was normalized by V_{1y} .

Figures 6.33 to 6.35 show the median normalized axial demand for the links in the three suites of ground motions. As shown, under all three earthquake suites, larger axial force

is expected in all links when a stiffer RC is used. The link demands predicted from the ground motions corresponding to 10% in 50 years and from the 2% in 50 years ground motions are very similar to the nonlinear pushover analysis results (see Figures 6.20 to 6.21). The link demands from the earthquakes corresponding to near-fault ground motions are smaller than those predicted in the pushover analysis for collapse prevention category.

Figure 6.33: Normalized median axial demand in links (Earthquakes corresponding to 10% in 50 years)

Figure 6.34: Normalized median axial demand in links (Earthquakes corresponding to 2% in 50 years)

Figure 6.35: Normalized median axial demand in links (Near-Fault Earthquakes)

6.3.1.4 Impact of Links with Energy Dissipation Capacity

In an attempt to further enhance the system seismic performance, the links are used with energy dissipation capacity. It is noted that LA9-RA results show that drift targets are met successfully; however LA9-RB is analyzed to study the effects on RC and link

demands. This section conducts the nonlinear dynamic analyses in LA9-RB. The links were designed using the same procedure described in Section 5.3.2.4 for LA3-RB. Based on the interaction equation considering axial load and bending moment in wide-flange cross-section presented in (Bruneau et al., 2011), the links were designed such that approximately 60% of their moment capacity was available due to the high axial demands from the nonlinear static analysis. As a result, two W36X160 sections spanning 5 feet in parallel are selected as the link component at each floor level.

Figures 6.36 to 6.38 compare the median of the maximum drifts from LA9-RA and LA9-RB under critical ground motions in the three suites. It was found that, using the links with energy dissipation capacity did not further reduce the inter-story drifts, actually for all three suites of ground motions the drifts were identical in LA9-RB to LA9-RA for higher α values.

Figure 6.36: Drift comparison of LA9-RA and LA9-RB (Earthquakes corresponding to 10% in 50 years)

Figure 6.37: Drift comparison of LA9-RA and LA9-RB (Earthquakes corresponding to 2% in 50 years)

Figure 6.38: Drift comparison of LA9-RA and LA9-RB (Near-Fault Earthquakes)

Figures 6.39 to 6.44 present the normalized median of the maximum shear and bending moment demands in RC in LA9-RB under each considered earthquake suite. Compared with the RC demands on LA9-RA the shear and moment demands on LA9-RB are higher for the 2% in 50 years and near-fault ground motions and similar in the case of 10% in 50

ground motions. In all cases the first story is the one that experiences the largest shear and moment demands. Similar to LA3-RB, the addition of energy dissipation capacity increased the demands on the RC in LA9-RB.

Figure 6.39: Normalized median shear demand in RC of LA9-RB (Earthquakes corresponding to 10% in 50 years)

Figure 6.40: Normalized median bending moment demand in RC of LA9-RB (Earthquakes corresponding to 10% in 50 years)

Figure 6.41: Normalized median shear demand in RC of LA9-RB (Earthquakes corresponding to 2% in 50 years)

Figure 6.42: Normalized median bending moment demand in RC of LA9-RB (Earthquakes corresponding to 2% in 50 years)

Figure 6.43: Normalized median shear demand in RC of LA9-RB (Near-Fault Earthquakes)

Figure 6.44: Normalized median bending moment demand in RC of LA9-RB (Near-Fault Earthquakes)

The normalized median values of the maximum axial load demands, maximum shear demands and maximum bending moment demands for the links in LA9-RB are shown in Figures 6.45 to 6.53 for each considered suite of ground motions. Compared with the

axial demands shown in Figures 6.33, 6.34 and 6.35 for LA9-RA, it is consistently observed that the axial demands in LA9-RB are similar.

Figure 6.45: Normalized median axial demand in links of LA9-RB (Earthquakes corresponding to 10% in 50 years)

Figure 6.46: Normalized median shear demand in links of LA9-RB (Earthquakes corresponding to 10% in 50 years)

Figure 6.47: Normalized median bending moment demands in links of LA9-RB (Earthquakes corresponding to 10% in 50 years)

Figure 6.48: Normalized median axial demand in links of LA9-RB (Earthquakes corresponding to 2% in 50 years)

Figure 6.49: Normalized median shear demand in links of LA9-RB (Earthquakes corresponding to 2% in 50 years)

Figure 6.50: Normalized median bending moment demands in links of LA9-RB (Earthquakes corresponding to 2% in 50 years)

Figure 6.51: Normalized median axial demand in links of LA9-RB (Near-Fault Earthquakes)

Figure 6.52: Normalized median shear demand in links of LA9-RB (Near-Fault Earthquakes)

Figure 6.53: Normalized median bending moment demands in links of LA9-RB (Near-Fault Earthquakes)

Overall, the system performance was not enhanced by introducing the links with energy dissipation capacity, in fact the demands increased as a result. The drifts remained the same as to those in LA9-RA at higher RC stiffness and the drift targets of the suites of ground motions were met in all three cases.

6.3.1.5 Residual Drift

The median residual drifts from all ground motions in the same suite are discussed in this section. Figures 6.54 and 6.55 show the median residual drifts of LA9-RA and LA9-RB under the three earthquake suites. It is shown that stiffer RC tends to reduce the permanent deformation in all cases. Moreover, the median residual drifts from LA9-RB similar to those from LA9-RA, indicating energy dissipation did not further reduce the permanent deformation caused by earthquakes.

Figure 6.54: Median residual drift in LA9-RB

Figure 6.55: Median residual drift in LA9-RA

6.4 Demonstration Retrofit using Reinforced Concrete RC

A similar reinforced concrete RC to the one described in Section 5.4 was designed but this time for LA9-RA. The reason for modeling LA9-RA and not LA9-RB is because LA9-RB did not provide superior results and because LA9-RA was able to meet all

performance targets. LA9-RA with the reinforced concrete RC was subjected to the three sets of ground motion suites for analysis. The governing α value was 4.2 and the largest normalized median moment and shear demand was 210% resulting in two cross sections of 98" by 36" with 38 #10 bars on both ends of the RC as shown in Figure 6.56.

Figure 6.56: Reinforced concrete RC cross-section design for LA9-RA

The results of this analysis are compared to those obtained for LA9-RA with elastic elements on RC in Tables 6.3 to 6.5 for each of the ground motion suites. In comparing the results it is evident that the drifts are very similar and the loading demands on the RC and the links are slightly higher for the reinforced concrete RC.

Table 6-3: LA9-RA-Comparison of simplified RC and reinforced concrete RC for 10% in 50 years suite

Comparison of Drifts and Loading Demands for 10% in 50 years suite		
$\alpha = 4.2$	Elastic RC	Reinforced Concrete RC
Median Drifts from Critical Earthquakes (%)	2.4	2.6
Median Drifts from All Earthquakes (%)	2.4	2.5
Normalized Median Moment on RC (%)	152	174
Normalized Median Shear on RC (%)	152	174
Normalized Median Axial on Links (%)	196	229

Table 6-4: LA9-RA-Comparison of simplified RC and reinforced concrete RC for 2% in 50 years suite

Comparison of Drifts and Loading Demands for 2% in 50 years suite		
$\alpha = 4.2$	Elastic RC	Reinforced Concrete RC
Median Drifts from Critical Earthquakes (%)	4.2	4.2
Median Drift from All Earthquakes (%)	4.2	4.1
Normalized Median Moment on RC (%)	210	245
Normalized Median Shear on RC (%)	210	245
Normalized Median Axial on Links (%)	222	267

Table 6-5: LA9-RA-Comparison of simplified RC and reinforced concrete RC for Near-Fault suite

Comparison of Drifts and Loading Demands for Near-Fault suite		
$\alpha = 4.2$	Elastic RC	Reinforced Concrete RC
Median Drifts from Critical Earthquakes (%)	3.8	3.9
Median Drifts from All Earthquakes (%)	3.5	3.5
Normalized Median Moment on RC (%)	184	209
Normalized Median Shear on RC (%)	184	209
Normalized Median Axial on Links (%)	206	254

7. Summary of Conclusions and Recommendations for Future Research

7.1 Conclusion

This thesis analytically investigated the adequacy of a seismic retrofit strategy to mitigate drift concentration and thus soft-story failures in low-rise and mid-rise steel moment frames. The proposed retrofit strategy consists of a sufficiently stiff RC pinned to ground and connected to an existing moment frame to redistribute inter-story drifts more uniformly along the height of the building. Two steel moment frames, including one three-story building and one nine-story building exhibiting soft-story failures are retrofitted using the considered strategy. FE models for the considered buildings were developed in OpenSees and were analyzed following the nonlinear static pushover analysis and nonlinear dynamic analysis procedures.

Based on the results from numerical simulations, the following conclusions may be drawn:

From the results from Nonlinear Static Pushover Analyses:

1. It was observed in both low-rise and mid-rise moment frame buildings that the addition of RC successfully shifts the failure mode from a soft-story mechanism to a more desirable system sway mechanism.
2. For the three performance objectives considered in this research (collapse prevention, life safety, and immediate occupancy), it was observed that RC is effective in reducing drift concentration regardless of the lateral seismic load distributions.

From the results from Nonlinear Dynamic Analyses:

1. It was observed that RC helps in redistributing the inter-story drifts more uniformly along the building's height. In addition, with properly selected RCs, both buildings can satisfy the BSO requirements outlined in FEMA 356.
2. The links with energy dissipation feature help reduce the drift demands on the low-rise building but have marginal contribution on the mid-rise building partially due to the fact that the links at the upper stories of the mid-rise building are not fully yielded during the earthquakes. Further research opportunities exist to develop more effective arrangement of energy dissipation links in the system.
3. It was found that the nonlinear static analysis underestimates the loading demands on RC and the links since it does not capture strength and stiffness degradations caused by low-cycle fatigue induced fractures in beam-to-column connections.
4. Finally, it was found that the RC reduces permanent deformation in the structures after seismic loading.

In summary, the proposed retrofit strategy is proved to be a viable retrofit option that can significantly reduce soft-story failures in steel moment frame buildings. Its robust performance, cost effectiveness, and simplicity make it an ideal retrofit option.

7.2 Recommendations for Future Research

It is recommended that experimental testing should be conducted in the future to further confirm adequacy and beneficial contribution of RC in enhancing performance of steel moment frames. It is important to develop more detailed models for RC (i.e., taking into

account its width) in the future to better capture the effect of RC geometries on demands of other components in the system. It would be interesting to explore the optimum type of RC that ensures system performance and provides the ease of construction.

LIST OF REFERENCES

- Alavi, B. and Krawinkler, H. (2004). "Strengthening of Moment-resisting Frame Structures Against Near-fault Ground Motion Effects", *Earthquake Engineering and Structural Dynamics*, Vol. 33, No. 6, pp. 707-722.
- ASCE (2005). Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-05. American Society of Civil Engineers, Reston, Virginia.
- Bruneau, M., Uang, C.M., Sabelli, R., (2011) *Ductile Design of Steel Structures*, 2nd edition McGrawHill.
- FEMA 356: *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (pp. 1-35). (2000). Rehabilitation Requirements. Reston, Va.: American Society of Civil Engineers.
- Gunay, S., Korolyk, D., Mar, D., Mosalam, K., and Rodgers, J. (2009). "Infill Walls as a Spine to Enhance the Seismic Performance of Non-ductile Reinforced Concrete Frames", *ATC & SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, ASCE, Dec. 2009.
- Gupta, A., & Krawinkler, H. (1999). "Seismic Demands For Performance Evaluation of Steel Moment Resisting Frame Structures". (SAC task 5.4.3). Stanford, Calif.: *The John A. Blume Earthquake Engineering Center*.
- McKenna, F., and Fenves, G. L. (2004). OpenSees command language manual, version 1.5.2, *Pacific Earthquake Engineering Research Center*, (<http://opensees.berkeley.edu>) (May 2004).
- Open System for Earthquake Engineering Simulation (OpenSees). <http://opensees.berkeley.edu> accessed June 2013
- Qu, Z., Wada, A., Motoyui, S., Sakata, H., and Kishiki, S. (2012). "Pin-supported walls for enhancing the seismic performance of building structures." *Wiley Online Library*, DOI: 10.1002/eqe.2175.
- Terzic, V. (2011). "Force-base Element vs. Displacement-base Element". *Open System for Earthquake Engineering Simulation*, (NEEScomm).
- Terzic, V. (2013). "Modeling SCB frames using beam-column elements". *Open System for Earthquake Engineering Simulation*, (NEEScomm).

- Uriz, P., Mahin S. (2008). "Toward Earthquake-Resistant Design of Concentrically Braced Steel-Frame Structures." *Pacific Earthquake Engineering Center*, University of California, Berkeley, 2008
- Wada, A., Qu, Z., Motoyui, S., Sakata, H. (2011). "Seismic retrofit of existing SRC frame using rocking walls and steel dampers." *Frontiers of Architecture and Civil Engineering in China*.
- Williamson, C. (2012). "Seismic Performance of Symmetric Steel Moment Frames with Random Reactive Weight Distribution". California Polytechnic State University, San Luis Obispo, Thesis.

Appendix A: Response Spectra

The following are the response spectra for the three suites of ground motions used in this research.

Figure A.1: DBE Design Spectrum and response spectra for 10% in 50 years ground motions

Figure A.2: MCE Design Spectrum and response spectra for 2% in 50 years ground motions

Figure A.3: DBE Design Spectrum and response spectra for Near-Fault ground motions

Appendix B: Maximum Inter-Story Drift Results from Analysis

The following tables provide the maximum inter-story drift results for each of the three ground motion suites and for all models (LA3-RA, LA3-RB, LA9-RA, and LA9-RB).

Table B-3: Maximum Inter-Story Drift (%) for LA3-RA (10% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA3-RA (10% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.003	5.8	4.1	6.5	2.2	4.9	2.2	3.5	4.4	6.7	4.7	8.4	2.7	5.5	6.2	4.5	8.9	11.1	8.7	2.6	6.7
0.004	5.7	4.1	6.5	2.1	4.8	2.2	3.5	4.4	6.7	4.7	8.5	2.7	5.5	6.1	4.5	8.9	11.0	8.7	2.6	6.7
0.005	5.7	4.1	6.4	2.1	4.8	2.2	3.5	4.4	6.7	4.7	8.5	2.7	5.5	6.1	4.5	8.9	10.9	8.7	2.6	6.7
0.006	5.6	4.1	6.4	2.1	4.8	2.2	3.5	4.4	6.7	4.7	8.5	2.7	5.5	6.1	4.5	8.9	10.8	8.7	2.6	6.7
0.008	5.5	4.1	6.3	2.1	4.8	2.2	3.5	4.4	6.7	4.7	8.5	2.7	5.5	6.1	4.5	8.9	10.7	8.7	2.6	6.7
0.010	5.5	4.1	6.3	2.1	4.7	2.2	3.4	4.4	6.7	4.7	8.5	2.7	5.4	6.0	4.5	8.8	10.5	8.7	2.6	6.7
0.013	5.6	4.0	6.2	2.1	4.7	2.2	3.4	4.3	6.7	4.7	8.5	2.7	5.4	6.0	4.6	8.8	10.4	8.7	2.6	6.7
0.017	5.5	4.0	6.1	2.1	4.6	2.2	3.4	4.3	6.7	4.6	8.5	2.7	5.4	5.9	4.6	8.7	10.2	8.7	2.6	6.8
0.02	5.3	4.0	6.0	2.1	4.5	2.2	3.4	4.3	6.7	4.6	8.5	2.7	5.4	5.9	4.6	8.6	9.8	8.6	2.6	6.8
0.03	5.1	3.9	5.9	2.1	4.5	2.2	3.4	4.2	6.8	4.6	8.5	2.7	5.3	5.8	4.6	8.5	9.5	8.6	2.6	6.8
0.04	4.9	3.9	5.8	2.0	4.4	2.2	3.4	4.2	6.8	4.5	8.5	2.7	5.3	5.7	4.7	8.4	9.0	8.5	2.6	6.8
0.05	4.8	3.8	5.6	2.0	4.3	2.1	3.3	4.1	6.8	4.5	8.4	2.7	5.2	5.6	4.7	8.2	8.5	8.4	2.5	6.8
0.06	4.7	3.7	5.4	2.0	4.2	2.1	3.3	4.0	6.8	4.4	8.3	2.7	5.1	5.4	4.7	8.0	8.0	8.0	2.6	6.7
0.07	4.5	3.7	5.2	1.9	4.1	2.1	3.3	3.9	6.9	4.3	8.1	2.7	5.1	5.2	4.8	7.8	7.3	7.7	2.6	6.7
0.10	4.4	3.6	4.9	1.9	4.0	2.1	3.2	3.8	6.8	4.2	7.9	2.6	5.0	5.0	4.8	7.5	6.7	7.3	2.7	6.5
0.12	4.2	3.5	4.7	1.9	3.9	2.0	3.2	3.7	6.8	4.1	7.7	2.6	4.9	4.8	4.8	7.2	6.1	7.0	2.8	6.4
0.16	4.0	3.4	4.4	1.8	3.8	2.0	3.1	3.6	6.6	4.0	7.3	2.5	4.8	4.6	4.7	6.9	5.6	6.7	2.8	6.2
0.20	3.8	3.2	4.2	1.8	3.6	1.9	3.0	3.5	6.4	3.9	7.0	2.5	4.5	4.4	4.7	6.5	5.3	6.4	2.9	6.0
0.26	3.6	3.1	4.0	1.7	3.5	1.9	2.9	3.3	6.2	3.8	6.6	2.4	4.4	4.2	4.6	6.2	5.0	6.0	2.9	5.7
0.33	3.5	3.0	3.7	1.7	3.3	1.8	2.8	3.2	6.0	3.7	6.3	2.3	4.4	3.9	4.4	5.8	4.8	5.7	2.9	5.5
0.42	3.3	2.8	3.6	1.6	3.2	1.7	2.7	3.0	5.8	3.6	5.9	2.2	4.2	3.8	4.3	5.5	4.5	5.4	2.9	5.2
0.54	3.2	2.8	3.4	1.5	3.1	1.6	2.6	2.9	5.6	3.5	5.6	2.1	4.1	3.7	4.1	5.3	4.3	5.1	2.9	5.0
0.69	3.1	2.8	3.2	1.5	2.9	1.5	2.4	2.7	5.4	3.4	5.2	2.0	4.0	3.5	4.0	5.1	4.1	4.9	2.8	4.7
0.89	2.9	2.7	3.1	1.5	2.8	1.5	2.3	2.6	5.2	3.3	5.0	1.9	3.9	3.3	3.9	5.0	4.0	4.6	2.6	4.5
1.13	2.7	2.5	2.9	1.4	2.6	1.4	2.3	2.5	5.1	3.3	4.8	1.8	3.7	3.3	3.7	4.9	4.0	4.4	2.4	4.3
1.45	2.5	2.4	2.8	1.4	2.5	1.4	2.2	2.3	5.0	3.2	4.8	1.8	3.7	3.2	3.5	4.8	3.9	4.0	2.2	4.2
1.86	2.4	2.4	2.6	1.4	2.4	1.3	2.1	2.2	4.9	3.2	4.9	1.7	3.5	3.1	3.4	4.8	3.9	3.6	2.2	4.1
2.38	2.4	2.2	2.5	1.3	2.3	1.3	2.1	2.2	4.9	3.0	4.8	1.5	3.4	2.9	3.5	4.7	3.8	3.3	2.2	3.9
3.05	2.4	2.1	2.3	1.3	2.3	1.2	2.0	2.1	4.8	2.9	4.6	1.5	3.2	2.7	3.5	4.5	3.7	3.3	2.2	3.6
3.90	2.4	2.1	2.3	1.3	2.2	1.2	2.0	2.0	4.8	2.9	4.4	1.4	3.2	2.7	3.4	4.3	3.6	3.2	2.0	3.5
5.00	2.2	2.0	2.2	1.3	2.1	1.1	2.0	1.9	4.7	2.7	4.3	1.4	3.1	2.7	3.4	4.3	3.6	3.2	1.9	3.4

*Critical Earthquakes

Table B-2: Maximum Inter-Story Drift (%) for LA3-RA (2% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA3-RA (2% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06 *	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.003	9.9	14.3	6.1	1163	9.1	12.4	27.6	10.7	5.5	8.1	11.2	9.1	30.1	25.4	1162	22.9	1161	1166	6.9	13.7
0.004	9.9	14.1	6.0	1190	9.1	12.3	25.8	10.7	5.5	8.1	11.1	9.1	28.6	24.6	32.8	21.0	1196	1204	6.9	13.7
0.005	9.9	13.9	6.0	1308	9.1	12.2	24.6	10.6	5.5	8.2	11.1	9.1	27.1	23.7	29.6	20.6	1309	1306	6.9	13.6
0.006	9.9	13.7	6.0	1981	9.1	12.1	23.0	10.6	5.4	8.3	11.0	9.1	25.9	22.7	29.4	20.5	1992	1981	6.9	13.7
0.008	9.9	13.4	5.9	1750	9.0	12.0	21.2	10.6	5.4	8.3	11.0	9.1	24.7	21.5	24.2	20.3	1747	1737	6.8	14.1
0.010	10.0	13.0	5.8	1498	9.0	11.8	19.1	10.5	5.4	8.3	10.9	9.1	23.2	20.1	23.8	20.0	36.0	1579	6.8	14.6
0.013	10.0	12.6	5.8	43.2	8.9	11.7	16.8	10.4	5.5	8.4	10.7	9.1	21.4	18.3	23.3	19.7	29.2	1292	6.7	15.2
0.017	10.0	12.0	5.7	34.1	8.9	11.7	14.7	10.3	5.5	8.4	10.5	9.1	20.1	16.7	22.6	19.2	26.0	1242	6.7	15.8
0.02	10.1	11.4	5.6	28.6	8.8	11.6	13.3	10.2	5.5	8.5	10.2	9.1	18.5	15.8	21.8	18.7	22.5	1227	6.6	16.3
0.03	10.1	10.8	5.4	23.7	8.7	11.6	12.0	10.5	5.5	8.6	9.6	9.1	16.6	15.2	20.8	18.0	20.5	1285	6.5	16.6
0.04	10.1	10.1	5.3	19.9	8.5	11.5	11.2	10.8	5.5	8.6	9.0	9.0	14.9	14.5	19.8	17.3	18.9	1737	6.4	16.7
0.05	10.0	9.5	5.1	17.6	8.4	11.4	10.8	11.0	5.5	8.6	9.1	8.9	14.1	13.8	18.6	16.4	17.7	1108	6.2	16.7
0.06	9.8	8.8	4.9	15.2	8.1	11.1	10.3	11.0	5.5	8.5	9.0	8.8	13.5	13.0	17.4	15.5	16.6	933	6.0	16.6
0.07	9.6	8.2	4.7	12.7	7.9	10.8	10.3	10.7	5.4	8.4	8.9	8.6	13.0	12.3	16.2	15.1	15.6	33.2	5.9	16.2
0.10	9.5	7.6	4.6	11.0	7.6	10.3	11.2	10.1	5.3	8.0	8.6	8.3	12.8	11.6	15.2	14.8	14.6	27.6	6.1	15.7
0.12	9.3	7.2	4.4	9.6	7.3	9.8	11.5	9.3	5.1	7.6	8.2	8.0	12.7	10.9	14.3	14.5	13.5	24.9	6.2	15.2
0.16	8.9	7.0	4.3	8.7	7.0	9.3	11.8	8.7	4.9	7.2	7.9	7.8	11.6	10.2	13.3	13.9	12.5	464	6.3	14.8
0.20	8.5	6.4	4.2	7.8	6.6	8.7	12.2	8.1	4.6	6.7	7.5	7.5	11.5	9.5	12.4	13.3	11.5	1436	6.2	14.2
0.26	8.1	6.1	4.1	7.1	6.3	8.2	11.9	7.5	4.3	6.2	7.3	7.2	11.7	8.8	12.8	13.3	10.7	1666	6.0	13.5
0.33	7.8	5.8	3.9	7.0	5.9	7.7	11.1	7.0	4.0	5.7	7.3	6.9	11.6	8.1	12.9	12.2	10.0	2018	5.7	12.8
0.42	7.5	5.5	3.9	6.8	5.5	7.2	10.2	6.7	3.8	5.3	7.6	6.6	11.0	7.5	12.2	11.7	9.4	499	5.5	12.2
0.54	7.2	5.3	3.9	6.5	5.2	6.8	9.5	6.4	3.6	4.9	7.7	6.3	10.3	7.0	11.8	11.2	8.9	488	5.3	11.7
0.69	7.0	5.2	3.9	6.3	4.8	6.6	9.0	6.0	3.4	4.4	7.4	5.9	9.3	6.7	10.8	13.1	8.6	481	5.1	11.2
0.89	6.7	5.0	3.9	6.2	4.5	6.5	8.7	5.7	3.4	4.3	7.3	5.7	8.3	6.3	9.9	10.9	8.2	477	4.9	10.9
1.13	6.6	5.0	3.8	6.1	4.2	6.5	8.6	5.5	3.3	4.3	6.4	5.3	7.8	6.0	9.5	10.3	8.0	468	4.7	10.6
1.45	6.4	5.0	3.8	6.0	4.0	6.5	8.5	5.3	3.2	4.4	6.1	5.2	7.5	5.8	9.3	10.1	7.7	465	4.6	10.3
1.86	6.2	4.7	3.9	5.9	3.8	6.4	8.3	5.3	3.0	4.4	6.1	5.5	7.2	5.8	9.1	9.8	7.5	462	4.5	10.1
2.38	6.0	4.6	3.7	5.8	3.8	6.2	7.9	5.3	2.8	4.5	6.0	5.3	7.1	5.2	10.1	9.7	7.4	459	4.4	9.8
3.05	5.8	4.6	3.6	5.7	3.8	5.9	7.6	5.2	2.6	4.5	5.4	4.8	6.8	5.0	9.0	9.9	7.3	459	4.4	9.6
3.90	5.8	4.5	3.6	5.6	3.7	5.8	7.5	5.0	2.5	4.4	5.3	4.3	6.5	4.9	8.7	9.9	7.1	458	4.2	9.5
5.00	5.7	4.5	3.5	5.6	3.8	5.9	7.5	4.9	2.6	4.3	5.4	4.2	6.0	4.8	8.7	9.7	7.0	28.1	4.1	9.4

*Critical Earthquakes

Table B-3: Maximum Inter-Story Drift (%) for LA3-RA (Near-Fault ground motions)

	Maximum Inter-Story Drift (%) Results for LA3-RA (Near-Fault suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06 *	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.003	7.9	6.8	1015	8.5	17.6	3.2	18.1	6.5	13.3	5.0	8.4	2.5	8.8	4.6	14.0	5.8	10.0	4.4	1943	8.0
0.004	7.8	6.8	1063	8.5	17.6	3.2	17.6	6.5	13.2	5.0	8.4	2.5	8.8	4.6	13.9	6.0	10.0	4.4	1783	8.0
0.005	7.8	6.8	1112	8.4	17.5	3.2	17.0	6.5	13.2	5.0	8.3	2.5	8.8	4.5	13.8	6.0	9.9	4.4	1755	7.9
0.006	7.8	6.8	1143	8.3	17.3	3.2	16.2	6.4	13.1	5.0	8.3	2.5	8.7	4.5	13.7	5.9	9.8	4.4	27.3	7.9
0.008	7.7	6.7	1173	8.3	17.2	3.2	15.8	6.4	13.0	4.9	8.3	2.5	8.7	4.4	13.5	5.9	9.7	4.4	18.9	7.9
0.010	7.7	6.6	1233	8.2	17.0	3.2	15.5	6.4	12.9	4.9	8.2	2.5	8.6	4.4	13.3	5.9	9.6	4.4	16.4	7.9
0.013	7.6	6.6	32.4	8.0	16.7	3.2	15.1	6.3	12.7	4.8	8.1	2.5	8.6	4.3	13.0	5.8	9.5	4.4	16.0	7.8
0.017	7.6	6.5	25.7	7.9	16.4	3.2	14.7	6.3	12.5	4.8	8.0	2.5	8.5	4.2	12.6	6.3	9.3	4.4	15.6	7.7
0.02	7.5	6.4	20.3	7.7	16.0	3.2	14.4	6.2	12.3	4.7	7.9	2.5	8.4	4.1	12.0	6.2	9.0	4.4	15.0	7.6
0.03	7.4	6.3	17.1	7.5	15.5	3.2	14.1	6.1	12.0	4.6	7.7	2.5	8.3	3.9	11.5	6.1	8.8	4.4	14.4	7.4
0.04	7.2	6.2	14.9	7.3	14.9	3.2	13.7	6.0	11.7	4.5	7.6	2.4	8.2	3.8	10.9	6.0	8.5	4.4	13.7	7.2
0.05	7.0	6.1	13.7	7.3	14.2	3.2	13.3	5.9	11.3	4.4	7.4	2.4	8.0	3.6	10.8	5.9	8.1	4.5	13.0	7.0
0.06	6.8	5.9	12.6	7.3	13.4	3.1	12.7	5.8	10.8	4.3	7.1	2.4	7.8	3.4	10.7	5.8	7.8	4.4	12.2	6.7
0.07	6.6	5.7	11.6	7.3	12.6	3.1	12.1	5.6	10.4	4.1	6.8	2.4	7.8	3.2	10.4	5.7	7.4	4.4	11.6	6.5
0.10	6.3	5.5	10.6	7.2	11.8	3.1	11.5	5.4	9.9	4.0	6.5	2.4	7.8	3.1	10.1	5.5	7.1	4.4	11.8	6.4
0.12	6.0	5.4	10.4	6.9	10.9	3.1	10.9	5.2	9.4	3.8	6.2	2.3	7.7	3.1	9.6	5.4	6.8	4.4	11.7	6.3
0.16	5.7	5.1	10.5	6.7	10.1	3.0	10.3	5.0	8.9	3.6	5.9	2.3	7.4	3.0	9.1	5.2	6.3	4.3	11.7	6.1
0.20	5.4	5.0	10.3	6.4	9.4	3.0	9.6	4.8	8.3	3.4	5.5	2.3	7.1	2.9	8.5	5.0	6.1	4.2	11.3	5.9
0.26	5.2	4.8	10.1	6.0	8.6	2.9	9.0	4.6	7.8	3.3	5.2	2.2	6.7	2.8	8.0	4.8	5.8	4.1	11.2	5.7
0.33	4.9	4.5	9.8	5.7	7.9	2.9	8.5	4.4	7.3	3.2	5.0	2.2	6.4	2.7	7.4	4.5	5.6	4.0	10.8	5.3
0.42	4.8	4.2	9.5	5.4	7.2	2.8	7.9	4.2	6.9	3.0	4.7	2.2	6.1	2.6	7.0	4.3	5.3	3.9	10.7	4.9
0.54	4.7	3.8	9.2	5.2	6.6	2.7	7.4	4.1	6.7	2.8	4.5	2.2	5.8	2.5	6.6	4.1	5.2	3.8	9.7	4.6
0.69	4.6	3.5	9.2	5.0	6.0	2.6	6.9	3.9	6.5	2.6	4.2	2.1	5.6	2.4	6.3	3.8	5.1	3.7	8.9	4.4
0.89	4.5	3.1	9.2	4.8	5.8	2.6	6.5	3.7	6.3	2.5	4.1	2.1	5.5	2.4	6.0	3.5	5.0	3.6	8.7	4.3
1.13	4.4	2.7	9.1	4.7	6.2	2.5	6.2	3.6	6.2	2.3	3.9	2.1	5.5	2.4	5.7	3.4	4.9	3.4	8.7	4.2
1.45	4.2	2.4	8.8	4.5	6.3	2.4	6.1	3.4	6.0	2.2	3.8	2.0	5.6	2.3	5.3	3.2	4.9	3.3	8.8	4.0
1.86	4.1	2.1	8.6	4.4	6.4	2.4	6.0	3.3	5.7	2.0	3.6	1.9	5.6	2.2	5.2	2.9	5.1	3.1	8.8	4.0
2.38	4.3	1.9	8.5	4.3	6.4	2.3	6.0	3.2	5.5	1.8	3.6	1.8	5.4	2.2	5.1	2.7	5.1	3.0	8.8	3.9
3.05	4.4	1.9	8.4	4.2	6.5	2.3	5.9	3.0	5.4	1.8	3.5	1.8	5.2	2.1	5.1	2.6	4.8	2.9	8.9	3.8
3.90	4.2	1.9	8.2	4.1	6.6	2.2	5.8	3.0	5.3	1.7	3.4	1.8	5.3	2.1	5.0	2.4	4.7	2.9	8.6	3.7
5.00	4.0	1.9	8.1	4.0	6.7	2.1	5.7	2.9	5.2	1.7	3.3	1.8	5.4	2.1	5.0	2.2	4.7	2.8	8.4	3.7

*Critical Earthquakes

Table B-4: Maximum Inter-Story Drift (%) for LA3-RB (10% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA3-RB (10% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.003	5.8	4.0	6.3	2.1	4.7	2.2	3.5	4.4	6.8	4.9	8.6	2.7	5.6	6.1	4.6	9.0	10.8	8.6	2.6	6.8
0.004	5.7	4.0	6.2	2.0	4.7	2.2	3.5	4.4	6.9	4.9	8.6	2.7	5.6	6.1	4.6	9.0	10.7	8.6	2.7	6.8
0.005	5.6	4.0	6.1	2.0	4.6	2.2	3.5	4.4	6.9	5.0	8.6	2.7	5.6	6.1	4.7	9.0	10.5	8.6	2.7	6.8
0.006	5.3	3.9	6.0	2.0	4.5	2.2	3.5	4.3	7.0	5.0	8.7	2.7	5.6	6.0	4.8	9.0	10.3	8.6	2.7	6.8
0.008	5.1	3.9	5.9	2.0	4.5	2.1	3.5	4.3	7.1	5.1	8.7	2.7	5.6	6.0	4.8	9.0	10.0	8.6	2.8	6.9
0.010	5.0	3.9	5.8	1.9	4.4	2.1	3.5	4.2	7.2	5.1	8.8	2.7	5.7	5.9	4.9	9.0	9.7	8.6	2.9	6.9
0.013	4.8	3.8	5.6	1.9	4.4	2.1	3.5	4.2	7.3	5.1	8.8	2.7	5.7	5.8	5.0	9.0	9.3	8.5	3.0	7.0
0.017	4.7	3.7	5.4	1.9	4.3	2.1	3.5	4.1	7.5	5.1	8.9	2.7	5.7	5.7	5.1	9.0	8.9	8.5	3.1	7.0
0.02	4.5	3.6	5.2	1.8	4.2	2.0	3.4	4.0	7.6	5.1	8.9	2.6	5.7	5.6	5.3	8.9	8.4	8.4	3.2	7.1
0.03	4.2	3.5	5.0	1.8	4.1	2.0	3.4	3.8	7.8	5.0	8.9	2.6	5.7	5.4	5.4	8.9	7.9	8.3	3.3	7.1
0.04	4.1	3.4	4.8	1.8	4.0	1.9	3.3	3.7	8.0	4.8	8.8	2.5	5.6	5.2	5.6	8.8	7.3	8.1	3.4	7.1
0.05	3.9	3.3	4.5	1.8	3.9	1.9	3.3	3.5	8.1	4.7	8.7	2.4	5.5	5.0	5.7	8.7	6.8	7.9	3.5	7.1
0.06	3.7	3.2	4.3	1.8	3.7	1.8	3.2	3.3	8.3	4.6	8.5	2.2	5.5	4.8	5.8	8.5	6.5	7.6	3.6	7.0
0.07	3.8	3.2	4.0	1.8	3.6	1.7	3.2	3.1	8.3	4.6	8.3	2.1	5.5	4.5	5.9	8.4	6.4	7.3	3.6	6.8
0.10	3.9	3.1	3.8	1.8	3.4	1.7	3.2	3.0	8.4	4.6	8.1	2.0	5.4	4.3	5.9	8.1	6.2	7.0	3.6	6.6
0.12	4.0	3.2	3.6	1.8	3.2	1.6	3.2	3.0	8.4	4.6	7.9	1.8	5.4	4.1	5.9	7.8	6.1	6.7	3.5	6.3
0.16	4.0	3.1	3.4	1.8	3.1	1.5	3.1	2.9	8.5	4.5	7.5	1.7	5.2	4.2	5.8	7.5	5.8	6.4	3.4	6.1
0.20	3.9	3.0	3.2	1.8	3.0	1.4	3.2	2.9	8.4	4.4	7.1	1.5	5.0	4.5	5.7	7.1	5.5	6.1	3.2	5.8
0.26	3.7	2.8	3.0	1.7	2.9	1.3	3.2	2.9	8.2	4.3	6.7	1.4	4.8	4.7	5.6	6.7	5.1	7.2	3.0	5.6
0.33	3.4	2.8	2.8	1.7	2.8	1.3	3.1	2.9	7.8	4.2	6.3	1.3	4.5	4.9	5.4	6.4	4.7	6.9	2.8	6.5
0.42	3.2	2.7	2.7	1.6	2.6	1.4	3.0	2.9	7.3	4.0	6.0	1.4	4.2	4.9	5.1	6.0	4.3	6.6	2.6	6.3
0.54	3.0	2.5	2.6	1.6	2.4	1.4	2.9	2.8	6.8	3.8	5.6	1.4	4.0	4.7	4.9	5.7	3.8	6.2	2.5	6.1
0.69	2.8	2.5	2.5	1.5	2.2	1.4	2.8	2.8	6.1	3.6	5.2	1.4	3.7	4.5	4.6	5.4	3.3	5.8	2.3	5.9
0.89	2.6	2.4	2.5	1.5	2.0	1.4	2.6	2.8	5.4	3.3	4.8	1.3	3.6	4.3	4.3	5.0	3.0	5.4	2.2	5.6
1.13	2.5	2.3	2.4	1.5	1.8	1.4	2.5	2.7	4.7	3.2	4.5	1.3	3.2	3.9	4.1	4.7	2.7	5.0	2.2	5.4
1.45	2.4	2.3	2.4	1.4	1.7	1.3	2.3	2.7	4.0	3.1	4.1	1.3	3.0	3.6	3.8	4.5	2.5	4.5	2.2	5.1
1.86	2.3	2.3	2.4	1.3	1.6	1.3	2.1	2.6	3.4	3.0	3.7	1.3	2.8	3.3	3.6	4.4	2.3	4.1	2.1	4.8
2.38	2.2	2.3	2.3	1.3	1.5	1.4	2.0	2.5	2.9	2.8	3.4	1.2	2.6	3.0	3.4	4.3	2.1	3.7	2.1	4.5
3.05	2.1	2.3	2.3	1.3	1.5	1.3	1.9	2.4	2.5	2.6	3.1	1.2	2.5	2.8	3.3	4.2	2.0	3.5	1.9	3.5
3.90	2.0	2.3	2.2	1.3	1.4	1.3	1.7	2.3	2.5	2.5	2.9	1.2	2.5	2.7	3.2	4.1	1.9	3.2	1.8	3.5
5.00	1.9	2.2	2.2	1.2	1.3	1.3	1.6	2.2	2.4	2.4	2.8	1.2	2.5	2.5	3.1	4.0	1.9	3.0	1.7	3.4

*Critical Earthquakes

Table B-5: Maximum Inter-Story Drift (%) for LA3-RB (2% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA3-RB (2% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06 *	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.003	10.0	14.1	6.0	1135	9.1	12.3	24.1	10.7	5.5	8.7	11.4	9.1	26.5	22.7	31.5	20.6	1147	1137	6.8	14.1
0.004	10.0	13.8	6.0	326	9.0	12.2	22.5	10.7	5.5	8.7	11.4	9.2	25.3	21.6	27.6	20.5	1165	1141	6.8	14.6
0.005	10.1	13.6	6.0	1235	9.0	12.1	20.6	10.7	5.5	8.8	11.2	9.2	23.9	19.9	24.3	20.3	38.0	1185	6.7	15.1
0.006	10.1	13.2	6.0	37.3	8.9	12.0	18.4	10.7	5.6	8.9	11.1	9.2	22.2	18.2	23.9	20.0	30.1	1240	6.7	15.8
0.008	10.2	12.8	5.9	33.3	8.8	12.0	16.0	10.7	5.6	9.0	10.8	9.3	20.8	16.6	23.4	19.7	27.7	197	6.6	16.6
0.010	10.3	12.4	5.9	27.9	8.7	12.1	14.6	10.6	5.7	9.1	10.5	9.4	19.2	15.8	22.9	19.4	24.6	174	6.5	17.4
0.013	10.4	11.8	5.8	22.6	8.6	12.2	13.3	10.6	5.7	9.1	10.3	9.4	17.3	15.3	22.2	19.0	21.7	230	6.3	18.2
0.017	10.5	11.2	5.7	18.6	8.5	12.3	12.1	10.5	5.8	9.1	9.9	9.4	15.2	14.7	21.4	18.5	20.6	1328	6.2	19.0
0.02	10.7	10.5	5.6	16.2	8.3	12.4	11.4	10.4	5.8	9.0	9.3	9.4	14.4	14.0	20.6	18.6	19.5	1521	6.0	19.7
0.03	10.8	9.5	5.5	13.3	8.1	12.5	12.0	10.3	5.8	8.8	9.3	9.4	13.9	13.3	20.0	17.3	19.5	182	6.3	19.9
0.04	10.9	9.0	5.4	10.9	7.9	12.5	13.0	10.3	5.7	8.5	9.2	9.4	13.3	12.7	19.7	17.6	19.2	994	6.7	19.7
0.05	11.0	8.8	5.3	9.6	7.6	12.5	14.8	10.3	5.4	8.0	9.2	9.3	13.9	12.0	19.4	18.2	18.5	101	7.1	19.3
0.06	10.8	8.6	5.3	10.3	7.2	12.2	14.9	10.2	5.1	7.4	9.0	9.0	11.9	11.1	19.1	18.5	17.6	1027	7.3	20.6
0.07	10.5	8.3	5.5	10.6	6.8	11.8	12.9	10.3	4.8	7.2	8.7	8.7	11.0	10.1	18.8	18.6	16.7	1025	7.4	20.2
0.10	9.9	8.4	5.6	10.4	6.7	11.4	13.2	10.4	4.5	7.2	8.3	8.3	10.6	9.2	18.4	18.6	15.8	209	7.4	19.9
0.12	9.5	7.9	5.5	10.1	7.0	11.0	13.5	10.4	4.1	7.2	7.8	8.0	10.7	8.3	17.8	18.4	15.0	28.0	7.3	19.2
0.16	9.0	7.4	5.2	10.8	7.2	12.0	12.6	10.2	3.8	7.1	7.3	7.7	10.6	8.6	16.9	18.1	14.2	24.0	7.1	18.4
0.20	8.7	6.9	4.9	10.4	7.4	11.5	11.4	10.1	3.6	6.9	7.4	7.9	8.1	10.7	15.9	17.6	13.4	22.2	6.8	17.5
0.26	8.3	6.4	4.6	10.1	7.5	11.0	10.3	9.9	3.4	6.5	7.3	7.7	6.7	7.8	14.6	17.0	12.7	20.4	6.5	16.6
0.33	9.8	5.9	4.2	9.8	7.6	10.5	9.3	9.5	3.3	6.1	7.3	7.7	5.6	7.7	13.6	16.4	11.7	24.6	6.1	15.8
0.42	9.3	5.6	3.9	9.3	7.5	10.1	8.5	9.0	3.2	5.6	6.5	7.7	5.3	8.1	14.1	15.0	11.1	7439	5.7	14.6
0.54	8.9	5.3	3.5	8.8	7.4	9.6	7.7	8.3	3.0	5.1	6.7	7.6	5.5	8.1	12.8	14.3	10.4	4007	5.3	13.5
0.69	8.6	5.0	3.2	8.2	7.2	9.0	7.0	7.8	2.8	4.5	6.4	7.3	5.5	7.8	12.5	226	9.7	19.9	4.9	12.5
0.89	8.2	4.8	2.9	7.6	7.0	8.4	6.4	7.2	2.6	4.0	6.1	7.0	5.3	7.3	11.9	15.5	9.0	18.7	4.6	12.6
1.13	7.7	4.7	2.7	7.0	6.7	7.8	6.0	6.5	2.6	3.6	5.8	6.5	5.0	6.6	11.3	16.6	8.5	1013	4.3	12.3
1.45	6.9	4.5	2.5	6.4	6.4	7.5	5.6	6.0	2.5	3.2	6.8	5.9	4.5	5.9	11.1	24.0	8.4	997	4.0	11.9
1.86	7.3	4.3	2.3	5.9	6.1	7.4	5.2	5.6	2.4	3.0	6.4	5.6	4.1	5.4	11.0	23.8	8.3	16.5	3.7	11.4
2.38	7.2	4.2	2.1	4.1	5.2	7.2	4.9	5.3	2.2	2.8	5.7	5.2	3.7	3.7	11.3	657	8.1	15.7	3.5	10.5
3.05	6.9	4.0	2.0	3.8	5.1	7.0	4.7	5.1	2.0	2.7	4.4	4.6	3.4	3.3	11.5	1097	8.0	15.1	3.3	9.7
3.90	6.7	4.0	2.0	3.5	5.0	6.8	4.6	4.9	2.0	2.6	4.3	4.2	3.2	3.0	11.5	217	7.8	14.8	3.1	6.8
5.00	6.2	4.1	1.9	3.3	5.0	6.8	4.5	4.9	1.9	2.6	3.9	4.2	3.0	2.9	11.6	1760	7.7	14.1	3.0	6.5

*Critical Earthquakes

Table B-6: Maximum Inter-Story Drift (%) for LA3-RB (Near-Fault ground motions)

	Maximum Inter-Story Drift (%) Results for LA3-RB (Near-Fault suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10	LA11 *	LA12	LA13 *	LA14	LA15 *	LA16 *	LA17 *	LA18	LA19 *	LA20 *
0.003	7.8	7.3	1139	8.1	17.2	3.3	16.7	6.6	13.1	4.7	8.4	2.5	8.8	4.3	12.8	7.0	11.2	4.6	30.4	7.9
0.004	7.8	7.3	1155	8.0	17.1	3.3	15.9	6.5	13.0	4.7	8.4	2.5	8.8	4.2	12.5	7.0	11.1	4.6	20.1	7.8
0.005	7.8	7.3	1195	7.8	16.8	3.3	15.6	6.5	13.0	4.6	8.3	2.5	8.7	4.1	12.2	7.0	11.1	4.6	16.6	7.7
0.006	7.8	7.3	29.1	7.6	16.6	3.3	15.3	6.5	12.9	4.5	8.3	2.5	8.7	4.0	11.9	7.0	11.0	4.7	16.2	7.6
0.008	7.8	7.3	23.3	7.4	16.2	3.3	14.9	6.5	12.8	4.4	8.2	2.5	8.6	3.8	11.4	7.0	11.0	4.7	15.8	7.5
0.010	7.7	7.4	17.8	7.7	15.8	3.4	14.7	6.5	12.6	4.3	8.1	2.5	8.5	3.7	10.8	7.0	10.9	4.8	15.3	7.4
0.013	7.7	7.3	14.6	8.0	15.3	3.4	14.5	6.4	12.4	4.1	8.0	2.5	8.4	3.5	10.6	7.0	10.8	4.8	14.7	7.2
0.017	7.7	7.2	13.6	8.3	14.7	3.4	14.2	6.4	12.1	3.9	7.9	2.5	8.3	3.4	10.6	7.0	10.6	4.9	14.1	7.0
0.02	7.6	6.9	12.4	8.4	13.9	3.4	13.9	6.3	11.7	3.7	7.7	2.5	8.3	3.3	10.5	6.9	10.4	5.0	13.4	6.2
0.03	7.6	6.4	11.7	8.3	13.1	3.5	13.6	6.2	11.3	3.6	7.5	2.5	8.8	3.3	10.4	7.0	10.2	5.0	13.2	5.9
0.04	7.6	5.8	12.9	8.0	12.2	3.5	13.1	6.1	10.8	3.5	7.2	2.5	9.2	3.2	10.3	6.8	10.0	5.1	14.4	5.9
0.05	7.5	5.0	14.2	7.8	11.2	3.5	12.7	6.0	10.4	3.3	6.9	2.5	9.4	3.1	10.2	6.4	9.9	5.1	17.7	5.9
0.06	7.5	4.1	14.5	7.6	10.2	3.6	12.0	5.8	10.5	3.0	6.6	2.5	9.6	3.0	9.9	5.9	9.7	5.2	17.9	5.9
0.07	7.5	3.1	14.1	7.3	9.6	3.5	11.4	5.6	10.2	2.7	6.3	2.5	9.6	3.2	9.5	5.3	9.5	5.1	17.5	6.0
0.10	7.5	2.8	13.9	7.1	10.7	3.5	10.7	5.3	9.9	2.5	6.0	2.5	10.1	3.4	9.1	4.6	9.5	5.0	13.8	5.9
0.12	7.6	2.9	14.3	6.9	11.5	3.4	10.1	5.1	9.5	2.2	5.7	2.5	10.0	3.6	8.7	3.9	8.9	4.9	14.0	5.9
0.16	7.5	3.0	13.9	6.7	11.9	3.3	9.5	4.8	9.0	2.1	5.4	2.5	10.0	3.7	8.3	3.1	8.4	4.8	23.3	5.9
0.20	7.3	3.0	13.6	6.5	12.1	3.1	9.0	4.6	8.5	2.0	5.2	2.5	10.0	3.8	7.9	2.7	8.0	4.6	22.5	5.8
0.26	7.0	3.0	13.1	6.3	12.1	2.9	8.5	4.3	8.1	1.9	4.8	2.4	9.8	3.8	7.5	2.4	7.7	4.4	20.0	5.7
0.33	6.6	2.8	12.5	6.1	11.7	2.7	7.9	4.1	7.6	1.9	4.4	2.3	9.5	3.8	8.8	2.2	7.3	4.2	18.6	5.6
0.42	6.1	2.6	11.8	5.8	11.2	2.5	7.4	3.9	7.2	1.8	3.9	2.1	9.2	3.6	8.3	2.0	6.9	4.0	17.9	5.4
0.54	5.6	2.5	11.0	5.5	10.7	2.3	7.0	3.7	6.8	1.7	3.4	2.0	8.9	3.5	7.8	1.9	7.0	3.8	15.2	5.1
0.69	5.0	2.4	10.2	5.2	10.1	2.2	6.5	3.5	6.4	1.7	3.1	1.9	8.5	3.3	7.3	1.9	7.1	3.5	16.1	4.9
0.89	4.5	2.2	9.3	5.0	9.6	2.1	6.1	3.4	6.0	1.6	2.9	1.7	8.0	3.1	6.7	2.0	7.0	3.3	15.8	4.6
1.13	4.0	2.1	8.7	4.7	8.9	2.0	5.7	3.2	5.6	1.5	2.6	1.6	7.5	2.8	5.1	2.0	7.0	3.2	14.5	5.1
1.45	3.5	2.0	8.3	4.3	8.4	1.9	5.3	3.1	5.3	1.5	2.4	1.5	7.2	2.7	4.7	2.1	6.6	3.1	13.8	4.8
1.86	3.3	1.9	7.9	4.0	8.5	1.8	4.9	2.9	5.0	1.4	2.3	1.4	7.4	2.4	4.3	2.1	6.4	2.9	13.4	4.5
2.38	3.1	1.9	7.6	3.7	8.4	1.7	4.7	2.8	4.8	1.4	2.1	1.4	7.3	2.2	4.0	2.1	6.2	2.7	12.2	4.2
3.05	2.9	1.8	7.2	3.5	8.2	1.7	4.5	2.7	4.6	1.3	1.9	1.2	7.2	2.0	3.8	2.1	5.7	2.5	11.8	3.2
3.90	2.7	1.8	6.7	3.4	7.9	1.6	4.4	2.5	4.4	1.3	1.7	1.1	7.1	1.8	3.6	2.0	5.4	2.3	11.2	3.1
5.00	2.5	1.7	6.3	3.2	7.7	1.6	4.2	2.5	4.3	1.2	1.6	1.1	6.9	1.7	3.5	2.0	5.4	2.2	10.7	3.0

*Critical Earthquakes

Table B-7: Maximum Inter-Story Drift (%) for LA9-RA (10% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA9-RA (10% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06 *	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19	LA20 *
0.02	10.0	4.6	489	488	490	10.9	5.3	5.5	12.0	3.7	5.7	2.9	3.9	4.4	3.9	3.3	6.3	4.9	2.3	5.3
0.03	9.3	4.6	468	466	471	9.5	5.2	5.3	9.8	3.7	5.7	2.9	3.9	4.3	3.9	3.3	6.2	4.8	2.3	5.3
0.04	8.8	4.5	447	16.0	480	9.0	5.1	5.2	8.2	3.6	5.6	2.8	3.8	4.2	3.8	3.3	6.2	4.9	2.3	5.2
0.05	8.1	4.5	440	13.3	435	8.6	5.3	5.0	7.2	3.6	5.5	2.8	3.7	4.1	3.7	3.2	6.1	4.9	2.3	5.1
0.07	7.9	4.4	13.7	12.1	17.0	8.0	5.6	4.7	6.7	3.6	5.4	2.8	3.6	4.0	3.6	3.2	6.0	4.9	2.3	5.0
0.09	7.6	4.3	11.2	11.0	14.8	7.4	5.8	4.5	6.2	3.5	5.3	2.7	3.5	3.9	3.6	3.1	5.8	4.9	2.3	4.8
0.12	7.2	4.2	10.4	9.9	13.7	6.8	5.8	4.2	5.7	3.4	5.2	2.7	3.3	3.8	3.5	3.1	5.6	4.9	2.3	4.7
0.16	6.8	4.1	9.6	8.9	12.6	6.2	5.7	4.0	5.2	3.4	5.0	2.6	3.2	3.7	3.4	3.0	5.4	4.8	2.3	4.5
0.22	6.4	3.9	8.8	8.0	11.4	5.6	5.4	4.0	4.7	3.3	4.8	2.6	3.0	3.5	3.4	2.9	5.1	4.6	2.3	4.3
0.29	6.0	3.8	8.1	7.2	10.2	5.1	5.1	3.8	4.5	3.1	4.6	2.5	3.0	3.4	3.3	2.9	4.9	4.4	2.3	4.0
0.39	5.7	3.6	7.4	6.4	9.2	4.6	4.8	3.6	4.5	3.0	4.4	2.4	2.9	3.3	3.2	2.9	4.7	4.2	2.2	3.8
0.53	5.3	3.4	6.7	5.7	8.2	4.1	4.5	3.6	4.4	2.9	4.2	2.3	2.8	3.1	3.0	2.9	4.5	4.0	2.1	3.6
0.71	5.0	3.2	6.1	5.2	7.4	3.7	4.3	3.4	4.3	2.8	4.0	2.2	2.7	2.9	2.8	2.8	4.4	3.8	2.0	3.4
0.96	4.7	3.0	5.6	4.7	6.7	3.4	4.0	3.2	4.0	2.7	3.8	2.1	2.5	2.8	2.7	2.7	4.3	3.6	1.8	3.2
1.3	4.4	2.8	5.1	4.2	6.0	3.1	3.7	3.0	3.7	2.5	3.6	2.0	2.3	2.6	2.5	2.6	4.2	3.5	1.7	3.0
1.7	4.1	2.7	4.7	3.9	5.4	2.9	3.4	2.8	3.5	2.4	3.5	1.9	2.2	2.5	2.4	2.3	4.1	3.3	1.6	2.8
2.3	3.9	2.5	4.5	3.6	4.9	2.6	3.1	2.5	3.2	2.2	3.3	1.7	2.1	2.4	2.2	2.2	4.0	3.2	1.6	2.6
3.1	3.7	2.5	4.2	3.3	4.5	2.4	2.8	2.3	3.0	1.9	3.2	1.4	2.1	2.4	2.1	2.2	3.8	3.1	1.5	2.4
4.2	3.4	2.3	3.9	3.0	4.1	2.3	2.6	2.1	2.7	1.8	3.0	1.3	1.9	2.4	2.1	2.3	3.5	2.9	1.2	2.4
5.7	3.0	2.2	3.6	2.8	3.7	2.2	2.3	2.0	2.6	1.7	2.7	1.4	1.6	2.3	2.1	2.4	3.1	2.7	1.1	2.3
7.7	2.6	2.2	3.4	2.6	3.5	2.0	2.0	1.9	2.4	1.6	2.4	1.5	1.8	2.2	1.9	2.1	2.8	2.6	1.0	2.2
10.3	2.5	2.3	3.2	2.4	3.3	2.0	1.9	1.8	2.2	1.5	2.2	1.5	1.6	2.0	1.7	2.1	2.9	2.4	1.0	1.9
13.9	2.7	2.1	3.2	2.3	3.2	1.9	1.8	1.7	2.0	1.5	2.2	1.4	1.9	1.7	1.7	2.1	2.8	2.6	1.1	2.0
18.7	2.6	2.0	3.2	2.3	2.9	1.8	1.7	1.6	2.0	1.4	2.3	1.1	1.7	1.7	1.5	2.1	2.7	2.7	1.1	2.0
25.2	2.2	1.9	3.1	2.2	2.8	1.8	1.5	1.6	1.9	1.3	2.3	1.0	1.6	1.7	1.5	2.0	2.4	2.6	1.1	1.9
33.9	2.3	2.0	2.8	2.1	2.7	1.7	1.5	1.5	1.8	1.3	2.2	0.9	1.4	1.7	1.6	1.9	2.5	2.5	1.3	1.9
45.6	2.1	1.8	2.8	2.0	2.6	1.6	1.5	1.5	1.7	1.2	2.1	0.9	1.4	1.6	1.5	1.9	2.4	2.5	1.2	2.0
61.5	2.1	1.8	2.9	2.0	2.6	1.6	1.4	1.5	1.7	1.2	2.1	0.8	1.3	1.6	1.5	1.8	2.3	2.5	0.9	2.0
82.7	2.1	1.8	2.8	1.9	2.6	1.6	1.4	1.4	1.7	1.2	2.1	0.9	1.3	1.5	1.4	1.8	2.3	2.3	0.7	2.0
111	2.0	1.8	2.8	2.0	2.6	1.6	1.4	1.4	1.7	1.2	2.0	0.8	1.3	1.5	1.3	1.8	2.3	2.3	0.8	1.8
150	2.0	1.8	2.8	1.9	2.5	1.6	1.4	1.4	1.7	1.2	2.0	0.8	1.3	1.5	1.3	1.8	2.3	2.3	0.7	1.7

*Critical Earthquakes

Table B-8: Maximum Inter-Story Drift (%) for LA9-RA (2% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA9-RA (2% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06 *	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.02	5.4	7.5	9.1	512	523	6.7	7.1	527	5.4	478	7.6	5.8	540	87.9	510	521	507	484	5.6	527
0.03	5.5	7.4	8.8	527	501	6.6	6.9	502	5.3	454	7.3	5.8	488	498	499	508	268	464	5.5	493
0.04	5.6	7.3	8.4	502	17.8	6.5	6.8	12.3	5.3	470	7.1	5.8	518	505	510	524	530	14.8	5.4	506
0.05	5.7	7.3	7.9	501	12.6	6.4	6.6	9.6	5.2	441	6.8	5.7	502	17.9	497	494	496	13.9	5.3	495
0.07	5.8	7.2	7.7	465	9.8	6.2	6.9	9.1	5.1	414	6.4	5.6	19.5	15.0	462	138	459	13.7	5.2	18.6
0.09	5.8	7.1	7.4	458	8.8	5.9	7.1	8.6	5.0	390	6.1	5.5	14.7	13.7	455	441	443	14.6	5.0	15.4
0.12	5.7	6.9	7.0	418	8.0	5.7	7.1	8.0	4.9	429	5.6	5.3	13.4	12.5	419	414	420	14.9	4.8	14.1
0.16	5.6	6.7	6.6	167	7.4	5.4	7.0	7.5	4.7	408	5.2	5.1	12.1	11.3	427	425	424	14.9	4.5	12.8
0.22	5.4	6.4	6.3	18.6	6.7	5.2	6.8	6.9	4.6	425	4.8	4.7	10.8	10.1	453	450	19.4	14.8	4.3	11.6
0.29	5.2	6.2	5.9	15.4	6.1	4.9	6.6	6.4	4.5	16.4	4.6	4.4	9.5	9.0	27.9	21.9	16.3	14.3	4.1	10.5
0.39	4.9	5.9	5.5	13.4	5.5	4.7	6.3	5.9	4.5	13.6	4.6	4.0	8.5	8.1	19.4	18.2	14.6	13.4	3.8	9.3
0.53	4.6	5.6	5.2	12.2	5.1	4.4	6.0	5.4	4.4	11.1	4.6	3.7	7.7	7.3	17.4	16.5	13.0	12.5	3.6	8.4
0.71	4.3	5.4	4.8	11.1	4.8	4.2	5.7	5.0	4.3	10.0	4.4	3.4	7.0	6.7	15.7	14.9	11.7	11.8	3.4	7.6
0.96	4.1	5.1	4.5	10.3	4.4	3.9	5.4	4.7	4.2	8.3	4.1	3.2	6.3	6.1	14.1	13.4	10.7	11.1	3.1	6.9
1.3	4.0	4.9	4.2	9.5	4.2	3.7	5.1	4.5	4.1	7.4	4.0	2.9	5.7	5.6	12.7	12.1	9.8	10.3	2.9	6.4
1.7	3.9	4.6	3.9	8.6	3.9	3.5	4.9	4.2	3.9	6.8	3.8	2.7	5.2	5.1	11.3	10.9	8.9	9.6	2.7	5.9
2.3	3.7	4.4	3.6	7.8	3.6	3.2	4.7	3.9	3.5	6.2	3.5	2.6	4.7	4.7	10.0	9.9	8.0	8.8	2.6	5.4
3.1	3.5	4.1	3.4	7.9	3.4	3.0	4.6	3.7	3.2	5.6	3.3	2.5	4.4	4.4	8.8	8.9	7.3	8.2	2.5	5.0
4.2	3.5	4.2	3.2	7.3	3.2	2.8	4.3	3.4	2.9	5.2	3.2	2.4	4.2	4.2	7.7	8.0	6.9	7.8	2.5	4.6
5.7	3.3	4.2	2.9	7.3	3.3	3.0	4.0	3.3	2.5	4.9	2.7	2.7	3.9	3.9	7.0	7.2	6.3	7.5	2.4	4.3
7.7	2.9	3.9	2.7	7.4	3.0	3.0	3.7	3.4	2.5	4.8	2.7	3.2	3.7	3.6	6.6	6.6	5.7	7.1	2.3	4.0
10.3	3.3	3.3	2.6	7.3	2.7	2.7	3.2	3.3	2.3	4.5	3.1	3.5	3.7	3.4	6.6	6.5	5.0	6.6	2.2	3.7
13.9	3.2	2.5	2.4	6.7	2.8	2.8	3.3	3.2	2.0	4.0	2.8	2.9	3.8	3.7	6.5	6.5	5.2	6.5	2.0	3.6
18.7	2.7	2.6	2.4	6.5	2.6	2.9	3.3	3.1	2.0	4.0	2.2	2.7	3.7	3.7	6.2	6.1	5.3	6.4	1.9	3.7
25.2	2.9	2.7	2.3	6.3	2.6	2.8	3.0	3.1	2.0	4.2	2.0	2.7	3.7	3.7	6.0	5.7	5.1	6.2	1.8	3.6
33.9	3.1	2.6	2.2	6.3	2.7	2.8	3.0	3.1	1.9	3.8	1.9	2.2	3.7	3.6	5.9	5.4	4.8	6.0	1.8	3.5
45.6	3.1	2.6	2.1	6.2	2.6	2.5	3.0	3.0	1.9	3.6	1.9	2.5	3.7	3.7	5.9	5.5	4.6	6.0	1.8	3.5
61.5	3.2	2.5	2.0	6.1	2.5	2.5	2.9	3.0	1.8	3.6	1.9	2.3	3.6	3.6	5.6	5.5	4.6	6.0	1.7	3.5
82.7	3.2	2.4	2.0	6.2	2.4	2.6	2.8	3.0	1.8	3.6	1.9	2.4	3.6	3.5	5.6	5.4	4.6	5.9	1.6	3.5
111	3.1	2.4	1.9	6.2	2.3	2.5	2.8	2.8	1.7	3.5	1.7	2.3	3.5	3.4	5.6	5.3	4.5	5.9	1.6	3.5
150	3.1	2.4	1.9	6.2	2.2	2.5	2.8	2.9	1.7	3.7	1.7	2.0	3.5	3.5	5.6	5.3	4.5	5.9	1.6	3.4

*Critical Earthquakes

Table B-9: Maximum Inter-Story Drift (%) for LA9-RA (Near-Fault ground motions)

	Maximum Inter-Story Drift (%) Results for LA9-RA (Near-Fault suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04 *	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10	LA11 *	LA12	LA13 *	LA14	LA15 *	LA16	LA17 *	LA18	LA19 *	LA20 *
0.02	485.2	485	522	532	523	3.7	494	9.6	6.7	3.4	514	2.0	8.3	4.9	6.8	1.8	8.4	4.3	510	9.3
0.03	470.8	463	264	500	493	3.7	466	8.2	6.6	3.4	501	2.0	8.2	4.8	6.7	1.8	8.2	4.2	468	8.7
0.04	441.3	9.5	527	530	512	3.6	449	6.9	6.5	3.4	509	2.0	8.0	4.7	6.5	1.8	7.9	4.1	447	7.9
0.05	437.6	10.0	499	18.1	16.8	3.6	431	6.1	6.3	3.3	496	2.0	7.8	4.7	6.3	1.8	7.8	4.0	13.8	7.6
0.07	16.5	10.5	461	12.1	14.7	3.5	15.6	5.4	6.1	3.3	463	2.0	7.5	4.8	6.1	1.8	7.6	3.9	10.9	7.5
0.09	13.6	10.1	443	10.5	13.7	3.5	12.6	5.2	6.2	3.2	441	2.0	7.2	4.9	6.1	1.8	7.4	3.7	9.9	7.3
0.12	12.4	9.2	415	9.8	12.7	3.4	11.6	4.9	6.3	3.1	418	1.9	6.8	5.0	6.3	1.7	7.2	3.5	9.4	7.1
0.16	11.3	8.4	427	9.0	11.7	3.3	10.7	4.6	6.2	3.1	18.5	1.9	6.5	5.0	6.3	1.7	6.9	3.4	8.9	6.8
0.22	10.1	7.5	451	8.2	10.8	3.2	9.8	4.3	6.1	3.1	15.3	1.8	6.1	5.0	6.3	1.7	6.6	3.4	8.2	6.4
0.29	9.1	6.7	28.2	7.6	9.9	3.1	8.9	4.1	5.9	3.0	12.7	1.8	5.8	4.9	6.1	1.6	6.3	3.4	7.6	6.1
0.39	8.3	6.0	19.6	6.9	9.1	3.0	8.2	3.8	5.7	2.9	10.8	1.7	5.5	4.7	5.9	1.6	6.0	3.3	7.0	5.7
0.53	7.5	5.3	15.7	6.4	8.4	2.8	7.5	3.6	5.5	2.9	9.1	1.5	5.2	4.6	5.7	1.6	5.7	3.2	6.4	5.3
0.71	6.8	4.9	13.1	5.9	7.8	2.7	6.9	3.3	5.2	2.8	7.9	1.4	4.9	4.4	5.4	1.6	5.5	3.0	5.9	4.9
0.96	6.0	4.5	11.8	5.4	7.2	2.6	6.3	3.3	4.9	2.6	7.0	1.3	4.6	4.2	5.1	1.6	5.2	2.7	5.4	4.6
1.3	5.3	4.2	10.7	5.1	6.7	2.4	5.7	3.2	4.7	2.5	6.1	1.2	4.3	4.1	4.8	1.6	5.0	2.5	5.0	4.3
1.7	4.7	3.8	9.8	4.8	6.2	2.3	5.2	3.0	4.5	2.4	5.4	1.1	4.0	3.8	4.6	1.6	4.8	2.3	4.7	3.9
2.3	4.2	3.4	8.9	4.5	5.8	2.1	4.9	2.7	4.3	2.4	4.8	1.1	3.8	3.5	4.3	1.5	4.6	2.1	4.4	3.5
3.1	4.0	3.2	8.3	4.4	5.4	2.0	4.7	2.4	4.1	2.3	4.3	1.1	3.5	3.3	4.0	1.4	4.5	1.9	4.1	3.3
4.2	3.9	3.0	8.0	4.3	5.0	1.9	4.6	2.4	3.8	2.4	3.8	1.1	3.3	3.1	3.7	1.3	4.7	1.8	3.7	3.1
5.7	3.9	2.9	7.7	4.2	4.6	1.8	4.4	2.4	3.6	2.3	3.6	1.0	3.0	3.0	3.5	1.1	4.6	1.7	3.3	2.9
7.7	3.5	2.4	7.8	4.2	4.4	1.7	4.3	2.3	3.3	2.2	3.3	0.8	2.8	2.7	3.3	1.1	4.1	1.8	3.2	2.7
10.3	3.2	2.2	7.8	4.1	4.4	1.6	4.1	2.1	3.0	2.0	3.1	0.8	2.9	2.5	3.2	1.2	3.3	1.7	3.2	2.5
13.9	2.9	2.1	7.2	3.5	4.1	1.4	3.9	2.0	2.7	1.8	2.7	0.8	2.8	2.3	3.1	1.0	3.3	1.6	3.2	2.5
18.7	2.9	2.0	6.6	3.4	3.9	1.4	3.6	1.9	2.7	1.9	2.6	0.6	2.7	2.4	2.9	1.4	3.0	1.5	2.9	2.5
25.2	3.0	1.9	6.8	3.5	3.8	1.3	3.4	1.8	2.7	1.9	2.4	0.6	2.6	2.3	2.7	1.0	3.1	1.4	3.0	2.3
33.9	2.9	1.8	6.6	3.4	3.7	1.3	3.2	1.7	2.7	1.8	2.4	0.6	2.6	2.2	2.8	1.1	3.5	1.2	2.9	2.2
45.6	2.7	1.8	6.2	3.4	3.6	1.3	3.0	1.7	2.7	1.8	2.3	0.7	2.5	2.1	2.7	0.9	3.4	1.1	2.9	2.3
61.5	2.7	1.8	6.2	3.5	3.5	1.3	2.9	1.7	2.6	1.8	2.3	0.6	2.4	2.1	2.7	1.0	3.3	1.1	3.0	2.3
82.7	2.6	1.7	6.1	3.4	3.5	1.3	2.8	1.6	2.7	1.7	2.3	0.7	2.3	2.1	2.8	0.9	3.3	1.0	2.9	2.2
111	2.7	1.8	6.1	3.4	3.5	1.3	2.8	1.6	2.7	1.7	2.3	0.7	2.3	2.0	2.7	0.9	3.3	1.0	2.9	2.2
150	2.6	1.7	6.0	3.4	3.5	1.3	2.8	1.6	2.7	1.7	2.2	0.7	2.2	2.0	2.7	0.9	3.3	1.0	2.9	2.2

*Critical Earthquakes

Table B-10: Maximum Inter-Story Drift (%) for LA9-RB (10% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA9-RB (10% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.02	10.4	5.4	649	16.0	124	9.0	6.3	4.5	8.2	4.1	6.1	2.9	3.4	4.4	4.2	3.3	6.5	6.0	2.5	5.3
0.03	9.9	5.4	236	13.3	238	8.5	6.7	4.1	6.9	4.1	6.1	2.9	3.3	4.3	4.1	3.3	6.5	6.0	2.6	5.3
0.04	9.4	5.4	192	12.0	193	7.9	6.8	4.0	5.9	4.0	6.1	2.8	3.4	4.2	4.0	3.4	6.5	5.8	2.6	5.2
0.05	10.0	5.4	15.0	11.1	19.4	7.4	6.8	3.9	5.5	3.9	6.1	2.7	3.4	4.1	3.9	3.5	6.4	5.8	2.5	5.0
0.07	9.3	5.3	11.4	10.2	15.3	6.7	6.6	4.1	5.3	3.7	6.0	2.7	3.4	4.0	3.7	3.6	6.3	5.8	2.3	4.9
0.09	8.6	5.1	10.6	9.3	14.0	6.1	6.1	4.3	5.3	3.6	5.9	2.6	3.2	3.8	3.5	3.6	6.1	5.7	2.2	4.7
0.12	8.2	4.8	9.7	8.4	12.7	5.5	5.7	4.2	5.3	3.4	5.7	2.5	3.1	3.7	3.4	3.6	5.9	5.6	2.0	4.5
0.16	7.8	4.5	8.8	7.5	11.4	4.9	5.2	4.0	5.2	3.2	5.6	2.4	2.9	3.6	3.3	3.6	5.6	5.5	1.9	4.4
0.22	7.3	4.1	7.9	6.6	10.2	4.4	4.8	3.7	5.0	2.9	5.5	2.2	2.8	3.4	3.2	3.5	5.7	5.3	1.9	4.2
0.29	6.8	3.7	7.7	5.8	9.1	3.9	4.3	3.4	4.7	2.7	5.3	2.1	2.7	3.3	3.1	3.3	5.8	5.9	1.8	4.0
0.39	6.3	3.4	8.8	5.5	8.1	3.6	3.8	3.1	4.3	2.4	5.0	2.0	2.6	3.1	3.0	3.2	5.8	5.6	1.8	3.8
0.53	5.7	3.2	10.8	4.6	7.0	3.3	3.2	2.8	3.9	2.2	4.8	1.8	2.5	2.9	2.8	3.1	5.7	5.3	1.7	3.8
0.71	5.3	3.1	6.6	4.1	8.8	3.1	2.9	2.6	3.5	2.0	4.5	1.7	2.4	2.9	2.7	3.1	5.5	5.0	1.6	3.8
0.96	4.8	3.0	6.2	3.8	10.1	2.8	2.7	2.5	3.2	1.9	4.3	1.6	2.4	2.8	2.5	3.0	5.2	4.8	1.4	3.8
1.3	4.4	2.8	5.7	3.6	5.7	2.6	2.4	2.3	2.9	1.8	4.1	1.5	2.4	2.7	2.4	2.9	4.9	4.5	1.3	3.7
1.7	4.0	2.6	5.2	3.4	5.3	2.4	2.3	2.2	2.6	1.6	4.0	1.4	2.4	2.7	2.2	2.7	4.5	4.2	1.2	3.6
2.3	3.7	2.4	4.7	3.2	4.9	2.2	2.2	2.0	2.5	1.5	3.7	1.2	2.2	2.6	2.1	2.5	4.1	3.5	1.1	3.5
3.1	3.3	2.3	4.3	2.9	4.5	2.1	2.0	1.9	2.4	1.4	3.5	1.2	2.0	2.5	2.0	2.5	3.7	3.3	1.1	3.0
4.2	2.9	2.1	3.9	2.7	4.2	1.9	1.7	1.7	2.3	1.3	3.2	1.3	1.8	2.4	1.9	2.5	3.4	3.0	1.0	2.9
5.7	2.5	2.0	3.6	2.5	3.8	1.8	1.5	1.6	2.2	1.2	2.7	1.4	1.8	2.2	1.8	2.5	3.0	2.6	1.0	2.7
7.7	2.3	2.1	3.5	2.3	3.4	1.8	1.4	1.5	2.1	1.2	2.5	1.3	1.8	1.9	1.7	2.3	2.9	2.5	1.0	2.4
10.3	2.2	1.9	3.4	2.1	2.9	1.7	1.4	1.4	1.9	1.1	2.3	1.3	1.8	1.7	1.6	2.3	2.8	2.6	1.1	2.1
13.9	2.3	2.1	3.1	2.0	2.8	1.7	1.4	1.3	1.9	1.1	2.3	1.2	1.8	1.6	1.5	2.2	2.5	2.8	1.2	1.9
18.7	2.1	2.2	2.8	2.0	2.7	1.6	1.4	1.3	1.9	1.0	2.3	0.9	1.8	1.7	1.5	2.1	2.5	2.9	1.1	2.0
25.2	2.1	2.1	2.7	1.9	2.6	1.6	1.2	1.2	1.7	1.0	2.3	1.0	1.7	1.7	1.5	2.0	2.4	2.7	1.1	2.1
33.9	1.9	1.9	2.6	1.8	2.6	1.5	1.2	1.2	1.6	1.0	2.2	0.8	1.4	1.7	1.5	2.0	2.4	2.5	1.3	2.1
45.6	1.9	1.9	2.7	1.8	2.5	1.5	1.2	1.1	1.7	0.9	2.2	0.7	1.5	1.6	1.5	2.0	2.4	2.5	1.1	2.2
61.5	1.9	1.8	2.6	1.7	2.5	1.4	1.2	1.1	1.7	0.9	2.2	0.7	1.4	1.6	1.4	2.0	2.4	2.6	0.9	2.2
82.7	1.8	1.9	2.5	1.6	2.5	1.4	1.2	1.1	1.7	0.9	2.2	0.7	1.4	1.6	1.4	2.0	2.4	2.4	0.7	2.2
111	1.8	1.9	2.5	1.6	2.4	1.4	1.1	1.1	1.6	0.9	2.1	0.7	1.3	1.5	1.3	2.0	2.3	2.3	0.7	2.0
150	1.7	1.8	2.5	1.6	2.4	1.4	1.1	1.1	1.7	0.9	2.0	0.7	1.3	1.5	1.3	2.0	2.3	2.3	0.7	1.9

*Critical Earthquakes

Table B-11: Maximum Inter-Story Drift (%) for LA9-RB (2% in 50 years suite)

	Maximum Inter-Story Drift (%) Results for LA9-RB (2% in 50 years suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.02	7.1	7.5	9.8	511	526	6.7	6.9	12.2	6.2	760	6.7	6.0	551	550	535	550	520	14.5	5.6	534
0.03	7.3	7.5	9.6	548	544	6.6	6.8	9.9	5.8	129	6.3	5.9	539	495	512	552	537	510	5.5	527
0.04	7.4	7.6	9.4	482	470	6.5	7.3	9.5	5.7	607	6.2	5.6	490	482	478	488	502	461	5.3	469
0.05	7.4	7.6	9.1	450	16.5	6.3	7.7	9.1	5.5	144	6.3	5.3	428	467	462	468	464	465	5.2	467
0.07	7.2	7.5	8.7	450	12.7	6.1	8.0	9.1	5.3	129	6.4	5.0	468	14.7	395	457	447	449	5.0	83.0
0.09	7.1	7.6	8.0	448	10.7	5.9	9.0	8.8	4.9	291	6.2	4.6	387	13.6	444	450	465	463	4.7	462
0.12	7.1	7.6	7.2	470	9.3	5.8	9.2	8.4	5.2	171	5.9	4.3	13.3	12.5	456	464	438	456	4.5	15.3
0.16	7.7	7.5	6.7	482	7.9	5.6	9.2	7.9	5.4	198	5.6	4.1	12.0	11.4	471	463	462	92.0	4.3	14.1
0.22	7.5	7.3	6.2	463	7.1	5.5	9.0	7.4	5.5	23.5	5.3	3.9	10.8	10.3	459	463	457	451	4.1	13.0
0.29	7.3	7.0	5.8	469	6.4	5.3	8.7	6.8	5.3	14.3	5.1	3.7	9.8	9.3	474	461	479	472	4.1	12.2
0.39	7.1	6.8	5.1	462	5.8	5.1	8.4	6.5	4.7	11.9	4.8	3.4	8.8	8.4	448	466	460	441	4.1	11.4
0.53	6.8	6.4	4.6	472	5.5	4.9	8.0	6.3	4.2	11.1	4.4	3.2	7.7	7.5	476	462	449	80.7	4.1	11.0
0.71	6.4	6.0	4.2	431	5.0	4.6	7.5	6.1	4.8	9.1	4.0	3.0	7.0	6.5	475	460	457	78.5	4.0	9.1
0.96	5.9	5.6	3.8	433	4.6	4.3	6.9	5.9	4.4	8.0	3.7	2.9	7.0	6.8	456	439	438	426	3.9	7.7
1.3	5.4	5.2	3.4	444	4.3	4.0	6.5	5.5	4.2	7.0	3.2	2.8	6.8	6.8	78.5	420	432	452	3.7	7.0
1.7	4.8	4.8	3.1	9.5	4.0	3.7	7.8	5.0	3.9	6.2	2.8	2.7	6.4	6.7	410	399	408	435	3.6	6.8
2.3	3.5	4.4	2.9	8.6	3.7	3.6	5.6	4.6	2.9	5.4	2.7	2.7	6.2	6.4	391	396	402	406	3.1	6.6
3.1	3.4	3.9	2.7	7.7	3.4	3.5	5.0	4.2	2.8	5.0	2.5	2.9	6.0	6.0	397	15.0	10.4	410	2.9	6.2
4.2	3.0	3.5	2.6	7.5	3.3	3.6	4.5	3.9	2.7	4.6	2.4	3.1	5.5	5.6	17.4	14.0	9.2	398	2.8	5.8
5.7	3.0	3.4	2.5	19.8	3.1	3.6	4.0	3.7	2.5	4.4	2.5	2.9	5.2	5.1	17.4	12.5	8.4	14.9	2.6	5.5
7.7	3.6	3.0	2.4	21.8	2.9	3.2	3.7	3.5	2.3	4.1	2.8	3.2	4.7	4.6	14.4	12.2	8.8	9.8	2.5	5.1
10.3	3.5	2.6	2.2	21.1	2.7	2.9	3.8	3.3	2.1	3.9	2.7	3.0	4.4	4.4	10.2	11.9	7.9	8.4	2.4	4.9
13.9	3.2	2.4	2.1	13.4	2.5	2.9	3.6	3.5	1.8	3.6	2.2	2.7	4.2	4.1	10.0	10.7	5.0	6.9	2.2	4.7
18.7	2.6	2.5	2.0	13.6	2.5	2.9	3.5	3.6	1.7	3.5	1.9	2.7	4.0	4.1	7.3	9.2	5.0	7.4	1.9	4.7
25.2	3.1	2.6	1.7	8.6	2.4	2.8	3.0	3.4	1.7	3.6	1.7	2.6	4.0	4.0	6.4	8.8	6.3	7.3	1.9	4.6
33.9	3.2	2.5	1.7	5.2	2.5	2.6	3.1	3.3	1.6	3.4	1.8	2.2	4.1	4.0	7.0	6.2	5.2	6.1	2.0	4.5
45.6	3.1	2.5	1.6	5.2	2.5	2.5	3.0	3.2	1.5	3.2	1.8	2.4	4.1	4.0	6.3	6.2	4.2	6.1	2.0	4.2
61.5	3.3	2.4	1.6	5.2	2.4	2.4	2.9	3.3	1.5	3.0	1.8	2.3	4.0	3.9	4.6	4.6	4.1	6.0	1.8	4.1
82.7	3.1	2.2	1.6	5.1	2.3	2.5	2.9	3.1	1.5	3.0	1.8	2.4	4.0	3.8	4.6	4.6	4.1	6.0	1.7	4.1
111	3.1	2.2	1.6	5.1	2.2	2.4	2.9	2.9	1.6	2.9	1.6	2.3	3.8	3.8	4.7	4.6	4.1	6.0	1.6	4.1
150	3.1	2.2	1.6	5.1	2.1	2.4	2.9	3.0	1.5	3.0	1.6	2.1	3.9	3.8	4.7	4.5	4.0	5.9	1.6	4.1

*Critical Earthquakes

Table B-12: Maximum Inter-Story Drift (%) for LA9-RB (Near-Fault ground motions)

	Maximum Inter-Story Drift (%) Results for LA9-RB (Near-Fault suite)																			
	Ground Motion																			
α	LA01 *	LA02 *	LA03 *	LA04	LA05 *	LA06	LA07 *	LA08 *	LA09 *	LA10 *	LA11 *	LA12 *	LA13 *	LA14 *	LA15 *	LA16 *	LA17 *	LA18 *	LA19 *	LA20 *
0.02	129.2	737	530	549	546	3.8	803	7.9	6.7	3.6	148	2.0	8.4	5.1	6.7	1.8	8.7	4.0	13.6	8.5
0.03	211.1	17.6	534	562	544	3.7	214	6.7	6.5	3.6	543	1.9	8.2	5.0	6.5	1.8	8.5	3.7	11.0	8.4
0.04	190.4	9.8	483	463	465	3.7	209	6.1	6.4	3.6	466	1.9	8.0	5.2	6.3	1.9	8.3	3.7	9.7	8.3
0.05	77.4	8.2	464	451	19.9	3.6	21.6	5.6	6.7	3.7	449	1.8	7.8	5.4	6.6	1.9	8.1	3.8	9.4	8.1
0.07	14.3	7.6	444	447	453	3.5	16.2	5.3	7.0	3.7	453	1.7	7.6	5.6	7.0	1.9	8.5	3.8	9.1	7.8
0.09	13.0	7.2	456	437	14.9	3.4	13.6	5.1	7.4	3.7	465	1.6	7.4	5.7	7.3	2.0	8.1	3.6	9.0	7.5
0.12	11.8	6.9	441	11.7	14.1	3.3	12.7	4.8	7.8	3.6	452	1.5	7.2	5.8	7.6	2.0	7.7	3.3	8.7	7.0
0.16	10.7	7.5	461	10.6	13.2	3.2	11.6	4.5	8.0	3.6	462	1.4	6.9	5.8	8.0	2.1	7.5	2.9	8.4	6.6
0.22	9.7	6.9	459	9.8	12.4	3.0	10.5	4.6	7.9	3.5	11.7	1.4	6.6	5.8	8.0	2.1	7.3	2.7	9.0	6.2
0.29	8.7	6.5	469	9.1	11.5	2.9	9.5	4.3	7.8	3.4	10.2	1.4	6.2	5.6	7.9	2.1	7.1	2.6	7.6	5.5
0.39	7.6	6.4	477	8.4	11.0	2.8	8.5	4.2	7.5	3.3	9.0	1.4	5.9	5.3	7.7	2.1	6.8	2.5	7.1	5.0
0.53	6.3	6.0	477	7.6	10.5	2.6	7.6	4.3	7.1	3.2	7.6	1.3	5.5	4.9	7.4	2.0	6.4	2.4	6.6	4.9
0.71	5.6	5.4	442	7.0	9.9	2.5	6.8	4.2	6.7	3.1	6.7	1.3	5.2	4.5	7.0	2.0	6.0	2.4	6.1	4.8
0.96	5.0	4.6	446	6.4	9.0	2.4	6.0	3.8	6.2	3.0	5.9	1.3	4.8	4.0	6.5	1.9	5.7	2.3	5.6	4.5
1.3	4.4	3.8	437	6.2	7.4	2.3	5.4	3.4	5.8	2.9	5.1	1.2	4.5	3.7	6.0	1.7	5.2	2.2	5.1	4.3
1.7	4.0	3.4	428	5.9	6.8	2.1	5.0	3.2	5.4	2.8	4.5	1.2	4.1	3.4	5.5	1.6	4.8	2.1	4.6	4.0
2.3	3.7	3.2	21.7	5.7	6.3	2.0	4.7	3.1	5.1	2.8	4.0	1.1	3.8	3.2	5.0	1.4	4.3	2.0	4.2	3.8
3.1	3.5	2.8	400	5.3	5.8	1.9	4.5	2.5	4.8	2.7	3.6	1.0	3.6	3.2	4.0	1.3	3.9	1.9	4.0	3.5
4.2	3.4	2.4	12.9	5.1	5.3	1.8	4.3	2.4	4.5	2.6	3.3	0.9	3.3	3.1	3.8	1.1	3.9	1.8	3.8	3.2
5.7	3.2	2.2	15.1	5.6	4.9	1.7	4.1	2.2	4.1	2.5	3.1	0.9	3.1	2.8	3.6	1.3	3.7	1.8	3.5	2.8
7.7	3.0	2.0	381	5.7	4.4	1.5	3.9	2.0	3.8	2.4	2.8	0.9	2.9	2.6	3.4	1.3	3.4	1.8	3.3	2.7
10.3	2.9	2.1	16.4	4.0	4.1	1.4	3.7	1.9	3.4	2.1	2.5	0.9	2.6	2.4	3.3	1.1	3.6	1.6	3.1	2.5
13.9	2.7	1.9	20.3	3.6	3.8	1.3	3.5	1.9	3.2	1.8	2.3	0.8	2.5	2.4	3.3	1.3	3.5	1.6	3.0	2.4
18.7	2.6	1.8	13.5	3.0	3.7	1.3	3.3	1.8	2.9	1.9	2.1	0.7	2.5	2.2	3.2	1.4	3.0	1.5	3.0	2.3
25.2	2.6	1.9	14.1	2.9	3.6	1.3	3.1	1.7	2.9	1.9	2.0	0.7	2.5	2.3	2.9	1.1	3.4	1.3	3.0	2.2
33.9	2.6	1.7	7.5	2.8	3.5	1.2	3.0	1.5	2.8	1.8	2.0	0.7	2.4	2.3	2.8	1.2	3.5	1.2	2.9	2.1
45.6	2.4	1.7	4.6	2.8	3.5	1.2	2.8	1.5	2.9	1.8	1.9	0.7	2.4	2.3	2.9	1.0	3.3	1.2	2.9	2.1
61.5	2.4	1.8	4.6	2.8	3.4	1.2	2.7	1.5	2.9	1.9	1.9	0.7	2.3	2.2	2.9	1.1	3.3	1.2	2.9	2.1
82.7	2.3	1.8	4.6	2.7	3.4	1.2	2.7	1.5	2.8	1.8	1.9	0.7	2.2	2.1	2.8	1.1	3.2	1.1	2.9	2.0
111	2.3	1.7	4.6	2.7	3.4	1.2	2.7	1.5	2.8	1.8	1.9	0.7	2.2	2.1	2.7	1.0	3.2	1.1	2.9	2.0
150	2.4	1.7	4.6	2.7	3.4	1.2	2.7	1.5	2.8	1.8	1.9	0.7	2.1	2.1	2.8	1.1	3.2	1.0	2.9	2.0

*Critical Earthquakes

Figure B.1: Number of earthquakes under each performance objective for LA3-RA (10% in 50 years)

Figure B.2: Number of earthquakes under each performance objective for LA3-RA (2% in 50 years)

Figure B.3: Number of earthquakes under each performance objective for LA3-RA (Near-Fault Earthquakes)

Figure B.4: Number of earthquakes under each performance objective for LA3-RB (10% in 50 years)

Figure B.5: Number of earthquakes under each performance objective for LA3-RB (2% in 50 years)

Figure B.6: Number of earthquakes under each performance objective for LA3-RB (Near-Fault Earthquakes)

Figure B.7: Number of earthquakes under each performance objective for LA9-RA (10% in 50 years)

Figure B.8: Number of earthquakes under each performance objective for LA9-RA (2% in 50 years)

Figure B.9: Number of earthquakes under each performance objective for LA9-RA (Near-Fault Earthquakes)

Figure B.10: Number of earthquakes under each performance objective for LA9-RB (10% in 50 years)

Figure B.11: Number of earthquakes under each performance objective for LA9-RB (2% in 50 years)

Figure B.12: Number of earthquakes under each performance objective for LA9-RB (Near-Fault Earthquakes)