

NIST GCR 10-917-9

Applicability of Nonlinear Multiple-Degree-of-Freedom Modeling for Design

NEHRP Consultants Joint Venture
*A Partnership of the Applied Technology Council and the
Consortium of Universities for Research in Earthquake Engineering*



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Prepared for
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Gaithersburg, Maryland*

By
NEHRP Consultants Joint Venture
A Partnership of the Applied Technology Council and the
Consortium of Universities for Research in Earthquake Engineering

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Preface

The NEHRP Consultants Joint Venture is a partnership between the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). In 2007, the National Institute of Standards and Technology (NIST) awarded the NEHRP Consultants Joint Venture a National Earthquake Hazards Reduction Program (NEHRP) “Earthquake Structural and Engineering Research” task order contract (SB1341-07-CQ-0019) to conduct a variety of tasks. In 2008, NIST initiated Task Order 68241 entitled “Improved Nonlinear Static Seismic Analysis Procedures – Multiple-Degree-of-Freedom Modeling.” The purpose of this project was to conduct further studies on multiple-degree-of-freedom effects as outlined in the Federal Emergency Management Agency (FEMA) report, FEMA 440, *Improvement of Nonlinear Static Seismic Analysis Procedures* (FEMA, 2005).

The FEMA 440 Report concluded that current nonlinear static analysis procedures, which are based on single-degree-of-freedom (SDOF) models, are limited in their ability to capture the complex behavior of structures that experience multiple-degree-of-freedom (MDOF) response. It further concluded that improved nonlinear analysis techniques to more reliably address MDOF effects were needed. In response, work on this project was structured to include a detailed review of recent research on nonlinear MDOF modeling and the conduct of focused analytical studies to fill gaps in available information. This work was intended to improve nonlinear MDOF modeling for structural design practice by providing guidance on: (1) the minimum level of MDOF model sophistication necessary to make performance-based engineering decisions; (2) selection of appropriate nonlinear analysis methods; and (3) possible alternative approaches to analytical modeling.

The NEHRP Consultants Joint Venture is indebted to the leadership of Mike Valley, Project Director, and to the members of the Project Technical Committee, consisting of Mark Aschheim, Craig Comartin, William Holmes, Helmut Krawinkler, and Mark Sinclair, for their significant contributions in the development of this report and the resulting recommendations. Focused analytical studies were led by Mark Aschheim and Helmut Krawinkler, and conducted by Michalis Fragiadakis, Dimitrios Lignos, Chris Putman, and Dimitrios Vamvatsikos. Technical review and comment at key developmental stages on the project were provided by the Project Review Panel consisting of Michael Constantinou, Jerry Hajjar, Joe Maffei, Jack Moehle, Farzad Naeim, and Michael Willford. A workshop of invited experts was convened to obtain feedback on preliminary findings and recommendations, and input from this

group was instrumental in shaping the final product. The names and affiliations of all who contributed to this project are included in the list of Project Participants at the end of this report.

NEHRP Consultants Joint Venture also gratefully acknowledges Jack Hayes (Director, NEHRP) and Kevin Wong (NIST Technical Monitor) for their input and guidance in the preparation of this report and Ayse Hortacsu and Peter N. Mork for ATC report production services.

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Table of Contents

Preface	iii
List of Figures	xv
List of Tables.....	xxi
1. Introduction	1-1
1.1 Background.....	1-1
1.2 Project Objectives and Scope.....	1-2
1.3 Report Organization and Content	1-2
2. Summary of Current Structural Analysis Practice	2-1
2.1 Relevant Codes, Standards, and Guidelines	2-1
2.2 Current State of Engineering Practice.....	2-2
2.2.1 Use of Nonlinear Static and Nonlinear Response History Analysis Procedures.....	2-2
2.2.2 Challenges in Implementing Nonlinear Static Procedures	2-3
2.2.3 Innovative Approaches	2-3
2.2.4 Comparisons between Nonlinear Static and Nonlinear Response History Analysis Results	2-3
2.2.5 Available Guidance for Implementing Nonlinear Response History Analyses.....	2-4
2.2.6 Simplified Multiple-Degree-of-Freedom Modeling	2-4
2.2.7 Modeling of Gravity Load-Carrying Elements.....	2-4
2.2.8 Response Quantities of Interest for Design	2-5
2.2.9 Suitability of Nonlinear Static Procedures by System Type.....	2-5
3. Summary of Recent Multiple-Degree-of-Freedom Modeling Research	3-1
3.1 Scope of Review	3-1
3.2 Overview of Findings	3-1
3.3 Relative Accuracy of Analytical Methods.....	3-2
3.4 Evaluation of Available Simplified Analysis Methods.....	3-5
3.4.1 Adaptive and Multi-Mode Pushover Analysis Methods.....	3-6
3.4.2 Consecutive Modal Pushover Analysis	3-7
3.5 Methods Applicable to Specific Situations.....	3-8
3.6 Probabilistic Approaches	3-9
4. Problem-Focused Analytical Studies	4-1
4.1 Overview of Focused Analytical Studies.....	4-1
4.1.1 Approach.....	4-1
4.1.2 Scope.....	4-2
4.2 Analysis Methods	4-2

4.2.1	Nonlinear Static Analysis	4-2
4.2.2	Nonlinear Response History Analysis	4-2
4.2.3	Elastic Modal Response Spectrum Analysis	4-3
4.3	Nonlinear Static Analysis Methods	4-3
4.3.1	Procedures.....	4-3
4.3.2	Nonlinear Static Procedure	4-3
4.3.3	Modal Pushover Analysis	4-3
4.3.4	Consecutive Modal Pushover	4-5
4.3.5	Load Patterns	4-6
4.3.6	Target Displacements	4-6
4.4	Analytical Modeling	4-7
4.4.1	Software Platforms	4-7
4.4.2	Component Model Types and Parameters	4-7
4.5	Response Quantities of Interest	4-9
4.6	Ground Motions.....	4-10
4.6.1	Intensity Measures and Scaling	4-11
4.6.2	Ground Motion Subsets	4-11
4.7	System Prototypes	4-12
4.7.1	Steel Moment Frame Prototypes.....	4-12
4.7.2	Reinforced Concrete Moment Frame Prototypes	4-12
4.7.3	Reinforced Concrete Shear Wall Prototypes	4-13
4.7.4	Systems with Special Characteristics	4-13
4.7.5	Summary of Structural System Prototypes and Modeling Decisions	4-14
5.	Results of Problem-Focused Analytical Studies	5-1
5.1	Summary Results for Regular Structural System Prototypes	5-1
5.2	Steel Moment Frame Studies.....	5-4
5.2.1	Steel Moment Frame Prototypes.....	5-4
5.2.2	Nonlinear Response History Analysis	5-5
5.2.3	Single Mode Nonlinear Static Analysis.....	5-9
5.2.4	Multi-Mode Nonlinear Static Analysis.....	5-18
5.2.5	Elastic Modal Response Spectrum Analysis	5-21
5.3	Reinforced Concrete Moment Frame Studies.....	5-23
5.3.1	Reinforced Concrete Moment Frame Prototypes	5-23
5.3.2	Nonlinear Response History Analysis	5-24
5.3.3	Single Mode Nonlinear Static Analysis.....	5-29
5.3.4	Multi-Mode Nonlinear Static Analysis.....	5-36
5.3.5	Elastic Modal Response Spectrum Analysis	5-46
5.4	Reinforced Concrete Shear Wall Studies.....	5-50
5.4.1	Reinforced Concrete Shear Wall Prototypes	5-50
5.4.2	Nonlinear Response History Analysis	5-54
5.4.3	Single Mode Nonlinear Static Analysis.....	5-55
5.4.4	Multi-Mode Nonlinear Static Analysis.....	5-61
5.5	Results for Structural System Prototypes with Special Characteristics.....	5-63
5.5.1	Prototypes with Special Characteristics.....	5-63
5.5.2	Response Sensitivity to Load Patterns and Strength Irregularities.....	5-63
5.5.3	Response Sensitivity to Changes in Yield Mode.....	5-67

6.	Results from Ancillary Studies	6-1
6.1	Ancillary Study on the Effects of Ground Motion Selection and Scaling on Response Quantity Dispersion.....	6-1
6.1.1	Intensity Measure Selection.....	6-2
6.1.2	Record Subset Selection	6-3
6.2	Ancillary Study on Direct Determination of Target Displacement	6-5
6.3	Ancillary Study on Practical Implementation of Analysis Methods.....	6-6
6.3.1	Approach.....	6-7
6.3.2	Results from Nonlinear Response History Analysis.....	6-10
6.3.3	Results from Nonlinear Static Pushover and Response Spectrum Analysis	6-15
6.3.4	Results from Modal Pushover Analysis.....	6-18
6.3.5	Results from Consecutive Modal Pushover Analysis	6-21
6.3.6	Results from Extended Consecutive Modal Pushover Analysis	6-21
6.3.7	Summary Response Quantity Ratios for Building A and Building B.....	6-26
6.3.8	Performance Prediction.....	6-34
6.3.9	Summary Observations.....	6-35
7.	Modeling Recommendations for Nonlinear Analysis of Multiple-Degree-of-Freedom Effects	7-1
7.1	Representation and Effect of Geometric Nonlinearity on Story Strength Demands.....	7-1
7.2	Torsion	7-5
7.3	Gravity Framing.....	7-8
7.4	Other Simplifications.....	7-9
7.4.1	Two-Dimensional Models	7-9
7.4.2	Beam-Column Joint Modeling.....	7-9
7.4.3	Single-Bay Frame Models	7-10
7.5	Suggestions for Further Study	7-11
8.	Assessment of Nonlinear Analysis Methods	8-1
8.1	Assessment of Nonlinear Static Procedures	8-1
8.1.1	Single-Mode Nonlinear Static Analysis with Invariant Load Pattern	8-3
8.1.2	Multi-Mode Nonlinear Analysis.....	8-4
8.2	Assessment of Nonlinear Response History Analysis	8-6
8.3	Utility of Various Demand Parameters by System Type	8-7
8.4	Suggestions for Further Study	8-8
9.	Treatment of Variability in Design Decisions.....	9-1
9.1	Treatment of Variability in Current Practice	9-1
9.2	Probabilistic Approaches	9-4
9.2.1	Sources of Variability	9-4
9.2.2	Quantification of Uncertainty	9-5
9.3	Determination of Variability.....	9-6
9.3.1	Target Displacement.....	9-6
9.3.2	Lateral Dynamic Instability	9-9
9.3.3	Intensity Measure S_a	9-10
9.3.4	Intensity Measure S_{di}	9-10

9.3.5	Selection of Ground Motion Subsets	9-16
9.3.6	Demand and Capacity	9-18
9.4	Approximate Assessment of Variability in Nonlinear Response	
History Analysis	9-19
9.4.1	Define Seismic Hazard	9-20
9.4.2	Develop Multiple-Degree-of-Freedom Model.....	9-20
9.4.3	Assess Potential for Lateral Dynamic Instability.....	9-20
9.4.4	Select Intensity Measure.....	9-21
9.4.5	Perform Nonlinear Response History Analysis	9-21
9.4.6	Evaluate Demand Parameters	9-21
9.5	Introducing Variability in Nonlinear Static Analysis	9-22
9.6	Suggestions for Further Study	9-23
References	10-1
Project Participants	11-1

Supporting Documentation

(on compact disc)

Preface	iii
List of Figures	xi
List of Tables.....	xvii
Introduction.....	1-1
Appendix A: Detailed Steel Moment Frame Studies	A-1
A.1 Introduction.....	A-1
A.2 Structures Utilized in Evaluation	A-2
A.3 Nonlinear Response History Analysis	A-3
A.3.1 Component Model and Analysis Platforms	A-3
A.3.2 Analysis Model Simplification	A-4
A.3.3 Results for 2-, 4-, and 8-Story Steel Moment Frames	A-13
A.3.4 Dispersion in Seismic Input and Engineering Demand Parameters.....	A-23
A.3.5 Results for Residual Drifts.....	A-26
A.3.6 Synthesis of Nonlinear Response History Analysis Results.....	A-29
A.4 Single Mode Nonlinear Static Procedure.....	A-30
A.4.1 Nonlinear Static Analysis Options Explored	A-31
A.4.2 Results for 2-, 4-, 8-Story Steel Special Moment Frames in Pre-Capping Region.....	A-33
A.4.3 Response Predictions in Negative Tangent Stiffness Region.....	A-46
A.4.4 Synthesis of Nonlinear Static Procedure Predictions.....	A-49
A.5 Multi Mode Nonlinear Static Procedure	A-54
A.5.1 Summary Description of Procedure.....	A-54
A.5.2 Results for 4- and 8-Story Steel Moment Frames.....	A-55
A.5.3 Synthesis of Modal Pushover Analysis Predictions.....	A-61
A.6 Assessment of Elastic Response Spectrum Analysis.....	A-62
A.7 Load Pattern Sensitivity and Response Prediction for a Frame Structure with Severe Strength Irregularity	A-64
A.7.1 Load Pattern Sensitivity.....	A-64
A.7-2 Response of a 4-Story Building with Strength Irregularity ...	A-65
A.8 Incorporation of Gravity System in Analysis Model.....	A-77
A.8.1 Potential importance of Incorporating Gravity System in Analysis Model.....	A-77
A.8.2 Case Study – 4-Story Steel Moment Frame Structure	A-77
Appendix B: Detailed Reinforced Concrete Moment Frame Studies	B-1
B.1 Ground Motions.....	B-1
B.2 Structural Systems	B-2
B.3 Nonlinear Static Procedures.....	B-4
B.3.1 ASCE/SEI 41-06 Displacement Coefficient Method.....	B-4

B.3.2	N2/EC8 Method.....	B-15
B.3.3	Modal Pushover Analysis	B-16
B.3.4	Consecutive Modal Pushover	B-24
B.3.5	Modal Response Spectrum Analysis	B-31
B.4	Summary and Comparison of the Analysis Methods	B-38
B.5	Correlation between Intermediate-Level and Component-Level Demand Parameters	B-48

Appendix C: Detailed Reinforced Concrete Shear Wall Studies C-1

C.1	Introduction.....	C-1
C.2	Structures Utilized in Evaluation	C-1
C.3	Nonlinear Response History Analysis	C-3
C.3.1	Component Models.....	C-3
C.3.2	Results for 2-, 4-, and 8-Story Reinforced Concrete Shear Wall Structure with Fiber Model	C-6
C.3.3	NRHA Results for 4-story Reinforced Concrete Shear Wall Structure with Simplified Spring Model.....	C-17
C.3.4	Synthesis of Nonlinear Response History Analysis Results	C-22
C.4	Single Mode Nonlinear Static Procedure.....	C-23
C.4.1	Nonlinear Static Analysis Options Explored	C-23
C.4.2	Results for 2- and 8-Story Reinforced Concrete Shear Wall Structures Utilizing FM-ASCE41.....	C-25
C.4.3	Results for 4-Story Reinforced Concrete Shear Wall Structure Utilizing Alternative Nonlinear Static Procedures	C-31
C.4.4	Synthesis of Nonlinear Static Procedure Predictions.....	C-38
C.5	Multi-Mode Nonlinear Static Procedure.....	C-42
C.5.1	Results for 4- and 8-Story Reinforced Concrete Shear Wall Structures	C-42
C.6	Importance of Failure Mode	C-47

Appendix D: Effect of Ground Motion Selection and Scaling on Engineering Demand Parameter Dispersion D-1

D.1	Effect of Intensity Measure on EDP Dispersion	D-1
D.1.1	Observations on Dispersion	D-5
D.1.2	Observations on Bias	D-15
D.2	Use of Record Subsets to Characterize EDP Distributions	D-26
D.2.1	Subset Selection Methods used for Estimating Median EDP Values.....	D-26
D.2.2	Subset Selection Methods used for Estimating 84% EDP Values	D-28
D.2.3	Subset Selection based on FEMA P-695 scaling	D-29
D.2.4	Subset Selection on S_{di} Scaling	D-46
D.2.5	Random Subset Selection on S_{di} Scaling.....	D-55
D.2.6	Subset Selection on S_{di} Scaling	D-57
D.2.7	Random Subset Selection on S_{di} Scaling	D-68
D.3	Characterization of Distributions of Response Quantities.....	D-71
D.3.1	Distribution of EDPs Obtained with S_a Scaling.....	D-72
D.3.2	Distribution of EDPs Obtained with S_{di} Scaling	D-81
D.4	Alternative Estimation of Dispersion Using Higher Intensity Levels	D-89

Appendix E: Direct Determination of Target Displacement	E-1
E.1 Introduction	E-1
E.2 Direct Computation of Equivalent Single-Degree-of-Freedom Response	E-1
E.3 The SPO2IDA Tool	E-6
E.4 Summary	E-10
Appendix F: Practical Implementation of Analysis Methods	F-1
F.1 Approach	F-1
F.2 General Modeling Assumptions	F-2
F.2.1 Ground Motions	F-2
F.3 Structures and Models	F-4
F.3.1 Building A	F-4
F.3.2 Building B	F-9
F.4 Analysis Methods	F-14
F.4.1 Nonlinear Response History Analysis	F-14
F.4.2 Response Spectrum Analysis	F-15
F.4.3 Nonlinear Static Procedure	F-15
F.4.4 Nonlinear Static Procedure with Elastic Higher Modes	F-16
F.4.5 Modal Pushover Analysis	F-16
F.4.6 Consecutive Modal Pushover Analysis	F-16
F.4.7 Extended Consecutive Modal Pushover Analysis	F-16
F.5 Results	F-18
F.5.1 Building A Results	F-18
F.5.2 Building B Results	F-30
F.6 Summary of Observations	F-71
F.6.1 Building A	F-71
F.6.2 Building B	F-78
F.6.3 Accuracy of Estimates of Demand Parameters	F-83
F.7 Conclusions	F-91
F.7.1 Modeling and Analysis Conclusions	F-91
F.7.2 Analysis Techniques Conclusions	F-92
F.7.3 Extended Consecutive Modal Pushover Conclusion	F-93
F.7.4 Assumptions and Limitations	F-93
Appendix G: Expanded Summaries of Relevant Codes, Standards, and Guidelines	G-1
G.1 ASCE/SEI 31-03 Seismic Evaluation of Existing Buildings	G-1
G.1.1 Scope of Application	G-1
G.1.2 Applicability of Analysis Procedures	G-2
G.1.3 Other Evaluation Requirements	G-3
G.1.4 Other Modeling Direction Provided	G-5
G.1.5 Ground Motion Characteristics	G-6
G.1.6 Discussion	G-6
G.2 ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings (with Supplement No.1)	G-6
G.2.1 Scope of Application	G-6
G.2.2 Applicability of Analysis Procedures	G-7
G.2.3 Other Modeling Direction Provided	G-10
G.2.4 Additional Analysis Requirements	G-11
G.2.5 Ground Motion Characterization	G-12

	G.2.6 Discussion.....	G-12
G.3	ATC-40 Seismic Evaluation and Retrofit of Concrete Buildings.....	G-13
	G.3.1 Scope of Application	G-13
	G.3.2 Applicability of Analysis Procedures	G-14
	G.3.3 Other Modeling Direction Provided	G-16
	G.3.4 Additional Analysis Requirements	G-17
	G.3.5 Ground Motion Characterization	G-17
	G.3.6 Discussion.....	G-19
G.4	FEMA 440 Improvement of Nonlinear Static Seismic Analysis Procedures.....	G-20
	G.4.1 Scope of Application	G-20
	G.4.2 Applicability of Analysis Procedures	G-20
	G.4.3 Other Modeling Direction Provided	G-24
G.5	FEMA P-440A, Effects of Strength and Stiffness Degradation on Seismic Response	G-25
	G.5.1 Scope of Application	G-25
	G.5.2 Applicability of Analysis Procedures	G-26
G.6	FEMA 351 Recommended Seismic Evaluation and Upgrade Criteria and FEMA 352 Recommended Post-Earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings.....	G-27
	G.6.1 Scope of Application	G-27
	G.6.2 Applicability of Analysis Procedures	G-27
	G.6.3 Other Modeling Direction Provided	G-28
	G.6.4 Additional Analysis Requirements	G-30
	G.6.5 Ground Motion Characterization	G-31
	G.6.6 Discussion.....	G-32
G.7	ASCE/SEI 7-05 Minimum Design Loads for Buildings and Other Structures	G-32
	G.7.1 Scope of Application	G-32
	G.7.2 Applicability of Analysis Procedures	G-33
	G.7.3 Other Modeling Direction Provided	G-34
	G.7.4 Additional Analysis Requirements	G-35
	G.7.5 Ground Motion Characterization	G-36
G.8	NEHRP Recommended Seismic Provisions for New Buildings and Other Structures	G-36
	G.8.1 Scope of Application	G-36
	G.8.2 Applicability of Analysis Procedures	G-37
	G.8.3 Other Modeling Direction Provided	G-37
	G.8.4 Additional Analysis Requirements	G-38
G.9	PEER/ATC-72-1 Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings	G-40
	G.9.1 Scope of Application	G-40
	G.9.2 Applicability of Analysis Procedures	G-40
	G.9.3 Other Modeling Direction Provided	G-49

	Appendix H: Bibliography of Recent Multiple-Degree-of-Freedom Modeling Research	H-1
H.1	Pushover Methods of Analysis	H-1
	H.1.1 General Features/Observations	H-1
	H.1.2 Target Displacement.....	H-3

H.1.3	Load Vectors and Approaches	H-6
H.1.4	P-delta Effects	H-31
H.1.5	Modeling Choices	H-32
H.1.6	Efficacy and Limitations	H-44
H.2	Dynamic Approaches	H-75
H.2.1	Incremental Dynamic Analysis and Approximations	H-75
H.2.2	Simplified Dynamic Analysis	H-82
H.2.3	Collapse Prediction	H-89
H.2.4	Sensitivity of Response to Modeling	H-91
H.2.5	Efficacy and Limitations Relative to Empirical Results	H-95
H.3	Special Configurations and Typologies	H-95
H.3.1	Torsional or Plan Irregularities	H-95
H.3.2	Weak Stories	H-118
H.3.3	Vertical Irregularities	H-119
H.3.4	Diaphragm Flexibility	H-123
H.3.5	Base-Isolated Buildings	H-126
H.3.6	Systems with High Viscous Damping	H-130
H.4	Engineering Demand Parameters	H-137
H.4.1	Estimation of EDPs Using Different Analysis Methods and Simplified Structural Models	H-137
H.4.2	Complexity of Response and Effect of Configuration on Accuracy of Estimation of EDPs	H-145
H.5	Probabilistic Treatments	H-147
H.6	Design Methods	H-154
H.7	Applications	H-166
H.7.1	Masonry	H-166
H.7.2	Wood	H-169
H.7.3	Reinforced Concrete	H-172
H.7.4	Steel Braced Frames	H-176
H.7.5	Moment Frames	H-177
References	I-1
Project Participants	J-1

List of Figures

Figure 3-1	Different collapse mechanisms occurring in a 4-story reinforced concrete moment frame structure.....	3-3
Figure 3-2	Comparison of floor overturning moments for: (a) regular 9-story steel frame and (b) weak-story version of the same frame	3-4
Figure 3-3	Effect of modeling assumptions related to strong column-weak-beam criteria on column nonlinearity and peak story drift ratios for different analysis procedures	3-5
Figure 4-1	ASCE 41 component model, adapted from ASCE/SEI 41-06.....	4-8
Figure 4-2	Backbone curve for component model Analyt.M1	4-8
Figure 4-3	Modeling of flexural behavior with simplified plastic hinge springs and elastic elements	4-9
Figure 4-4	Response spectra for the FEMA P-695 far-field record set	4-10
Figure 5-1	Plan view of steel moment frame prototypes.....	5-4
Figure 5-2	Simplified model of steel moment frame prototypes.....	5-5
Figure 5-3	Distribution of peak story drifts over height in the 4-story steel moment frame (left) and 8-story steel moment frame (right)	5-6
Figure 5-4	Dispersion in story-level demand parameters over height in the 4-story steel moment frame subjected to different suites of ground motions: full set of 44 records (left); subset of 17 records (right)	5-7
Figure 5-5	Comparison between the distribution of story-level demand parameters (left) and local demand parameters (right) in the 4-story steel moment frame.....	5-8
Figure 5-6	Residual story drift ratios (left) and peak floor accelerations (right) in the 4-story steel moment frame	5-9
Figure 5-7	Comparison of global pushover curves for the 4-story steel moment frame, using ASCE41 (left) and Analyt.M1 (right) component models	5-10
Figure 5-8	Comparison of global pushover curves for 1- and 3-bay models of 2-, 4-, and 8-story steel moment frames showing target displacements for SF= 0.5, 1 and 2	5-10
Figure 5-9	Comparison of nonlinear response history analysis results (median and dispersion) to results from various single-mode nonlinear static analysis options for 2-story steel moment frames	5-12
Figure 5-10	Comparison of nonlinear response history analysis results (median and dispersion) to results from various single-mode nonlinear static analysis options for 4-story steel moment frames	5-13
Figure 5-11	Comparison of nonlinear response history analysis results (median and dispersion) to results from various single-mode nonlinear static analysis options for 8-story steel moment frames	5-14

Figure 5-12	Ratios of response quantity estimates to median nonlinear response history analysis results over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story steel moment frame structures at scale factors SF=1.0 (left) and SF=2.0 (right)	5-17
Figure 5-13	First mode and second mode pushover curves and equivalent SDOF systems for the 4-story steel moment frame	5-18
Figure 5-14	Comparison of nonlinear response history analysis results (median and dispersion) to results from Modal Pushover Analysis of the 4-story steel moment frame (left) and 8-story steel moment frame (right).....	5-19
Figure 5-15	Deflected shapes of the 8-story steel moment frame structure	5-21
Figure 5-16	Comparison of nonlinear response history analysis results (median and dispersion) to results from elastic modal response spectrum analysis of the 4-story steel moment frame at scale factors 0.5 (left) and 2.0 (right)	5-22
Figure 5-17	Plan and elevation of reinforced concrete moment frame prototypes	5-23
Figure 5-18	Selected results for nonlinear response history analysis of the 2-story reinforced concrete moment frame.....	5-25
Figure 5-19	Selected results for nonlinear response history analysis of the 4-story reinforced concrete moment frame.....	5-26
Figure 5-20	Selected results for nonlinear response history analysis of the 8-story reinforced concrete moment frame.....	5-27
Figure 5-21	Median nonlinear response history analysis results at scale factors of 0.5, 1.0, and 2.0 for the 4-story (upper left) and 8-story (remaining) reinforced concrete moment frames.....	5-28
Figure 5-22	Comparison of global pushover curves for 2-, 4-, and 8-story reinforced concrete moment frames showing target displacements at scale factor 0.5, 1.0 and 2.0	5-29
Figure 5-23	Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factor 2.0	5-30
Figure 5-24	Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 4-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0.....	5-31
Figure 5-25	Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 8-story reinforced concrete moment frame for displacement response quantities at scale factors 0.5 and 2.0	5-32
Figure 5-26	Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 8-story reinforced concrete moment frame for force response quantities at scale factors 0.5 and 2.0	5-33
Figure 5-27	Ratios of response quantity estimates to median nonlinear response history analysis results over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story reinforced concrete moment frame	

	structures for story drift ratio (left), story shear (middle), and overturning moment (right) at scale factors 0.5 and 2.0	5-35
Figure 5-28	Modal pushover curves for the 2-, 4-, and 8-story reinforced concrete moment frames and first-mode target displacement estimates at scale factors 0.5, 1.0, and 2.0	5-36
Figure 5-29	Comparison of nonlinear response history analysis to Modal Pushover Analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factor 2.0	5-37
Figure 5-30	Comparison of nonlinear response history analysis to Modal Pushover Analysis of the 4-story reinforced concrete moment frame for peak story shear (left) and peak overturning moment (right) at scale factors 0.5 and 2.0.....	5-38
Figure 5-31	Comparison of nonlinear response history analysis to Modal Pushover Analysis of the 8-story reinforced concrete moment frame for peak story drift ratio (left), peak story shear (middle), and peak overturning moment (right) at scale factor 2.0	5-39
Figure 5-32	Ratios of response quantity estimates to median nonlinear response history analysis results over height for Modal Pushover Analysis of 2-, 4-, and 8-story reinforced concrete moment frame structures for story drift ratio (left), story shear (middle), and overturning moment (right) at scale factors 0.5 and 2.0.....	5-41
Figure 5-33	Comparison of nonlinear response history analysis to Consecutive Modal Pushover analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factor 2.0.....	5-42
Figure 5-34	Comparison of nonlinear response history analysis to Consecutive Modal Pushover analysis of the 4-story reinforced concrete moment frame for peak story shear (left) and peak overturning moment (right) at scale factors 0.5 and 2.0.....	5-43
Figure 5-35	Comparison of nonlinear response history analysis to Consecutive Modal Pushover analysis of the 8-story reinforced concrete moment frame for peak story drift ratio (left), peak story shear (middle), and peak overturning moment (right) at scale factor 2.0	5-44
Figure 5-36	Ratios of response quantity estimates to median nonlinear response history analysis results over height for Consecutive Modal Pushover analysis of 2-, 4-, and 8-story reinforced concrete moment frame structures for story drift ratio (left), story shear (middle), and overturning moment (right) at scale factors 0.5 and 2.0	5-45
Figure 5-37	Comparison of nonlinear response history analysis to elastic modal response spectrum analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0	5-47
Figure 5-38	Comparison of nonlinear response history analysis to elastic modal response spectrum analysis of the 4-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0	5-48

Figure 5-39	Comparison of nonlinear response history analysis to elastic modal response spectrum analysis of the 8-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0	5-49
Figure 5-40	Configuration of reinforced concrete shear wall prototypes.....	5-51
Figure 5-41	Peak story drift ratios, story shears, and floor overturning moments for the 4-story reinforced concrete shear wall structure using fiber models (left) and simplified spring models (right), with ground motion scale factor 2.0	5-52
Figure 5-42	Peak story drift ratios, story shears, and floor overturning moments for the 4-story reinforced concrete shear wall structure using fiber models (left) and simplified spring models (right), with ground motion scale factor 3.0	5-53
Figure 5-43	Comparison of global pushover curves for fiber models of 2-, 4-, and 8-story reinforced concrete shear wall structures showing target displacements for scale factors 0.5, 1.0 and 2.0	5-55
Figure 5-44	Comparison of nonlinear response history analysis to single-mode nonlinear static analysis for the 2-, 4, and 8-story reinforced concrete shear wall structures using fiber models	5-56
Figure 5-45	Global pushover curve for the 4-story reinforced concrete shear wall structure using ASCE41 component models	5-58
Figure 5-46	Different multi-linear equivalent single-degree-of-freedom approximations to global pushover curves	5-58
Figure 5-47	Ratios of response quantity estimates to median nonlinear response history analysis results over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story reinforced concrete shear wall structures using different methods of target displacement estimation: ASCE41 (left) and EqSDOF (right), at scale factor 2.0	5-60
Figure 5-48	Comparison of nonlinear response history analysis results (median and dispersion) to results from Modal Pushover Analysis of the 4-story reinforced concrete shear wall (left) and 8-story reinforced concrete shear wall (right).....	5-62
Figure 5-49	Comparison of pushover and nonlinear response history analysis results for a 2-story steel shear building with slightly different story shear capacities	5-64
Figure 5-50	Pushover curves and deflection profiles for a 4-story steel shear building: base case (left) and irregular case (right)	5-65
Figure 5-51	Comparison of nonlinear response history analysis results and single-mode nonlinear static analysis results for a 4-story steel shear building: base case (left) and irregular case (right).	5-66
Figure 5-52	Comparison of nonlinear response history analysis results to nonlinear static analysis predictions of story drift ratio (left) and story shear force (right) for a 4-story reinforced concrete shear wall structure designed to be flexure-critical.....	5-67
Figure 6-1	Average spectra for ground motions used in this study	6-8
Figure 6-2	Typical floor plan of Building A	6-9

Figure 6-3	Three-dimensional SAP2000 model (left) and two-dimensional transverse frame model (right) for Building A.	6-9
Figure 6-4	PERFORM-3D model for Building B.	6-11
Figure 6-5	Dispersion in story-level demand parameters over height in Building A subjected to different ground motions.....	6-12
Figure 6-6	Dispersion in story-level demand parameters over height in Building B for longitudinal direction (H1)	6-13
Figure 6-7	Dispersion in story-level demand parameters over height in Building B for transverse direction (H2)	6-14
Figure 6-8	Modal pushover curves for Building A	6-15
Figure 6-9	Modal pushover curves for Building B.....	6-15
Figure 6-10	Comparison of nonlinear response history analysis median results to nonlinear static and elastic response spectrum analysis results for Building A.....	6-16
Figure 6-11	Comparison of nonlinear response history analysis median results to nonlinear static and elastic response spectrum analysis results for Building B in the longitudinal direction (top) and transverse direction (bottom)	6-17
Figure 6-12	Comparison of nonlinear response history analysis results to various multi-mode pushover analysis results for Building A.....	6-19
Figure 6-13	Comparison of nonlinear response history analysis results to various multi-mode pushover analysis results for Building B in the longitudinal direction (top) and transverse direction (bottom).	6-20
Figure 6-14	Two-stage Extended Consecutive Modal Pushover and deflected shapes for Building A.....	6-22
Figure 6-15	Three-stage Extended Consecutive Modal Pushover and deflected shapes for Building A.....	6-23
Figure 6-16	Comparison of Extended Consecutive Modal Pushover analysis results for Building A using different modal pushover sequences	6-25
Figure 6-17	Comparison of peak story drift ratio (left) and peak story shear (right) for Building A from different analysis options.....	6-26
Figure 6-18	Variation in ratios of peak story drift and story shear over height from nonlinear static pushover and elastic response spectrum analysis of Building A	6-28
Figure 6-19	Variation in ratios of peak story drift and story shear over height for all analyses of Building A	6-29
Figure 6-20	Variation in ratios of peak story drift and story shear over height from nonlinear static pushover and elastic response spectrum analysis of Building B in the longitudinal direction (top) and transverse direction (bottom).	6-32
Figure 6-21	Variation in ratios of peak story drift and story shear over height from all analyses of Building B in the longitudinal direction (top) and transverse direction (bottom)	6-33
Figure 7-1	Illustration of direct and indirect P-Delta effects.....	7-2
Figure 7-2	Differences between V_i and $V_{I+P-\Delta}$	7-3

Figure 7-3	Influence of direct P-Delta effect on base shear of 4-story steel moment frame.	7-4
Figure 7-4	Influence of P-Delta effect on first story column shears (left and right columns) of 4-story 1-bay steel special moment frame.....	7-4
Figure 7-5	Model of a moment frame structure, illustrating simplified modeling of P-delta effects and fishbone modeling of gravity framing	7-9
Figure 7-6	Comparison of global pushover curves ($V/W - \theta_r$) of 4-story and 8-story 1-bay and 3-bay, steel moment frame analytical models.....	7-10
Figure 7-7	Comparison of nonlinear response history analysis results for 1-bay and 3-bay models of a 4-story steel moment frame.....	7-11
Figure 8-1	Component model for analysis Option 3 in PEER/ATC-72-1 (PEER/ATC, 2010), which accounts for cyclic deterioration.....	8-7
Figure 9-1	Capacity boundary (pushover curve) of the equivalent single-degree-of-freedom system fitted to the results of a first-mode pushover of the 4-story reinforced concrete moment frame building model..	9-7
Figure 9-2	Complete set of 44 IDA curves and the summary 16 th , 50 th , 84 th percentile IDA curves in S_a - S_{di} coordinates as produced by nonlinear response history analysis for the equivalent single-degree-of-freedom fitted to a 4-story reinforced concrete moment frame building model.....	9-8
Figure 9-3	Percentile IDA curves in R - μ and S_a - S_{di} coordinates as produced by SPO2IDA for the equivalent single-degree-of-freedom fitted to a 4-story reinforced concrete moment frame building model.....	9-8
Figure 9-4	Comparison of Equivalent SDOF and SPO2IDA estimates of roof displacement with results from 44 nonlinear response history analyses and ASCE/SEI 41-06 coefficient method estimates for a 4-story reinforced concrete moment frame.....	9-9
Figure 9-5	The mean S_a hazard curve used in evaluation of the 4-story reinforced moment frame structure.....	9-12
Figure 9-6	Vertical slices and the 16 th , 50 th , 84 th percentile values of required S_a to achieve a given S_{di}	9-13
Figure 9-7	The S_{di} hazard curve, as estimated via SPO2IDA, and the power-law approximation in the area of the design point. Only minor differences appear if nonlinear response history results are used instead	9-16

List of Tables

Table 4-1	Structural System Prototypes, Nonlinear Static Analysis Methods, and Software Platforms Used in Focused Analytical Studies.....	4-14
Table 4-2	Structural System Prototypes, Component Models, and Damping Models Used in Focused Analytical Studies	4-15
Table 5-1	Summary Ratios of Response Quantity Estimates to Median NHRA Results	5-2
Table 5-2	Ratios of Response Quantity Estimates for Single-Mode Nonlinear Static Analysis of Steel Moment Frame Structures	5-16
Table 5-3	Ratios of Response Quantity Estimates for Modal Pushover Analysis of Steel Moment Frame Structures	5-20
Table 5-4	Ratios of Response Quantity Estimates for Elastic Modal Response Spectrum Analysis of Steel Moment Frame Structures	5-21
Table 5-5	Ratios of Response Quantity Estimates for Single-Mode Nonlinear Static Analysis of Reinforced Concrete Moment Frame Structures	5-34
Table 5-6	Ratios of Response Quantity Estimates for Modal Pushover Analysis of Reinforced Concrete Moment Frame Structures	5-40
Table 5-7	Ratios of Response Quantity Estimates for Consecutive Modal Pushover Analysis of Reinforced Concrete Moment Frame Structures	5-46
Table 5-8	Ratios of Response Quantity Estimates for Elastic Modal Response Spectrum Analysis of Reinforced Concrete Moment Frame Structures	5-46
Table 5-9	Ratios of Response Quantity Estimates for Single-Mode Nonlinear Static Analysis of Reinforced Concrete Shear Wall Structures	5-59
Table 5-10	Ratios of Response Quantity Estimates for Modal Pushover Analysis of Reinforced Concrete Shear Wall Structures	5-61
Table 6-1	Modal Properties for Building A	6-10
Table 6-2	Modal Properties for Building B	6-10
Table 6-3	Ratios of Response Quantity Estimates for Building A.....	6-27
Table 6-4	Ratios of Response Quantity Estimates for Building B – Longitudinal (H1) Direction	6-30
Table 6-5	Ratios of Response Quantity Estimates for Building B – Transverse (H2) Direction	6-31
Table 6-6	Predicted Levels of Performance for Building A.....	6-34
Table 8-1	Key Demand Parameters for Schematic and Preliminary Design	8-8
Table 9-1	Statistics of S_a Values Associated with Ten S_{di} Values as Determined via SPO2IDA	9-13
Table 9-2	Final S_{di} Hazard Results by Combining Inelastic and Elastic Hazard Estimates	9-15

1.1 Background

Prevailing practice for performance-based seismic design is based largely on products that have been developed under the direction of National Earthquake Hazards Reduction Program (NEHRP) agencies and other key contributors. Many of these documents recognize and allow several different performance-based analytical methods, but much of their focus is on nonlinear static analysis procedures.

The Federal Emergency Management Agency (FEMA) report, FEMA 440 *Improvement of Nonlinear Static Seismic Analysis Procedures* (FEMA, 2005), was commissioned by FEMA to evaluate and develop improvements to nonlinear static analysis procedures. In FEMA 440, differences between nonlinear static and nonlinear response history analysis results were attributed to a number of factors including: (1) inaccuracies in the “equal displacement approximation” in the short period range; (2) dynamic P-Delta effects and instability; (3) static load vector assumptions; (4) strength and stiffness degradation; (5) multiple-degree-of-freedom effects; and (6) soil-structure interaction effects.

Recommendations contained within FEMA 440 resulted in immediate improvement in nonlinear static analysis procedures and were incorporated in the development of the American Society of Civil Engineers (ASCE) standard ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings* (ASCE, 2007). The FEMA 440 report, however, also identified certain technical issues needing additional study. These included: (1) expansion of component and global modeling to include nonlinear degradation of strength and stiffness; (2) improvement of simplified nonlinear modeling to include multiple-degree-of-freedom effects; and (3) improvement of modeling to include soil-foundation-structure interaction effects.

FEMA has since supported further developmental work on the first of these issues, nonlinear degradation of strength and stiffness. The results of this work are contained in the FEMA P-440A report, *Effects of Strength and Stiffness Degradation on Seismic Response* (FEMA, 2009a).

Regarding the second of these issues, FEMA 440 concluded that current nonlinear static analysis procedures, which are based on single-degree-of-freedom (SDOF) models, are limited in their ability to capture the complex behavior of structures that experience multiple-degree-of-freedom (MDOF) response, and that improved nonlinear analysis techniques to more reliably address MDOF effects were needed.

This project was initiated by the National Institute of Standards and Technology (NIST) to address this need.

1.2 Project Objectives and Scope

Using FEMA 440 as a starting point, the purpose of this project was to improve nonlinear MDOF modeling for structural design practice by providing guidance on: (1) the minimum level of MDOF model sophistication necessary to make performance-based engineering decisions; (2) selection of appropriate nonlinear analysis methods; and (3) possible alternative approaches to analytical modeling.

The scope of the investigative effort included two primary activities. The first was to review current research and practice on nonlinear modeling of multiple-degree-of-freedom effects. The second was to conduct focused analytical studies to fill gaps in available research and synthesize available information. A workshop of invited experts was convened to obtain feedback on preliminary findings and to refine the resulting recommendations.

1.3 Report Organization and Content

This report presents findings, conclusions, and recommendations resulting from a review of available research and practice regarding nonlinear MDOF effects, and focused analytical studies targeted to investigate selected issues related to MDOF modeling and response characteristics. The report is organized into two parts: (1) a main body of summary information and conclusions; and (2) supporting documentation contained in a series of appendices presenting the results of detailed analytical studies and findings from research and practice reviews.

Chapter 1 introduces the project context, objectives, and scope of the investigation.

Chapter 2 summarizes current structural analysis practice for nonlinear multiple-degree-of-freedom modeling. It lists current codes, standards, and guidelines that serve as resources for seismic analysis and design, and discusses how nonlinear modeling techniques vary among practitioners. Expanded summaries of relevant engineering resources are provided as supporting documentation in Appendix G.

Chapter 3 summarizes recent MDOF modeling research published between 2002 and the present. It provides a brief evaluation of currently available pushover methods and discusses new methods recently proposed for general application and specific situations. A list of relevant publications, including full citations and abstracts, is provided as supporting documentation in Appendix H.

Chapter 4 describes the approach used in selecting, scoping, and conducting focused analytical studies on issues related to MDOF modeling and response characteristics. It identifies the analysis procedures, modeling techniques, software, response

quantities of interest, ground motion input, and structural prototypes used to assess the accuracy and complexity of MDOF analytical methods employed in practice.

Chapter 5 summarizes the results obtained from problem-focused analytical studies conducted on steel moment-resisting frames, reinforced concrete moment-resisting frames, and reinforced concrete shear wall systems. Detailed results from these studies are reported and provided as supporting documentation in Appendices A, B, and C.

Chapter 6 summarizes ancillary studies undertaken in support of the system-specific problem-focused studies. These studies include effects of ground motion selection and scaling on dispersion, direct determination of target displacement, and practical application to the analysis of special-case buildings that have been encountered in practice. Detailed results from these studies are reported and provided as supporting documentation in Appendices D, E, and F.

Chapter 7 provides practical recommendations for nonlinear modeling of MDOF effects, based on the results of problem-focused analytical studies and synthesis of recently published research.

Chapter 8 provides guidance on the selection of appropriate nonlinear analysis methods, based on the results of problem-focused analytical studies and synthesis of recently published research.

Chapter 9 describes approaches for incorporating uncertainty into various steps of static and dynamic analyses. It discusses target displacement estimation and evaluation of lateral instability on the basis of first mode nonlinear static analysis, addresses the treatment of variability in dynamic response of MDOF systems, and recommends possible extensions of these approaches to introduce consideration of variability into nonlinear static analyses.

Supporting documentation, consisting of Appendices A through H, is provided on the compact disc accompanying this report.

Summary of Current Structural Analysis Practice

Current structural analysis practice is based on available codes, standards, and guidelines for performance-based seismic design, but the approaches employed by individual practitioners can differ from prescribed methods for a variety of reasons. Differences in the focus of their practice, the need to resolve engineering issues not fully addressed in available resources, past experience, familiarity with more recent research, or judgment that the documents are deficient in some way are all reasons that can cause deviations between individual analytical practice and standardized procedures.

This chapter identifies relevant codes, standards, and guidelines that serve as resources for performance-based seismic design, and summarizes how current analytical practice deviates from the procedures defined in these resources.

2.1 Relevant Codes, Standards, and Guidelines

Beginning with the publication of ATC-40, *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996), guidance on nonlinear static analysis procedures has been provided in many engineering resource documents. Some of the most recent publications are the result of a series of successive documents, each aimed at improving the accuracy and applicability of the procedures over earlier publications.

The following codes, standards, and guidelines are relevant to multiple-degree-of-freedom modeling in current engineering practice:

- ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996)
- FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* (FEMA, 2000b)
- FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (FEMA, 2000c)
- ASCE/SEI 31-03, *Seismic Evaluation of Existing Buildings* (ASCE, 2003)
- FEMA 440, *Improvement of Nonlinear Static Seismic Analysis Procedures* (FEMA, 2005)
- ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings* (ASCE, 2007)

- FEMA P-440A, *Effects of Strength and Stiffness Degradation on Seismic Response* (FEMA, 2009a)
- FEMA 750, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1: Provisions* (FEMA, 2009c)
- ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010)
- PEER/ATC-72-1, *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings* (PEER/ATC, 2010)

Detailed summaries of the analysis procedures as defined in these documents, including scope, applicability, direction on modeling, analytical requirements, and ground motion characterization, are provided in Appendix G. These summaries also include a list of response quantities (i.e., demand parameters) that can be evaluated using the procedures as they have been defined in the respective documents.

2.2 Current State of Engineering Practice

Analytical procedures defined in currently available codes, standards, and guidelines are generally indicative of structural engineering practice. Most practitioners employ linear static and linear dynamic analysis procedures for nearly all design, evaluation, and rehabilitation projects. Some advanced practitioners routinely use nonlinear analysis procedures in their work.

In order to understand the nature of current analytical practice, information was obtained from practicing engineers who routinely perform nonlinear structural analysis methods. Observations from practitioners with regard to nonlinear static and nonlinear response history analyses are provided in the sections that follow. These observations are shared to identify the current state of engineering practice as context for the present study. Inclusion in this report, however, is not an endorsement of the validity of any individual perception or observation regarding different analysis procedures currently in use.

2.2.1 *Use of Nonlinear Static and Nonlinear Response History Analysis Procedures*

Nonlinear analysis procedures are thought to be less conservative (i.e., more accurate) than linear analysis procedures. They are thought to provide insight on behavior that would not otherwise be available, such as how a yielding mechanism might affect subsequent building response.

Nonlinear static procedures are commonly used for rehabilitation of low-rise buildings and low-rise systems with concentrated nonlinearity, such as buckling restrained braced frames and special truss moment frames. They are not considered useful for high-rise buildings. They are thought to predict global displacements well, but do a poor job of predicting member forces.

Nonlinear response history analysis procedures are generally used for rehabilitation design and non-code design of mid-rise and high-rise buildings, buildings with dampers or isolators, buildings targeting higher performance objectives, systems with highly nonlinear response, and mid-rise buckling restrained brace frame systems. Some practitioners use nonlinear response history analyses almost exclusively, and developed their own programs to perform such analyses before they were commercially available.

2.2.2 Challenges in Implementing Nonlinear Static Procedures

Challenges associated with nonlinear static analysis modeling are similar to those for nonlinear response history analysis modeling, but the analytical results are considered less valuable. Even with well-prepared models representing all modes of failure, nonlinear static analyses are thought to miss important behaviors. Challenges associated with nonlinear static procedures include the following:

- Unrealistic concentration of nonlinear response
- Inability to capture variations in nonlinear response associated with record-to-record variability
- Difficulty in selecting appropriate load vectors
- Limited applicability to three-dimensional models
- Inability to capture bi-directional and torsional response
- Modeling of multiple towers on a single podium base
- Non-convergence associated with sudden drops in component strength
- Component-level acceptance criteria that do not accurately represent system-level performance

2.2.3 Innovative Approaches

In response to the challenges associated with performing nonlinear analysis, practitioners have implemented certain innovative approaches. These include revised modeling of component strength drops to improve numerical convergence, performance of project-specific testing to determine modeling parameters, and explicit consideration of soil-structure interaction effects. Some practitioners have attempted cyclic nonlinear static analyses to capture degradation of fiber elements and quasi-static nonlinear response history analyses to capture load redistribution.

2.2.4 Comparisons between Nonlinear Static and Nonlinear Response History Analysis Results

In general, nonlinear static analyses are considered less informative and less reliable than nonlinear response history analyses. Practitioners report that both procedures produce similar results for roof drift. For a given model, results are similar near yield, but diverge at highly nonlinear responses. Nonlinear static procedures are

thought to produce story overturning moments that are overconservative at the base and story shears that are unconservative over the height of a structure.

2.2.5 Available Guidance on Implementing Nonlinear Response History Analyses

Practitioners find guidance currently provided for performing nonlinear response history analysis to be inconsistent, incomplete, incoherent, and disorganized. Available resources are often detailed and prescriptive, but they tend to overgeneralize results, do not provide clear rationale for the requirements, and do not allow room for judgment. Some feel that available guidance is too academic for frequent use and requires large effort for small return.

For improved implementation of nonlinear response history analysis in practice, additional guidance is needed in the following areas:

- Ground motion selection and scaling, and how to address multiple source types
- Orientation of ground motion records (i.e., rotating and flipping)
- Appropriate stiffness for modeling cracked concrete
- Equivalent viscous damping at element and system levels (both magnitude and characterization)
- Modeling of below-grade mass and stiffness, and where to input ground motion
- Appropriate hysteretic modeling for various behaviors
- Modeling of interactions (e.g., shear-moment, shear-tension, and biaxial bending)
- Treatment of cyclic degradation, and how to establish an initial capacity boundary
- Appropriate statistical quantities (e.g., maximum, mean, median, or other) for failure rates of ductile versus brittle elements and lateral load-resisting versus gravity load-carrying elements

2.2.6 Simplified Multiple-Degree-of-Freedom Modeling

Simplified multiple-degree-of-freedom modeling employed in practice includes stick models, which are generally used for preliminary analysis or independent checking of more complex models; crude models enveloping diaphragm stiffness; and fishbone models of gravity framing. Stick model representations of concrete core walls have proven to be unreliable.

2.2.7 Modeling of Gravity Load-Carrying Elements

Advanced practitioners commonly consider P-Delta effects associated with gravity load-carrying elements by including at least one P-Delta column in their models. They also often model the gravity framing, even if only in a simplified manner, to

assess displacement compatibility demands. Use of gravity load-carrying elements in models has been found to help with serviceability checks or stability issues.

2.2.8 Response Quantities of Interest for Design

Understanding the controlling mechanism is considered important to understanding structural response. The following individual response quantities (i.e., demand parameters) are considered most useful for design:

- Peak story drift
- Peak plastic hinge rotation (in moment frame beams and coupling beams)
- Peak shear force (in shear walls)
- Peak floor acceleration
- Peak forces for failure-sensitive components
- Residual deformation
- Controlling mechanism

The above quantities, however, are not necessarily treated properly in currently available codes, standards, and guidelines. In some cases, they are treated inconsistently or not at all. Some feel that highly prescriptive and unnecessarily complex provisions have contributed to over-reliance on computers and loss in the ability to employ engineering judgment in interpreting analytical results.

2.2.9 Suitability of Nonlinear Static Procedures by System Type

Nonlinear static procedures are considered most suited for short, stiff, and regular buildings. They are also most useful on small retrofit projects. Nonlinear static procedures are considered least suited for tall buildings, torsionally irregular systems, non-orthogonal systems (i.e., coupled response), highly nonlinear or degrading systems, systems with dampers, and multistory moment frame systems.

Chapter 3

Summary of Recent Multiple-Degree-of-Freedom Modeling Research

The scope of the investigative effort included an extensive review of existing research on multiple-degree-of-freedom (MDOF) modeling. The purpose of this review was to identify and assemble recent available research that could be used to address the challenges associated with nonlinear MDOF modeling in current engineering practice. Information from this review was used as a starting point for problem-focused analytical studies described in Chapter 4.

This chapter summarizes results from recent research considered most relevant to nonlinear MDOF modeling. It discusses how comparisons between different analytical procedures can be sensitive to the properties of a structural system or the analytical model of that system. It cites research evaluating different nonlinear static analysis procedures, identifies possible procedures of interest, and lists studies related to probabilistic consideration of structural response quantities.

A comprehensive list of relevant publications, including full citations and abstracts, is provided as supporting documentation in Appendix H.

3.1 Scope of Review

Review of available research was focused on identifying possible improvements to nonlinear static analysis procedures and possible methods of performing simplified nonlinear response history analyses. It also included investigation of possible solutions to specific MDOF challenges related to flexible diaphragm systems, torsionally irregular systems, and highly-damped systems. The extent of the review was limited to sources published in 2002 and later.

Research on possible improvements to nonlinear static (i.e., pushover) analysis procedures included review of alternative methods of estimating peak displacement, and variations on applying and/or combining results from different load vectors (e.g., invariant load patterns, adaptive load patterns, single-mode pushovers, and multi-mode pushover procedures).

3.2 Overview of Findings

There has been an extensive volume of research investigating nonlinear MDOF effects. A number of variations, complications, and enhancements to nonlinear static analysis procedures have been independently proposed and investigated in the

literature. Anecdotal results from isolated, independent studies, however, are difficult to aggregate into a cohesive set of conclusions.

To date, there has not been a single comprehensive study that evaluates different analytical methods and their limitations, while considering a broad variety of response quantities across a broad range of structural systems and configurations, and uses a consistent suite of ground motion records (or response spectra). Based on a review of available research on nonlinear MDOF effects, the following conclusions were drawn:

- There is, unfortunately, no one single method that has been identified in the literature as being uniformly applicable, consistently accurate, and relatively simple in capturing all nonlinear MDOF response quantities of interest.
- Nonlinear static analysis methods generally are incapable of representing the development of multiple inelastic mechanisms, the variety of modal interactions, and the timing that produce maxima in nonlinear response history analyses.
- Of primary importance is whether or not simplified nonlinear static analysis methods can produce good estimates of the central tendency (mean or median) of a variety of response quantities needed for design.
- Any evaluation of the accuracy of nonlinear static analysis methods relative to nonlinear response history analysis must be conditioned on assumptions made in the analytical modeling.
- Because clear bounds defining the domain over which pushover procedures can be relied upon are presently lacking, the accuracy of various invariant load patterns, adaptive load patterns, single-mode pushovers, and multi-mode pushover procedures should be investigated.
- There is practical interest in the potential of recently proposed pushover methods to provide point estimates of the central tendency (mean and median) response quantities observed in nonlinear response history analyses.
- It is possible that a relatively small set of ground motions could be used to establish relevant response statistics in a design context.
- Inelastic spectral displacement, S_{di} , should be considered for use as an intensity measure and for simplified record selection.

These findings and conclusions were used to structure the problem-focused and ancillary studies conducted on this project. The following sections describe these findings in more detail and cite relevant research in support of these conclusions.

3.3 Relative Accuracy of Analytical Methods

In nearly all research, results from nonlinear response history analyses are the benchmark by which nonlinear static and other simplified analysis methods are

evaluated. In some cases, experimental results are used as a benchmark. For comparison with experimental results, the fidelity of the model is very important. Where nonlinear response history analysis results are used as the benchmark, the use of high-fidelity models is not necessarily considered essential.

Nonlinear dynamic response involves fairly complex interactions among the evolving modes of the structure. The relative accuracy of nonlinear analysis procedures depends on the following characteristics of the structure and the model:

- Response quantity of interest and location in model
- Intensity of excitation and degree of inelastic response
- Type of inelastic mechanism and number of potential mechanisms
- Modeling assumptions
- Ground motion record-to-record variability

Figure 3-1 illustrates how the occurrence of different failure mechanisms can affect structural response. In this example, a suite of 44 ground motions caused six different collapse mechanisms to develop in a 4-story reinforced concrete frame structure (Haselton and Deierlein, 2007). The occurrence of different inelastic mechanisms will increase dispersion in the values of many response quantities of interest.

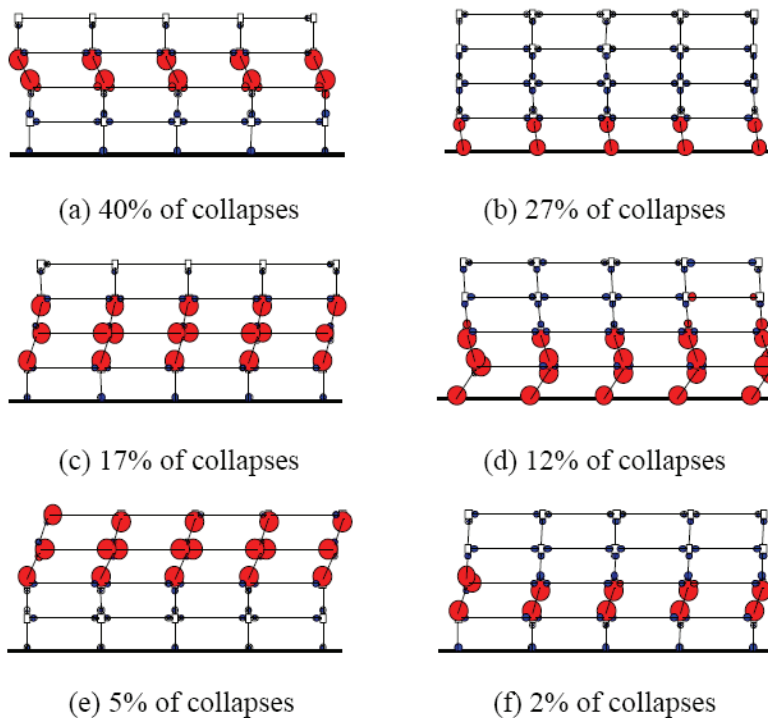


Figure 3-1 Different collapse mechanisms occurring in a 4-story reinforced concrete moment frame structure (Haselton and Deierlein, 2007).

Figure 3-2 shows a comparison between estimates of floor overturning moments determined for a 9-story steel moment frame structure using two different multi-mode pushover methods and nonlinear response history analysis. Results for a regular frame (Figure 3-2a) suggest that relatively good estimates can be obtained using an energy-based multi-mode approach. Figure 3-2b, however, shows that the presence of a weak story changes the relative accuracy of the methods, and invalidates this conclusion as a general finding.

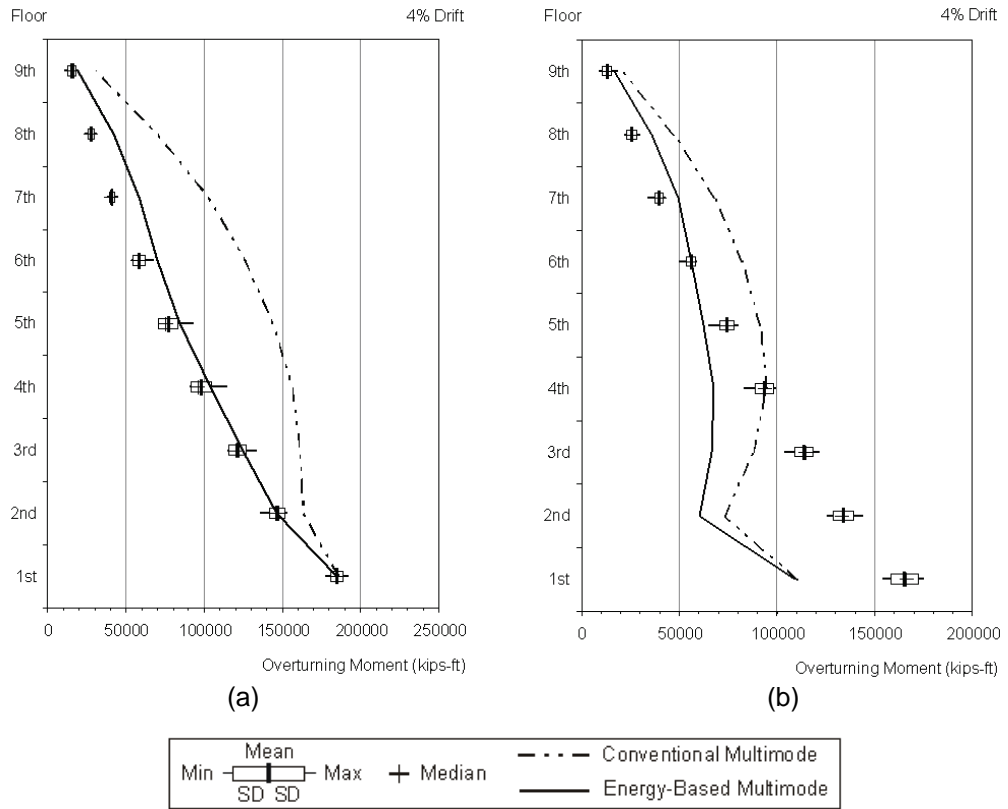


Figure 3-2 Comparison of floor overturning moments for: (a) regular 9-story steel frame and (b) weak-story version of the same frame (FEMA, 2005).

Also shown in Figure 3-2 are error bars that indicate the statistical variation in overturning moments predicted using nonlinear response history analysis. Multi-mode pushover results that plot outside these error bars indicate that any single nonlinear response history analysis (using a suitably scaled ground motion record) would produce a more consistent estimate of mean response than either of the nonlinear static analysis methods shown in the figure. This result can be generalized for other response quantities and other nonlinear static analysis procedures (FEMA, 2005).

Figure 3-3 illustrates how different modeling assumptions can affect the relative behavior of models using different analytical procedures. The figure also shows that the relative strength of beam and column elements in the model is varied. In the strong-column-weak-beam (SCWB) model, the columns remained elastic, while in

the actual model, the columns were allowed to yield. This modeling difference, and the extent of column nonlinearity, had a different effect on the results plotted for each analytical procedure. Results for inelastic drift ratios (IDR) for first mode pushover analyses were insensitive to this change, but results for both the nonlinear response history analyses (NRHA) and multi-mode pushover analyses were divergent (Kunnath and Erduran, 2008).

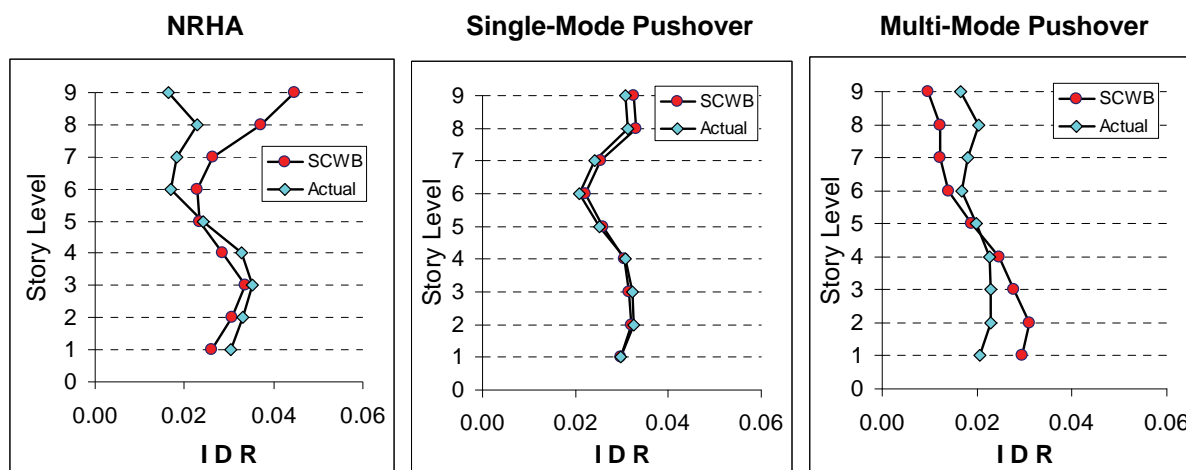


Figure 3-3 Effect of modeling assumptions related to strong column-weak-beam criteria on column nonlinearity and peak story drift ratios for different analysis procedures (Kunnath and Erduran, 2008).

Divergent peak story drift ratios were also observed depending on the choice of element model (e.g., distributed versus lumped plasticity), whether or not P-Delta effects were modeled, and variability in the ground motion record set selected for the analysis suite. These and other results illustrate that conclusions regarding the relative accuracy of nonlinear static and nonlinear response history analyses are dependent upon the details of the structure and the analytical model.

3.4 Evaluation of Available Simplified Analysis Methods

There are many examples in the literature that describe variations and enhancements that can be applied to currently available nonlinear analysis methods. In recent years, fairly complex methods have been proposed involving: (1) single run pushover analyses with vectors that represent the effects of multiple modes; (2) explicit consideration of response in multiple modes using step-by-step analyses; and (3) progressive changes in the load (or displacement) vector applied to the structure.

Each has been individually proposed for use and evaluated relative to some benchmark that is intended to illustrate its applicability for solving a specific problem or addressing a specific structural system type. In many cases, the proposed methods are somewhat complex to implement and have not been verified for general applicability to all systems or situations. Experience with these methods is limited, and their relative complexity is a barrier to implementation.

Simplified analysis methods considered relevant to this project must have the ability to produce reasonable estimates of the response quantities of interest for engineering design and must not be more complex or more time-consuming to perform than corresponding nonlinear response history analyses. Selected multi-mode pushover analysis methods described in the literature appear to exhibit these characteristics.

3.4.1 Adaptive and Multi-Mode Pushover Analysis Methods

Adaptive methods feature the ability to adjust pushover load vectors to account for higher mode effects and stiffness degradation on the story force distribution. Multi-mode pushover methods utilize the inertial force distribution in each mode to determine the load vectors. Combining pushover demands in each mode in an approximate way using modal combination techniques provides an estimate of the total inelastic demand on a system.

The Modal Pushover Analysis (MPA) method was developed by Chopra and Goel (2001). As originally conceived, inaccuracies in the estimation of local response quantities (e.g., plastic hinge rotations) were recognized, and the MPA method was suggested for estimating global response quantities such as peak floor or roof displacements and story drifts. A variant known as the Modified Modal Pushover Analysis (MMPA) method (Chopra et al., 2004) was conceived in order to avoid difficulties posed by reversals in higher mode capacity curves. In the MMPA, higher mode responses are assumed to remain elastic. A subsequent modification extended the MPA to determine local member forces (Goel and Chopra, 2005). This extension was necessary for cases where modal combinations in the MPA and MMPA procedures resulted in forces that were in excess of member capacities.

Onem (2008) considered various single-mode and multi-mode pushover methods applied to moment frames 4 to 20 stories in height and dual systems 8 to 24 stories in height. Onem considered procedures which combine multi-mode effects at each pushover step, including Incremental Response Spectrum Analysis (IRSA), defined by Aydinoglu (2003), and Displacement Adaptive Pushover (DAP), proposed by Antoniou and Pinho (2004), as well as other, more simplified single-mode and multi-mode procedures. First mode pushover analyses produced accurate estimates of peak floor displacements in low- and mid-rise structures, but tended to overestimate peak displacements in the taller structures. Onem showed that more complex methods could provide improved estimates over more simplified methods in the case of story drift and plastic hinge rotation in moment frame systems and story shears in dual systems. No method, however, was identified that could consistently provide more reliable estimates of these response quantities for all structural models considered.

Diotallevi et al. (2008) compared MPA with various single load vector (e.g., first mode, triangular, and square-root-sum-of-the-squares) and adaptive pushover methods on reinforced concrete frame systems 3 to 12 stories in height. Ground motions were scaled to achieve prescribed peak roof drifts. For all methods

considered, MPA resulted in the least average error on estimates of story shear for nearly every building examined, including regular and irregular configurations. The best method for predicting story drifts was less clear. Although MPA was one of the better predictors, some methods were better than others for particular building types.

Kalkan and Kunnath (2007) considered reinforced concrete moment frames (7 and 20 stories in height) and steel moment frames (6 and 13 stories in height). In their implementation of the pushover method in accordance with FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA, 2000d), they considered the larger of the results obtained in first mode and uniform load pattern pushover analyses. They reported that the FEMA 356 procedure underestimates story drifts in the upper stories and overestimates story drifts in the lower stories, while the MMPA method may underestimate or overestimate story drifts in the upper stories. They reported that the MMPA was inconsistent in its ability to predict yielding in plastic hinge elements.

Results reported in FEMA 440 (FEMA, 2005) suggest that the accuracy of the MMPA procedure varies with structural system, configuration, and drift level (i.e., intensity of inelastic response). Goel (2005) reports that the MPA procedure does not provide a reasonable estimate of response where a soft first story exists. These and other results in the literature leave a mixed impression as to the ability of the MPA and MMPA procedures to reasonably predict estimates of peak floor displacements, story drifts, plastic hinge rotations, and story shears, and suggest that further study is warranted.

3.4.2 Consecutive Modal Pushover Analysis

The Consecutive Modal Pushover (CMP) analysis procedure was developed by Poursha et al. (2009). In the CMP, structural response quantities are determined by enveloping the results of multi-stage and single-stage pushover analyses. This procedure is of interest because: (1) interaction of multiple modes is considered in a way that will allow different inelastic mechanisms to form; and (2) member forces resulting from the analysis are consistent with member capacity limits.

The CMP considers up to three modes, applied consecutively in stages in a single pushover analysis after the application of gravity loads. In this way, the CMP may come closest to representing higher mode responses that take place when the peak displacement response is realized dynamically. Modal forces applied in each stage have an invariant pattern.

The basic approach is that first mode forces are applied until a certain displacement is reached. The analysis continues with incremental forces applied using a second mode distribution and then a third mode distribution. In the CMP, there are two or three separate pushover analyses, and the maximum values of the response quantities

from each pushover analysis are retained. Implementation of the procedure includes the following:

- Three analyses are required for buildings having a fundamental period of vibration of 2.2 seconds or higher.
- The first pushover analysis uses an inverted triangular load pattern for medium-rise buildings and a uniform force distribution for high-rise buildings.
- The second pushover analysis consists of a sequence of first and second mode forces. The first mode forces are applied until the roof displacement equals $\alpha_1 \delta_i$, where α_1 is the first mode modal mass ratio and δ_i is the target displacement determined for the first mode. Upon reaching $\alpha_1 \delta_i$, incremental forces are applied in a second mode pattern until the roof displacement increases by $(1-\alpha_1)\delta_i$.
- The third pushover analysis, if done, consists of a sequence of first, second, and third mode forces. The first mode forces are applied until the roof displacement equals $\alpha_1 \delta_i$. Upon reaching $\alpha_1 \delta_i$, incremental forces are applied in a second mode pattern until the roof displacement increases by $\alpha_2 \delta_i$. At this point, incremental forces are applied in a third mode pattern until the roof displacement increases by $(1-\alpha_1-\alpha_2)\delta_i$.

Poursha et al. (2009) reports that relatively good estimates of story drifts and plastic hinge rotations are obtained using this approach on 10-, 20- and 30-story buildings. The estimates are generally better than those obtained with the MPA method, particularly for plastic hinge rotations. Similar to the MPA method, however, there are floors for which significant errors occur relative to the results of nonlinear response history analysis.

This method would appear to have an advantage over the MPA method in that nonlinear interactions among the modes are explicitly modeled, and capacity limits on demands (e.g., shear forces in hinging beams) are inherently represented in the analyses. Further study of this method is warranted, along with potential improvements such as varying the signs of the higher modes in additional pushover analyses, and relating the amplitude of the displacement increments to the frequency content of the ground motion.

3.5 Methods Applicable to Specific Situations

Though not generally applicable, methods that address to certain specific situations might be useful in practice. Appendix H contains abstracts of methods developed specifically to address torsional (plan) irregularities, weak story behavior, vertical irregularities, diaphragm flexibility, base-isolated buildings, and systems with supplemental damping. Although these methods are not explicitly investigated in this project, these abstracts are provided as a resource to practicing engineers and

researchers. Inclusion of the abstracts in this report, however, should not be considered a recommendation or endorsement as to the validity of these methods.

3.6 Probabilistic Approaches

Advances in the application of probabilistic concepts for design have occurred since the publication of FEMA 350 *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* (FEMA, 2000a) and FEMA 351 *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* (FEMA, 2000b), which presented a design-oriented treatment of randomness and uncertainty based on the work of Cornell et al. (2002). The following recent research results are related to probabilistic estimation of response quantities of interest:

- Vamvatsikos and Cornell (2002) describe the Incremental Dynamic Analysis (IDA) procedure as a way to evaluate response for any demand parameter, conditioned on a given ground motion intensity. Typically, the 5% damped spectral acceleration at the first mode period, $S_a(T_1, 5\%)$, is selected as the intensity measure. The hazard curve for this intensity measure is generally available, which allows the hazard curve for the demand parameter to be defined. A hazard curve indicates the mean annual frequency of exceeding specific values of a parameter, such as peak ground acceleration (PGA) or an individual demand parameter.
- Jalayer and Cornell (2009) describe alternate methods for making probability-based assessments, such as estimates of parameter values associated with specific probabilities of exceedance and direct probabilistic performance assessments. So-called “single-stripe” and “double-stripe” methods are described (made at one or two values, respectively, of the selected intensity measure). These methods can be used for making “local” probabilistic demand assessments using as few as 20 to 40 nonlinear response history analyses, rather than the hundreds of analyses typically required to develop a full IDA curve.
- Azarbakth and Dolsek (2007) provide methods to select a subset of ground motions for use in IDA to estimate median response quantities of interest. In effect, the ground motion records that result in nearly median response values for a simple structural model are selected for nonlinear response history analysis of the complete structural model. When applied to a 3-story building, median estimates of peak roof displacement and peak story drift made using just four nonlinear response history analyses (i.e., using the four best ground motion records) were within 10% of the median values determined using 24 or 30 records. However, the applicability of such an approach for peak displacements after the onset of strength and stiffness degradation (i.e., post-capping region) is unclear.

- Tothong and Cornell (2006) cite several reasons why the peak displacement of an inelastic oscillator, S_{di} , is preferable to $S_a(T_1)$ as a basis (or intensity measure). As justification for their work developing an attenuation relationship for S_{di} , they say that: (1) S_{di} provides a better nonlinear response prediction of MDOF structures than a prediction based on $S_a(T_1)$ or PGA; (2) S_{di} should reduce the so-called peak-valley effects associated with period elongation (due to nonlinear response) into a spectral valley; and (3) S_{di} can reduce the potential bias in scaling the amplitude of ground motions thus simplifying record selection by avoiding strong emphasis on other ground-motion record properties such as epsilon (Baker and Cornell, 2005), magnitude, and distance. To maintain these advantages for taller buildings (where higher modes are more significant), S_{di} should be augmented by consideration of higher mode contributions.

The results of the MDOF studies in FEMA 440, which were analyzed for fixed values of roof drift, are nearly the same as those that would have been obtained for fixed values of S_{di} (equal to peak roof drift divided by first mode participation factor), and conclusions in FEMA 440 are essentially valid for S_{di} scaling. These results suggest the possibility that a relatively small set of ground motions could be used to establish relevant response statistics in a design context and should be studied further. Also, the use of S_{di} as an intensity measure could be used to simplify record selection.

Problem-Focused Analytical Studies

This chapter summarizes the characteristics of structural analysis procedures and analytical models that were undertaken to fill gaps in available research and synthesize available information into recommendations for improving nonlinear multiple-degree-of-freedom (MDOF) modeling. It describes the different analysis methods, component modeling techniques, software packages, response quantities of interest, ground motion records, and structural system characteristics that were investigated.

Detailed reports on the analytical studies and supporting ancillary studies are provided in Appendices A through F as supporting documentation accompanying this report.

4.1 Overview of Focused Analytical Studies

Past research indicates that nonlinear static analysis methods are generally incapable of capturing modal interactions and multiple inelastic mechanisms that are possible in MDOF systems. It is also apparent that there is no one single method that has been identified in the literature as being uniformly applicable, consistently accurate, and relatively simple in capturing all nonlinear MDOF response quantities of interest.

There is, however, practical interest in the potential of recently proposed pushover methods to provide point estimates of the central tendency (mean and median) response quantities observed in nonlinear response history analyses. Since clear bounds defining the domain over which pushover procedures can be relied upon are presently lacking, the accuracy of selected invariant load, single-mode, and multi-mode pushover procedures have been investigated using problem-focused analytical studies and supporting ancillary studies.

4.1.1 Approach

The approach taken in conducting problem-focused analytical studies emphasizes practical application. Nonlinear response history analysis, which is the most rigorous analysis method available for prediction of structural response to earthquake ground motion, is used as a benchmark against which the accuracy and complexity of less rigorous methods are measured. It is intended that all of the methods and models employed in these studies lie within the realm of advanced structural engineering practitioners to understand and implement.

4.1.2 *Scope*

Evaluation of the accuracy of nonlinear static analysis methods relative to nonlinear response history analysis must be conditioned on the details of the structural system and the assumptions made in the analytical model. Analytical studies have been structured to perform independent evaluations of multiple methods of analysis, to consider a broad variety of response quantities of interest across a broad range of structural systems and configurations, and to use consistent suites of ground motion records and response spectra. Rather than seeking to calibrate and further refine the approximate methods that are available at present, studies are intended to assess the accuracy and complexity of these methods as they would be employed in practice.

The following sections describe variations in analysis methods, component modeling, response quantities of interest, ground motions, and structural system characteristics included in the scope of the focused analytical studies and supporting ancillary studies.

4.2 **Analysis Methods**

The following analysis methods were employed in conducting focused analytical studies:

- Nonlinear Static Analysis
- Nonlinear Response History Analysis
- Elastic Modal Response Spectrum Analysis

4.2.1 *Nonlinear Static Analysis*

Nonlinear static analysis (NSA) procedures utilize single-degree-of-freedom (SDOF) representations of MDOF systems to relate story drifts and component actions to a global displacement demand parameter (i.e., target displacement) through a pushover curve. Ground motions are represented as response spectra. Variations in nonlinear static analysis procedures investigated in the focused analytical studies are described below.

4.2.2 *Nonlinear Response History Analysis*

Nonlinear response history analyses (NRHA) utilize detailed nonlinear models subjected to a suite of ground motion records to produce estimates of component actions for each degree of freedom included in the model. At each intensity level, multiple time histories produce a distribution of results for each response quantity so that variability in structural response can be explicitly considered. Nonlinear response history analysis results are used as a benchmark for comparison with other methods in the focused analytical studies.

4.2.3 Elastic Modal Response Spectrum Analysis

Elastic modal response spectrum analyses (RSA) utilize the equal displacement approximation to extrapolate linear behavior to nonlinear displacement response. Demands are calculated through linear-elastic analysis using lateral load patterns proportional to the modes of vibration. Response quantities are obtained through square-root-sum-of squares (SRSS) combination of modal results, which are then scaled to estimate nonlinear response using coefficients in ASCE/SEI 41-06 (ASCE, 2007). Elastic modal response spectrum analyses were studied as a possible alternative to nonlinear static analysis procedures.

4.3 Nonlinear Static Analysis Methods

4.3.1 Procedures

The nonlinear static procedures listed below were specifically investigated in focused analytical studies:

- Nonlinear Static Procedure
- Modal Pushover Analysis
- Consecutive Modal Pushover

These procedures are currently used in practice or have been investigated in the literature with some level of success. Other procedures reported in the literature, such as Incremental Response Spectrum Analysis (Aydinoglu, 2003) and Displacement Adaptive Pushover (Antoniou and Pinho, 2004), were judged too complex for routine use in practice and were not studied further.

4.3.2 Nonlinear Static Procedure

The Nonlinear Static Procedure (NSP), defined in ASCE/SEI 41-06, is the basic pushover procedure used in practice. It utilizes the application of a single invariant load pattern to determine structural response. As defined in ASCE/SEI 41-06, the load pattern is based on the first mode shape, and the target displacement is determined using the coefficient method.

4.3.3 Modal Pushover Analysis

The Modal Pushover Analysis (MPA) procedure, initially proposed by Chopra and Goel (2001), combines results from multiple pushover analyses generated by assuming load patterns based on the first mode and one or more higher modes. The method was extended with a second analysis phase to better estimate member forces (Chopra and Goel, 2004), but this adaptation has not been included in the focused analytical studies conducted herein.

The MPA procedure, as implemented, follows the more common application among practitioners of determining all response quantities in a single analysis phase. The basic steps are summarized below. Detailed implementation of the procedure is presented in Appendix A.

1. Compute the elastic mode shapes, $\boldsymbol{\phi}_n$, and natural periods, T_n , of the structural system based on eigenvalue analysis.
2. Develop the base shear, V_{bn} , versus roof displacement, u_{rn} , pushover curve using the n th-“mode” lateral load distribution, s_n^* which is given by $s_n^* = \mathbf{m}\boldsymbol{\phi}_n$, in which \mathbf{m} is the mass matrix of the structural system.
3. Idealize the pushover curve as a multi-linear curve.
4. Develop the base shear V_{bn}^* versus displacement δ^* of the n th-“mode” equivalent SDOF system. This step requires the computation of the effective modal mass, $M_n^* = L_n \Gamma_n$, in which, $L_n = \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n$ and $\Gamma_n = \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n / (\boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n)$.
 - a. The vertical axis of the n th-“mode” equivalent SDOF system is created after scaling the base shear V_{bn} of the MDOF system with M_n^* to obtain $V_{bn}^* = V_{bn} / M_n^*$.
 - b. The horizontal axis of the equivalent SDOF system is created after scaling the roof displacement of the MDOF system with $\Gamma_n \boldsymbol{\phi}_{rn}$ to obtain $\delta^* = \delta^* / \Gamma_n \boldsymbol{\phi}_{rn}$, in which $\boldsymbol{\phi}_{rn}$ is the contribution of the n th-“mode” at the roof of the MDOF system.
 - c. The vibration period T_n of the n th-“mode” SDOF system is given by $T_n = 2\pi (M_n^* \delta_n^* / V_{bn})^{1/2}$.
5. Compute the displacement history $\delta_n^*(t)$ of the n th-“mode” inelastic SDOF system with mass proportional damping ζ_n equal to the value that is assigned to the n th-“mode” of the MDOF system. In this study, nonlinear response history analysis is employed to compute the maximum displacement response of the n th-“mode” equivalent SDOF systems for a set of ground motions. Using the median of absolute maximum displacements, $\delta_{n,max}^*$, of the equivalent modal SDOF system, the target roof displacement δ_{rno} is calculated for the n th mode pushover in the MDOF domain from $\delta_{rno} = \Gamma_n \boldsymbol{\phi}_{rn} \delta_{n,max}^*$.
6. Determine the total response of the MDOF system by combining the “modal” responses r_{n+g} (internal forces and deformations due to combined gravity and lateral forces) using a modal combination rule. In this study SRSS combination was used since the natural periods of the examples used are well separated. The total response r is given by, $r = \max[r_g \pm (\sum r_n^2)^{1/2}]$.

4.3.4 Consecutive Modal Pushover

The Consecutive Modal Pushover (CMP) procedure, developed by Poursha et al. (2009), considers up to three modes, applied consecutively in stages in a single pushover analysis after the application of gravity loads. Modal forces applied in each stage have an invariant load pattern. First mode forces are applied until a certain displacement is reached. The analysis then continues with incremental forces applied using a second mode distribution, and then a third mode distribution. The maximum values of response quantities in each pushover analysis are retained.

The steps of the CMP procedure, as implemented, are summarized below. Detailed implementation of the procedure is presented in Appendix B.

1. Calculate the natural frequencies, modal shapes, and lateral load patterns $s_n^* = \mathbf{m}\boldsymbol{\phi}_n$, where $\boldsymbol{\phi}_n$ is the mode shape and \mathbf{m} is the mass matrix of the system.
2. Compute the total target displacement δ_t using the ASCE/SEI 41-06 *R-C₁-T* relationship.
3. Perform a single-stage or multi-stage pushover analyses as follows:
 - a. Develop the base shear, V_{bn} , versus roof displacement, u_{rn} , pushover curve using a single stage pushover analysis until the roof displacement equals the target displacement, δ_t . An inverted triangular or first-mode lateral load pattern is used for mid-rise buildings and a uniform load pattern is used for high-rise buildings.
 - b. The second pushover analysis is a two-stage pushover analysis. In the first stage, lateral forces are proportional to the first mode, $s_1^* = \mathbf{m}\boldsymbol{\phi}_1$, until the displacement increment at the roof reaches $u_{r1} = \alpha_1\delta_t$, where α_1 is the first mode mass participation factor. The second stage is implemented with incremental lateral forces proportional to the second mode, $s_2^* = \mathbf{m}\boldsymbol{\phi}_2$, until the displacement increment at the roof equals $u_{r2} = (1 - \alpha_1)\delta_t$. The initial condition of the second stage is the condition at the last increment of the first stage.
 - c. For buildings with period $T_1 \geq 2.2s$, an additional third pushover (three-stage) analysis should be performed.
4. Calculate the peak values of the response quantities of interest resulting from the one-, two-, and three-stage pushover analyses, denoted as r_1 , r_2 , and r_3 , respectively.
5. Calculate the envelope, r , of the peak values as $r = \max\{r_1, r_2, r_3\}$.

Additional ancillary studies were also conducted using an extension of the CMP procedure by changing the order in which the modal force patterns were applied,

varying the signs used in load application, and computing both enveloped and averaged responses. Implementation of the Extended Consecutive Modal Pushover procedure, as investigated in the ancillary studies, is presented in Appendix F.

4.3.5 Load Patterns

In FEMA 440 (FEMA, 2005), the use of multiple load patterns did not provide a consistent benefit in improving agreement between nonlinear static and nonlinear response history analysis results. Based on this finding, the use of a single load pattern based on the first mode shape was recommended and ultimately incorporated into the requirements of ASCE/SEI 41-06. Unless otherwise noted, a load pattern based on the first mode shape is the default pattern used in the focused analytical studies.

4.3.6 Target Displacements

Target displacements are used to determine maximum displacements in nonlinear static analysis procedures. Results from nonlinear response history analyses are compared with results from nonlinear static analyses that have been “pushed” to target displacements estimated using one or more of the methods listed below.

- **Coefficient Method:** Defined in ASCE/SEI 41-06, the coefficient method of estimating the target displacement consists of a series of approximate inelastic displacement modifiers (i.e., coefficients) applied to the elastic spectral displacement of a system. Although other methods, such as equivalent linearization (i.e., capacity spectrum), are mentioned in ASCE/SEI 41-06, the coefficient method is the method most often used in practice.
- **Equivalent SDOF:** In this method, an equivalent single-degree-of-freedom representation of a multiple-degree-of freedom system is subjected to nonlinear response history analysis to determine peak inelastic displacements. The process is illustrated in FEMA P-440A (FEMA, 2009a). A nonlinear static analysis can be used to determine an idealized force-deformation curve for the system, which can then be used as a capacity boundary to constrain the hysteretic behavior of the idealized system. Incremental dynamic analysis (IDA) can be used to determine peak displacements and the corresponding statistics (median and dispersion) on displacement response.
- **SPO2IDA:** The open source software tool, Static Pushover 2 Incremental Dynamic Analysis, SPO2IDA (Vamvatsikos and Cornell, 2002), can be used to estimate inelastic displacements in systems with static pushover curves that are approximated with a quadrilinear backbone curve. SPO2IDA utilizes an extensive database of incremental dynamic analysis (IDA) results to develop equations that relate features of a characteristic backbone curve to nonlinear response history analysis results for a set of suitably scaled earthquake records.

- **N2/EC8 Method:** N2 is a simplified analytical method based on pushover and inelastic response spectrum approaches that has been adopted in *Eurocode 8: Design of Structures for Earthquake Resistance* (CEN, 2005). The target displacement of the MDOF system is δ_t and the displacement of the corresponding SDOF is d_t^* , where $d_t = \Gamma_n d_t^* = C_0 d_t^*$. Different expressions are used for structures in the short-period range and in the medium- and long-period ranges. Detailed implementation of the N2/EC8 target displacement procedure is presented in Appendix B.

4.4 Analytical Modeling

Analytical modeling was performed using different software platforms and various nonlinear component models and parameters.

4.4.1 Software Platforms

Analytical models were developed using the following software platforms commonly used in research and practice:

- Drain-2DX, *Static and Dynamic Analysis of Inelastic Plane Structures* (Prakash et al., 1993)
- OpenSees, *Open System for Earthquake Engineering Simulation* (McKenna, 1997)
- Perform 3D, *Nonlinear Analysis and Performance Assessment for 3D Structures* (Computers and Structures, Incorporated)
- SAP2000, *Integrated Software for Structural Analysis and Design* (Computers and Structures, Incorporated)
- IILDAP, *Interactive Interface for Incremental Dynamic Analysis* (Lignos, 2009)

4.4.2 Component Model Types and Parameters

Analytical models were developed using the following nonlinear component model types and parameters:

- **ASCE41:** Component models designated ASCE41 are based on the generic ASCE/SEI 41-06 backbone curves assuming a post-capping stiffness obtained by linearly connecting peak point C and point E. This modification, illustrated in Figure 4-1, is made in order to avoid numerical instability problems in the analysis and to better conform to component test data developed over the last decade. This model is used only for pushover analyses.

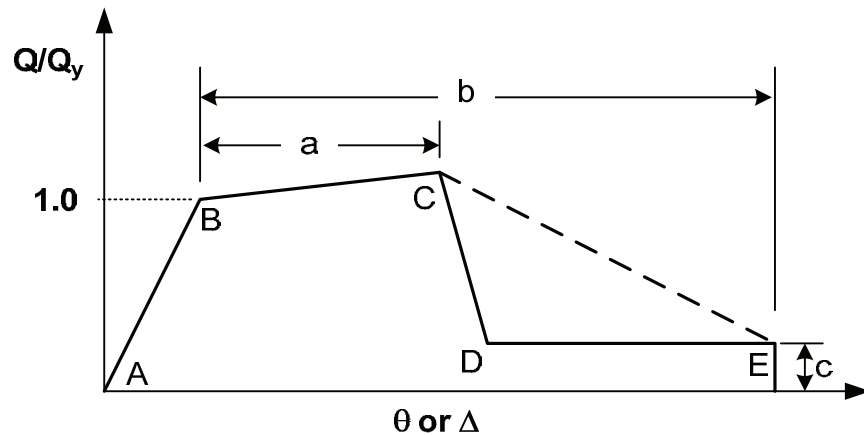


Figure 4-1 ASCE 41 component model, adapted from ASCE/SEI 41-06 (PEER/ATC, 2010).

- Analyt.M1:** Component models designated Analyt.M1 utilize a modified Ibarra-Krawinkler (modified IK) backbone curve of the type shown in Figure 4-2, a bilinear hysteretic model, and cyclic deterioration parameters per analysis Option 1 in PEER/ATC-72-1 (PEER/ATC, 2010). Nonlinear response history analyses utilizing Analyt.M1 component models account for cyclic deterioration effects explicitly. Nonlinear static analysis utilizing Analyt.M1 component models do not account for cyclic deterioration because they are based on the initial (monotonic) backbone curve.

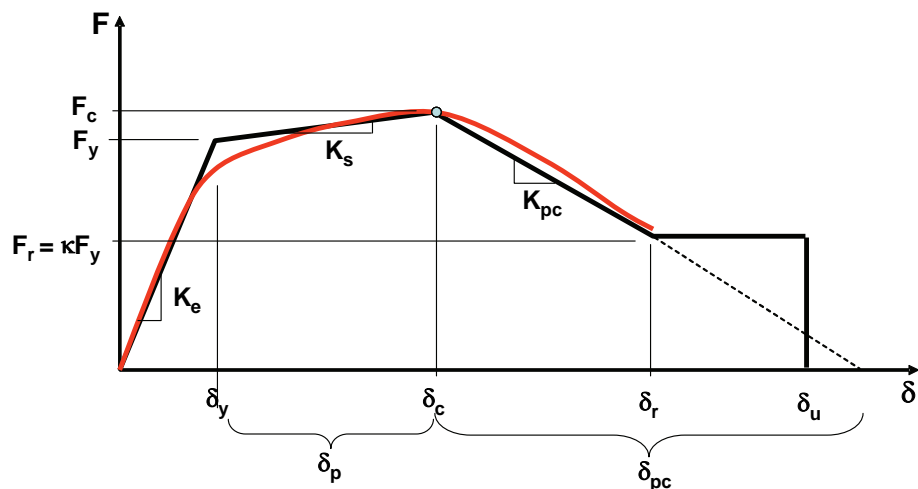


Figure 4-2 Backbone curve for component model Analyt.M1 (PEER/ATC, 2010).

- Analyt.M3:** Component models designated Analyt.M3 utilize a backbone curve of the type shown in Figure 4-2, which is then modified to account for cyclic deterioration effects in an approximate manner per analysis Option 3 in PEER/ATC-72-1. Nonlinear response history analyses utilizing Analyt.M3 component models account for cyclic deterioration only indirectly. Nonlinear

static analysis utilizing Analyt.M3 component models account for cyclic deterioration as a result of the modified backbone curve.

- **Fiber Model (FM):** Used for shear wall elements, fiber component models utilized displacement based beam-column elements together with translational shear springs. Nonlinear degrading properties for concrete and steel materials, including concrete crushing and steel buckling and fracture, are explicitly considered.
- **Simplified Spring Model (SM):** Also used for shear wall elements, simplified plastic hinge spring models were used to assist in evaluating results obtained from fiber models. Use of simplified spring models of the type shown in Figure 4-3 permits explicit modeling of post-yield and post-capping behavior using modeling parameters similar to those used for Analyt.M1 and Analyt.M3.

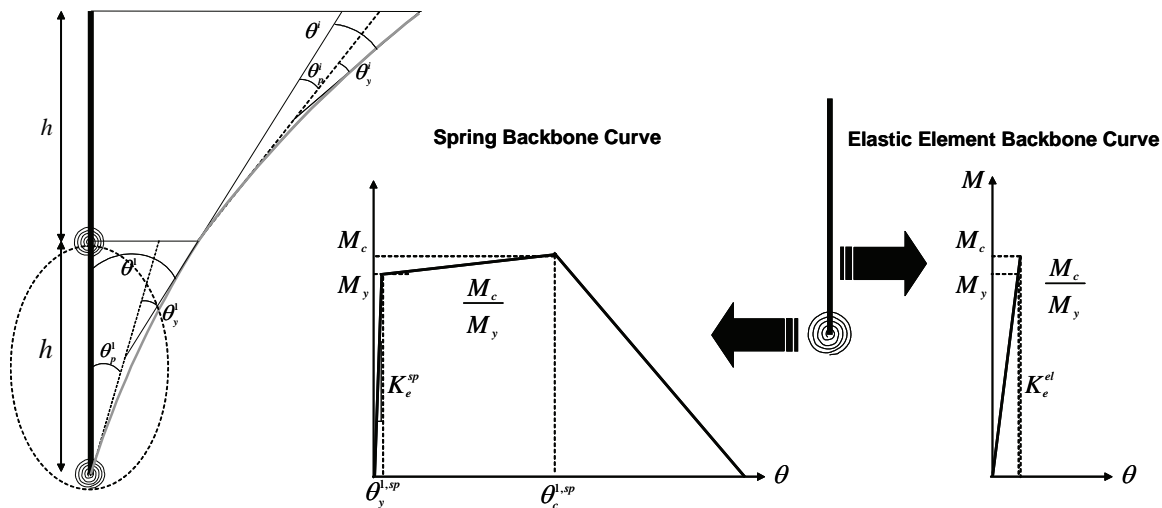


Figure 4-3 Modeling of flexural behavior with simplified plastic hinge springs and elastic elements (Zareian and Krawinkler, 2009).

4.5 Response Quantities of Interest

The response quantities of interest for engineering design (i.e., demand parameters) are listed below. Story-level demand parameters were emphasized, but since design codes and standards generally focus attention on component-level force and deformation demands, focused studies examined the correlation between component-level demand parameters and story-level demand parameters. In each case comparisons are made between peak values, rather than time-varying values, of the parameters. Primary demand parameters addressed in all of the studies included the following:

- Story drift ratios
- Story shear forces

- Floor overturning moments

Some of the studies addressed the following additional demand parameters:

- Residual story drift ratios
- Floor absolute accelerations

4.6 Ground Motions

Nonlinear response history analyses have been performed using the suite of 44 ground motion records of the far-field record set utilized in FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009b). Ground motion records are normalized on the basis of peak ground velocity, as discussed in FEMA P-695, and scaled by scale factors (SF) equal to 0.5, 1 and 2. These scale factors correspond to ground motions at a Los Angeles, California site with mean recurrence intervals of approximately 100, 400, and 2475 years, respectively. Figure 4-4 shows the individual spectra and statistical spectra overlaid onto a smooth design spectrum for a mean recurrence interval of 400 years (SF = 1).

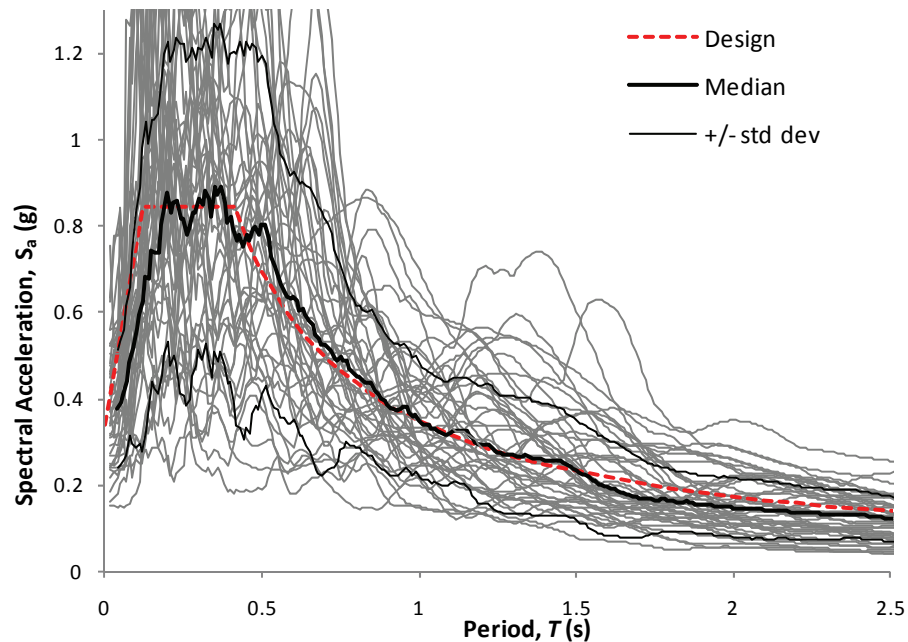


Figure 4-4 Response spectra for the FEMA P-695 far-field record set.

Codes, standards, and guidelines governing the application of nonlinear response history analysis prescribe, for example, the use of maximum of three or average of seven ground motions, as single components or in pairs. FEMA P-750, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Part 2: Commentary (FEMA, 2009d) states that while such ground motion suites might “provide reasonable estimates of mean response for the individual

response parameters,” they certainly are not “adequate to give an accurate estimate of the variability.” Because focused studies aim to examine the relative accuracy of different analysis methods in the context of realistic variations in response, the blind application of such codified rules was considered inadequate, and alternatives were studied.

4.6.1 Intensity Measures and Scaling

The intensity measure selected as a basis for determining the strength of ground motions used in nonlinear response history analysis affects the dispersion in computed response quantities. The following scaling methods were investigated in focused analytical studies and supporting ancillary studies:

- **FEMA P-695:** 22 pairs of orthogonal horizontal records were scaled to a common intensity that retains a degree of natural dispersion. This is functionally equivalent to using the geometric mean of the peak ground velocity of the two horizontal components of the ground motion, averaged over different orientations, as the intensity measure.
- $S_a(T_1)$: The records were scaled to the pseudo-spectral acceleration at the first mode period of the structure.
- $S_a(T_a, T_b)$: The records were scaled with a vector-valued approach where $S_a(T_a, T_b)$ is taken equal to the geometric mean of two elastic spectral values, $S_a(T_a)^{0.5} S_a(T_b)^{0.5}$ where T_a = the first mode period of the structure and T_b = the second mode period of the structure.
- $S_{di}(T_1)$: The records were scaled to the peak displacement of an equivalent SDOF oscillator.

4.6.2 Ground Motion Subsets

In order to investigate possible reductions in the number of nonlinear response history analyses necessary for reliable results, the following techniques for selecting ground motion subsets were attempted:

- **Spectra-Based Subset Selection on FEMA P-695 Normalized Ground Motions:** Using the FEMA P-695 record set for each of the given scale factors (SF = 0.5, 1, and 2) progressively larger subsets (of size 1, 3, 5, ..., 17) are selected in an attempt to more economically estimate the median (or mean) of the entire set. By design, results using these smaller subsets will exhibit less dispersion.
- **Spectra-Based Subset Selection on S_{di} -Normalized Ground Motions:** A method found to produce small maximum errors for subsets is tested extensively on the basis of S_{di} -scaled ground motions. In this case, a search is made for

“optimal” record subsets for a given level of S_{di} that allows accurate estimation of the 50th or 84th percentile demand parameters for the entire set of 44 records.

- **Random Subset Selection on S_{di} -Normalized Ground Motions:** To further test the potential of subset selection based on S_{di} scaling, a number of trials with random selection of record subsets was performed.

4.7 System prototypes

Structural system prototypes were used to test the analytical procedures, model types, demand parameters, and ground motions outlined above. Prototypes for regular structural systems and systems with special characteristics were developed. The following regular structural system prototypes were investigated:

- Steel Moment Frame
- Reinforced Concrete Moment Frame
- Reinforced Concrete Shear Wall

Individual prototypes were selected based on several criteria: (1) structures types that are commonly encountered in practice; (2) structures for which nonlinear static procedures are expected to produce accurate results and some for which they are not; and (3) structures from previous studies funded by FEMA and NIST to reduce modeling effort. Additional prototypes with special structural characteristics were also investigated. Specific design and modeling information for individual structural system prototypes are provided in Chapter 5, Chapter 6, and the Appendices.

4.7.1 Steel Moment Frame Prototypes

Steel moment frame (SMF) prototypes consist of a subset of the steel special moment frame archetypes designed and analyzed in the NIST GCR 10-917-8 report, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors* (NIST, 2010b).

The subset consists of 2-, 4-, and 8-story structures of three-bay moment-resisting frames with reduced-beam section (RBS) connections designed in accordance with AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2005a).

4.7.2 Reinforced Concrete Moment Frame Prototypes

Reinforced concrete moment frame (RCMF) prototypes consist of a subset of the reinforced concrete moment frame archetypes designed and analyzed in the FEMA P-695 report, *Quantification of Building Seismic Performance Factors* (FEMA, 2009b).

The subset consists of 2-, 4-, and 8-story structures of three-bay space frames designed as special reinforced concrete moment frames in accordance with the 2003 *International Building Code* (ICC, 2003).

4.7.3 Reinforced Concrete Shear Wall Prototypes

Reinforced concrete shear wall (RCSW) prototypes consist of a subset of the reinforced concrete special shear wall archetypes designed and analyzed in the NIST GCR 10-917-8 report, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors* (NIST, 2010b).

The subset consists of 2-, 4-, and 8-story structures of cantilever walls designed as special shear walls within building frame systems ($R = 6$) and detailed in accordance with ACI 318-08, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2008).

4.7.4 Systems with Special Characteristics

Studies of 2-, 4-, and 8-story steel moment frames, reinforced concrete moment frames, and reinforced concrete shear walls described above were supplemented with analyses of other systems to expand the coverage provided by the basic systems that were selected and to further investigate specific issues. The following additional structural systems were studied:

- “Equivalent” simplified models calibrated to the detailed models described above, including a single-bay steel moment frame (Appendix A) and 4-story reinforced concrete shear wall buildings with simplified springs (Appendix C)
- 4-story steel moment frame with uniformly increasing stiffness to reduce structure period (Appendix A)
- 4-story steel moment frame with gravity system included in the model (Appendix A)
- 2-story steel single-bay “shear building” with load-pattern-sensitive story mechanisms (Appendix A)
- 4-story irregular (weak-story) steel single-bay “shear building” (Appendix A)
- 4-story reinforced concrete shear wall designed to fail in bending (Appendix C)
- 6-story steel moment frame (Appendix F)
- 3-story steel concentrically braced frame (SCBF) with a flexible roof diaphragm, modeled in 3-D (Appendix F)

4.7.5 Summary of Structural System Prototypes and Modeling Decisions

Structural system prototypes, nonlinear static analysis methods, and software platforms utilized in focused analytical studies and supporting ancillary studies are summarized in Table 4-1. Structural system prototypes along with component modeling decisions and consideration of damping are summarized in Table 4-2.

Table 4-1 Structural System Prototypes, Nonlinear Static Analysis Methods, and Software Platforms Used in Focused Analytical Studies

System prototype	Nonlinear Static Analysis Methods				Software Platforms			
	NSP	MPA	CMP	Extended CMP	OpenSees	Drain-2DX	SAP 2000	Perform 3D
2-story SMF	x				1-bay	3-bay		
4-story SMF	x	x			1-bay	3-bay		
8-story SMF	x	x			1-bay	3-bay		
2-story RCMF	x	x	x		x			
4-story RCMF	x	x	x		x			
8-story RCMF	x	x	x		x			
2-story RCSW	x				x			
4-story RCSW	x	x			x			
8-story RCSW	x	x			x			
6-story SMF	x	x	x	x			x	
3-story SCBF	x	x	x	x				x

Table 4-2 Structural System Prototypes, Component Models, and Damping Models Used in Focused Analytical Studies

System prototype	Component models		Damping models
2-story SMF	Beam:	lumped plasticity flexure	Rayleigh (proportional to mass and initial stiffness) 2.5% at T_1 and $0.2 T_1$
4-story SMF	Column:	lumped plasticity flexure (no axial load dependence)	
8-story SMF	Panel zone:	trilinear shear (nondeteriorating)	
2-story RCMF	Beam:	lumped plasticity flexure	Rayleigh (proportional to mass and initial stiffness) 5% at T_1 and T_3
4-story RCMF	Column:	lumped plasticity flexure	
8-story RCMF	Joints:	shear distortion and bond slip	
2-story RCSW	Flexure:	fibers	Rayleigh (proportional to mass and initial stiffness) 2.5% at T_1 and $0.2 T_1$
4-story RCSW	Shear:	translational spring	
8-story RCSW	4-story:	separate models were used with simplified rotational flexure springs and translational shear springs capped at $1.5 V_n$ with pinched hysteresis)	
6-story SMF	Beam:	lumped plasticity flexure	Rayleigh (proportional to mass and initial stiffness) 5% at T_1 and T_2
	Column:	lumped plasticity flexure (no axial load dependence)	
3-story SCBF	Brace:	asymmetric axial (with compression buckling, tension yielding, and cyclic pinching)	5% thus: 2% Rayleigh (proportional to mass and initial stiffness) and 3% modal
	Beam:	lumped plasticity flexure with cyclic degradation	
	Grade beam:	lumped plasticity flexure with cyclic degradation	
	Column:	lumped plasticity flexure with cyclic degradation (and axial load dependence)	
	Foundation:	(undamped) uplift elements to allow rocking	
	Roof deck:	shear with strain hardening and cyclic degradation	

Results of Problem-Focused Analytical Studies

This chapter summarizes the results of problem-focused studies outlined in Chapter 4 and presented in detail in Appendices A, B, and C. Findings are presented in terms of the degree of agreement between nonlinear static (pushover) and modal response spectrum analysis procedures with more rigorous nonlinear response history analyses.

Using the term “accuracy” implies that the truth is known. Clearly this is not the case, although all mathematical models, whether analyzed under static load application or under ground motion input, have inherent uncertainty. Nevertheless, median results obtained from nonlinear response history analyses are used as benchmark values against which the accuracy of all simplified methods is measured. In this context the term “accuracy” is relative and should not be construed as an absolute measure of closeness to truth.

Evaluation of the accuracy of nonlinear static analysis methods relative to nonlinear response history analysis must be conditioned on the details of the structural system and the assumptions made in the analytical model. Unless otherwise noted in the appendices, the geometry and component section properties are the same for all analysis methods explored in order to minimize the effects of component modeling in contributing to differences in response predictions.

Although the graphic and tabular summaries provided in this chapter are necessary and helpful for understanding trends in the results, such summaries can be misleading. Most static approaches for representing a dynamic phenomenon are case specific and may not be representative of other common cases. Furthermore, a focus on the worst predictions at any story may obscure considerably better predictions at other stories.

Caution should be taken in drawing conclusions about these findings. Even when reasonable accuracy for nonlinear static procedures is reported, blind reliance on static approaches is unwise; on the other hand, when static methods do not provide sufficient accuracy for making final design decisions, they can provide valuable insight into system behavior.

5.1 Summary Results for Regular Structural System Prototypes

Table 5-1 summarizes ratios of response quantity estimates to median results from nonlinear response history analysis (NRHA) for each simplified analysis procedure,

for each demand parameter, and for each regular structural system prototype. This table aggregates data presented in Table 5-2 through Table 5-10 over all scale factors. It permits head-to-head comparison of the methods employed and systems investigated, but obscures potentially meaningful differences in response over height and at various levels of intensity, which can be observed in figures comparing peak response quantities over height.

Table 5-1 Summary Ratios of Response Quantity Estimates to Median NRHA Results

Structural System Prototype	Response Quantity Ratios ¹			
	Displacement	Story Drift	Story Shear	Floor Overturning Moment
Single-Mode Nonlinear Static Analysis				
2-story SMF		0.9 to 1.1	0.8 to 1.0	0.8 to 1.0
4-story SMF		0.6 to 1.3	0.3 to 0.9	0.3 to 1.0
8-story SMF		0.4 to 1.6	0.3 to 1.0	0.3 to 1.1
2-story RCMF	0.9 to 1.1	0.9 to 1.1	0.8 to 1.0	0.8 to 1.0
4-story RCMF	1.0 to 1.2	0.8 to 1.2	0.6 to 1.0	0.6 to 1.0
8-story RCMF	0.9 to 1.5	0.4 to 1.4	0.4 to 1.0	0.4 to 0.9
2-story RCSW		1.2 to 1.6	0.9 to 1.2	0.9 to 1.3
4-story RCMF		1.1 to 1.8	0.8 to 1.3	0.8 to 1.4
4-story RCSW (δ_i – ESDOF)		0.9 to 1.1	0.8 to 1.0	0.8 to 1.1
8-story RCMF		1.3 to 2.5	0.7 to 1.3	0.7 to 1.4
Multi-Mode Nonlinear Static Analysis (Modal Pushover Analysis)				
4-story SMF		0.7 to 1.1	0.8 to 1.0	0.8 to 1.0
8-story SMF		0.7 to 1.9	0.7 to 1.2	0.8 to 1.2
2-story RCMF	0.9 to 1.1	0.9 to 1.1	0.8 to 1.0	0.8 to 1.0
4-story RCMF	1.0 to 1.2	0.8 to 1.2	0.8 to 1.0	0.8 to 1.0
8-story RCMF	1.0 to 1.5	0.7 to 1.5	0.8 to 1.1	0.8 to 1.2
4-story RCSW		1.0 to 1.6	1.0 to 1.3	1.0 to 1.8
8-story RCSW		1.0 to 1.4	0.8 to 1.2	0.9 to 1.2
Multi-Mode Nonlinear Static Analysis (Consecutive Modal Pushover)				
2-story RCMF	0.9 to 1.1	0.9 to 1.1	0.8 to 1.2	0.8 to 1.2
4-story RCMF	1.0 to 1.2	1.0 to 1.4	0.7 to 1.4	0.7 to 1.4
8-story RCMF	0.9 to 1.5	0.4 to 1.4	0.4 to 1.0	0.4 to 0.9
Elastic Modal Response Spectrum Analysis				
4-story SMF		0.8 to 1.2	0.8 to 2.0	0.8 to 2.4
2-story RCMF	0.9 to 1.1	0.9 to 1.1	0.9 to 2.7	0.9 to 2.7
4-story RCMF	1.0 to 1.1	0.9 to 1.2	0.8 to 2.5	0.8 to 2.5
8-story RCMF	1.0 to 1.3	0.8 to 1.6	0.8 to 2.6	0.8 to 2.5

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

For the regular structural system prototypes investigated, the following general observations were made:

- For the 2-story systems considered, response quantities from single-mode nonlinear static analysis generally agree well with median results from nonlinear response history analysis. Results for systems with special characteristics (Section 5.5), however, show that this observation may not always hold true.
- The accuracy of response quantities from single-mode nonlinear static analysis decreases with increasing height. Response may be underestimated substantially for 4-story steel moment frames (SMF) and 8-story reinforced concrete moment frames (RCMF). Underestimation of response occurs mostly in the upper stories and varies significantly between story drift, story shear, and floor overturning moment response parameters.
- In general, estimates of demand parameters for reinforced concrete shear wall (RCSW) structures were better than those for moment frame structures of the same height, provided the target displacement estimate is reasonably accurate. The ASCE/SEI 41-06 coefficient method, which is based on an effective elastic stiffness, often will not result in a good estimate of the target displacement if there are clear differences between the pre- and post-cracking stiffness of the shear wall elements.
- Use of multi-mode nonlinear static analysis generally improves the prediction of demand parameters for regular steel and reinforced concrete moment frame systems. In the case of 8-story structures, however, improved response prediction was still substantially underestimated.
- Use of multi-mode nonlinear static analysis may improve or worsen the prediction of demand parameters for reinforced concrete shear walls, but in most such cases, the response is overestimated.
- As expected, elastic modal response spectrum analysis consistently overestimates story shears and overturning moments for yielding systems. However, the estimates of floor displacement and story drift were consistently as good as, or better than, those from any nonlinear static analysis method.

When judging the adequacy of analytical methods for design decisions, the relevance of the response quantity to design should be considered. For instance, column axial forces due to earthquake loading, which are related to floor overturning moments, may not be critical design considerations in the upper stories of a moment frame system, but are likely to be critical in the lower stories of the system.

Also, differences in analytically predicted response quantities can indicate changes in performance, but the significance of such differences must be considered in light of the acceptance criteria that define performance. Performance predictions may not be equally sensitive to variations in response quantity estimation. In some cases (e.g.,

non-ductile components) small changes in a response quantity can lead to large differences in expected performance, but in other cases (e.g., ductile components) performance can be relatively insensitive to changes in a response quantity.

5.2 Steel Moment Frame Studies

This section summarizes results of focused analytical studies on 2-, 4-, and 8-story steel moment frame prototypes. Nonlinear response history analysis was performed using the FEMA P-695 far-field ground motion set, and various options of single-mode nonlinear static analysis and modal pushover analysis procedures were explored and evaluated for their ability to predict peak values of story drift ratio, story shear force, and floor overturning moment. Detailed results are presented in Appendix A.

5.2.1 Steel Moment Frame Prototypes

Steel moment frame prototypes consist of a subset of the steel special moment frame (SMF) archetypes designed and analyzed in the NIST GCR 10-917-8 report, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors* (NIST, 2010b). The subset consists of three structures designed by means of response spectrum analysis (RSA) for seismic design category D_{max} ($S_{DS} = 1.0g$ and $S_{DI} = 0.60g$), designated as follows:

- 2-story steel SMF (archetype ID 2- D_{max} -RSA)
- 4-story steel SMF (archetype ID 4- D_{max} -RSA)
- 8-story steel SMF (archetype ID 8- D_{max} -RSA)

The structures consist of three-bay perimeter moment-resisting frames and a steel gravity framing system with the plan configuration shown in Figure 5-1. As designed, the moment frames resist all seismic forces and receive tributary gravity loads as indicated in the shaded portion of the figure.

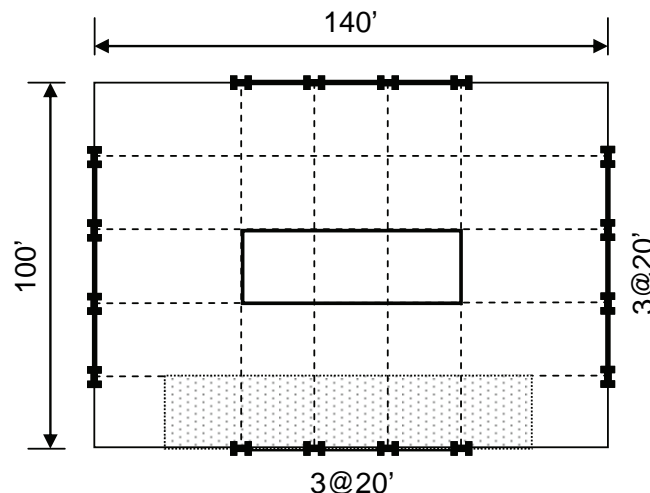


Figure 5-1 Plan view of steel moment frame prototypes (NIST, 2010).

The moment-resisting frames consist of modern code-conforming elements, and all beams are reduced-beam section (RBS), designed in accordance with AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2005a).

Analytical models are based on the moment-resisting elements of the bare steel frame, and no contribution to lateral strength and stiffness is provided by the steel gravity framing. A leaning column is used to model P-Delta effects from gravity loads not directly tributary to the moment-resisting elements. Nonlinear component models include the ASCE41, Analyt.M1 and Analyt.M3 options.

For the purpose of assessing global and story level response quantities (e.g., story drift), additional comparative studies were conducted using simplified models, as shown in Figure 5-2. In these models, the properties of a three-bay frame are lumped into a single-bay frame to reduce computational effort and facilitate interpretation of global results.

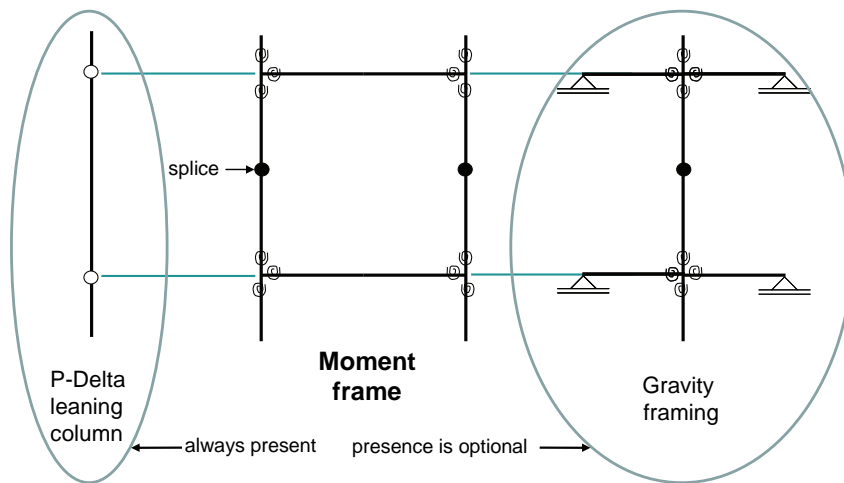


Figure 5-2 Simplified model of steel moment frame prototypes.

Detailed information on the steel moment frame prototype design parameters and modeling assumptions are provided in Appendix A.

5.2.2 Nonlinear Response History Analysis

Nonlinear response history analysis of steel moment frame structures led to the following observations regarding prediction of response quantities:

1. The distribution of peak story drift becomes more non-uniform over height for taller structures with higher degrees of nonlinearity (Figure 5-3). Reasons for this include concentration of inelastic deformations in specific regions of the structure due to P-delta effects.
2. The dispersion in response is largest for story drift ratios, smaller for story shears, and smallest for floor overturning moments (Figure 5-4). The dispersion in story shears and floor overturning moment demands is much smaller in the

inelastic range than in the elastic range because of saturation of story shear demands due to yielding in beam (or column) plastic hinge regions.

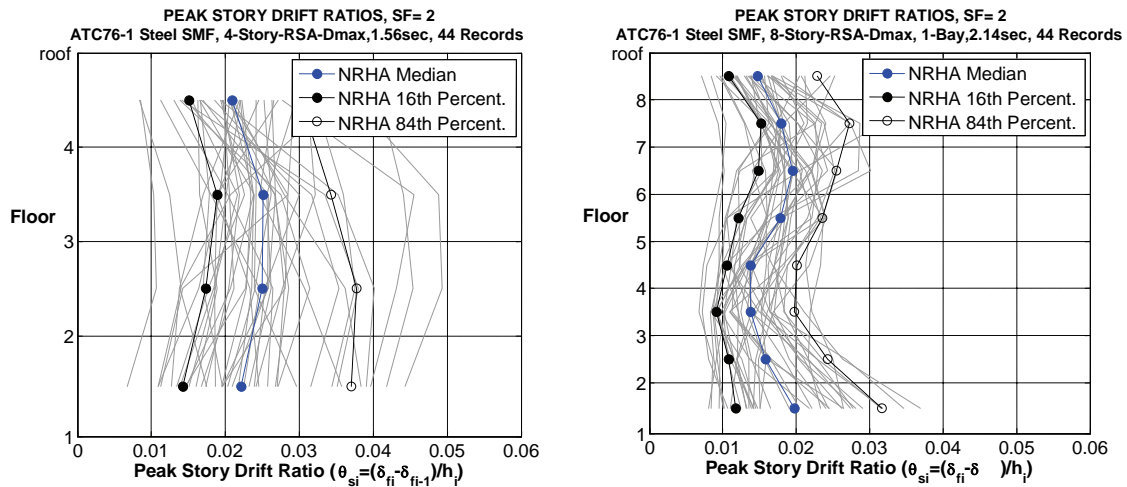


Figure 5-3 Distribution of peak story drifts over height in the 4-story steel moment frame (left) and 8-story steel moment frame (right).

3. The dispersion in story drift ratios is similar to the dispersion in spectral acceleration at the first mode period of the structure. When the dispersion in S_a at T_1 is reduced by selecting record subsets that provide a better match to the median spectra, the dispersion in story drift is correspondingly reduced (Figure 5-4). This conclusion is specific to the FEMA P-695 set of 44 ground motion records and does not hold true when the structure is pushed into the post-capping range of response (negative tangent stiffness region of global pushover curve), in which case the median from the subset can be much smaller than the median from the full set of records.
4. The distribution of story shear demands is not consistent with the shear distribution based on the lateral load pattern for which the structure was designed or the first mode load pattern in a pushover analysis (Figure 5-4). Higher mode effects and inelastic dynamic redistribution can cause large shear force amplifications in specific stories, resulting in a shear force distribution that is much more uniform over the height than would be anticipated. The base shear from nonlinear response history analysis can be considerably larger than the value obtained from a pushover analysis.
5. The distribution of peak floor overturning moments over height can be close to linear, implying close to constant story shear over height (Figure 5-4). It might also be convex, rather than concave, as would be expected from the distribution of design story shear forces or a first mode pushover load pattern.
6. The distribution of component-level demand parameters (e.g., beam plastic hinge rotation, column shear force, and column axial force) correlates well with story-level demand parameters (e.g., story drift, story shear, and overturning moment), as shown in Figure 5-5.

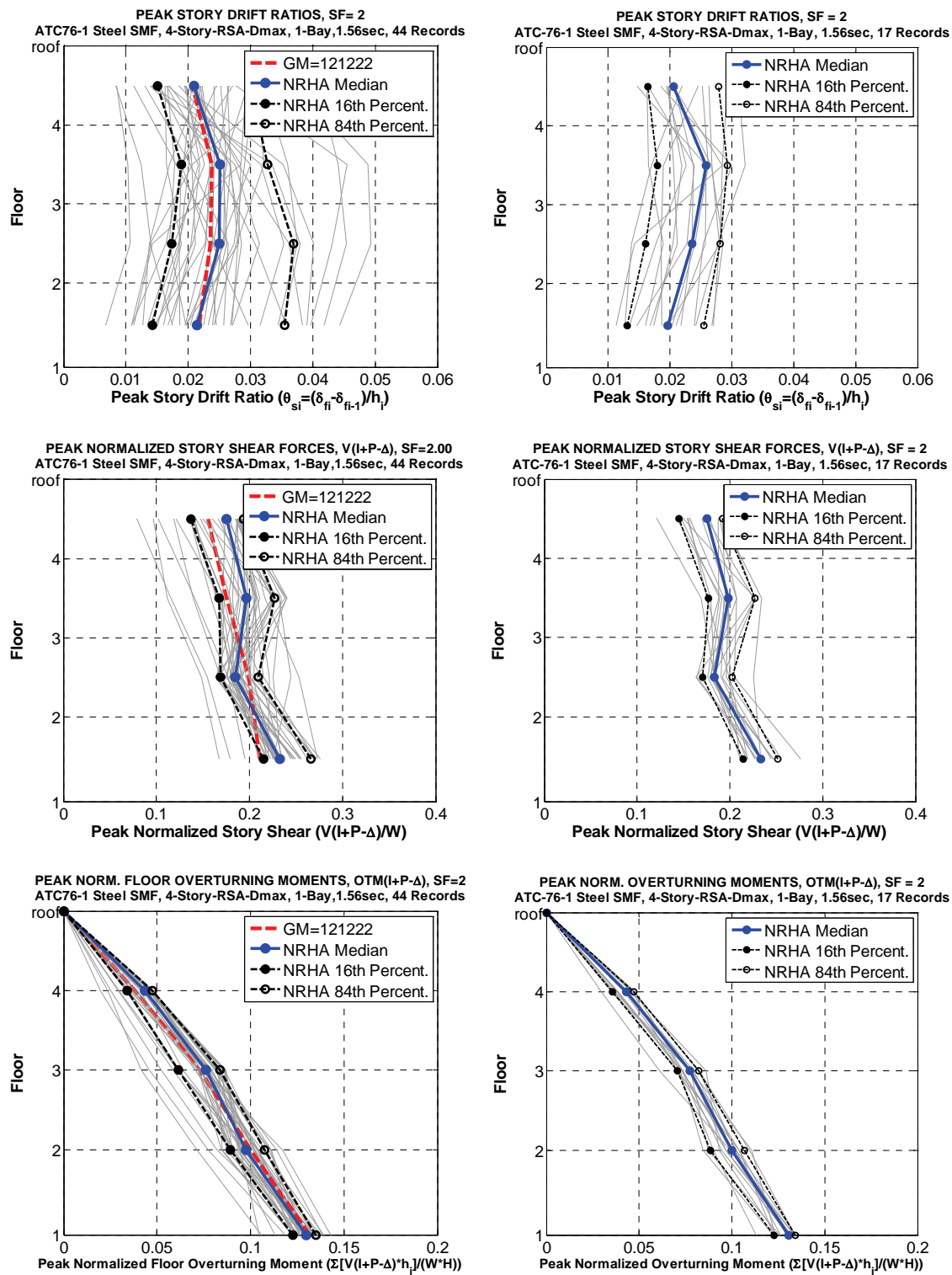


Figure 5-4 Dispersion in story-level demand parameters over height in the 4-story steel moment frame subjected to different suites of ground motions: full set of 44 records (left); subset of 17 records (right).

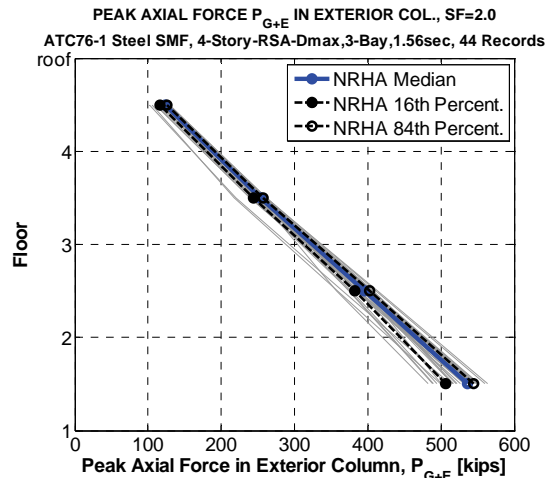
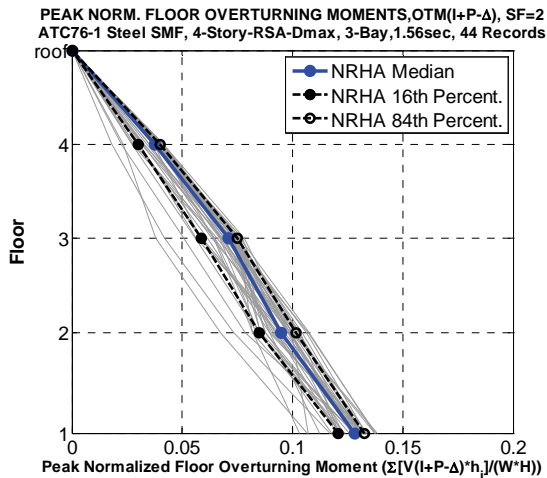
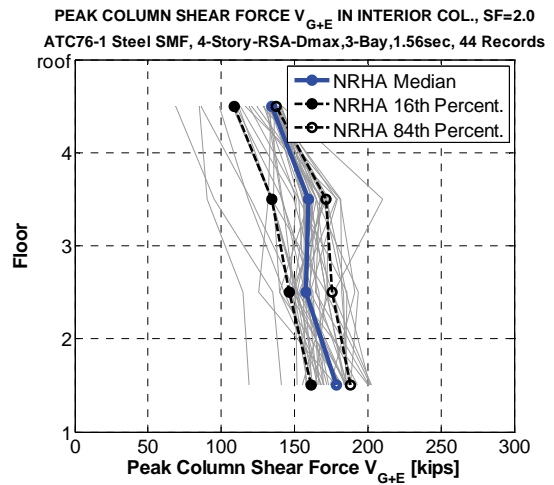
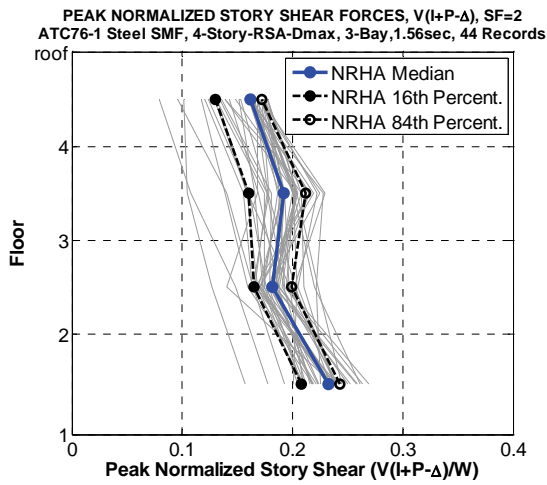
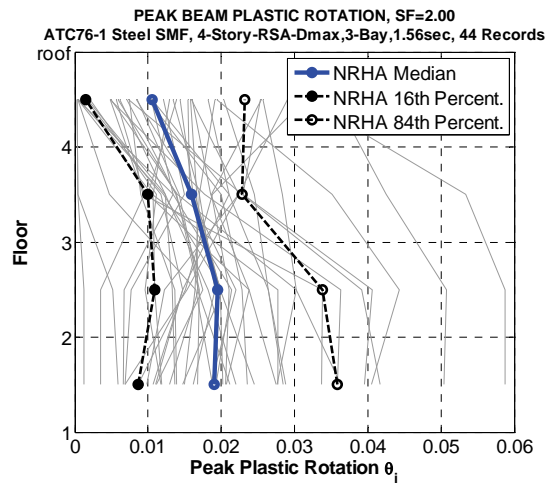
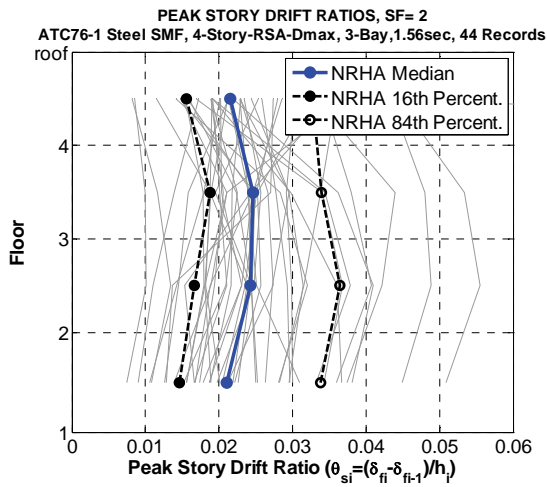


Figure 5-5 Comparison between the distribution of story-level demand parameters (left) and local demand parameters (right) in the 4-story steel moment frame.

Response quantities such as residual story drift and peak absolute floor acceleration (Figure 5-6) can be predicted with nonlinear response history analysis, but cannot be estimated using nonlinear static analysis. The median residual drift ratio depends strongly on the degree of nonlinearity, and the dispersion is much larger than the dispersion for peak story drift ratios. Peak absolute floor acceleration profiles also depend on the degree of nonlinearity. In the figure, the distribution over height is almost uniform because the structure response is highly nonlinear.

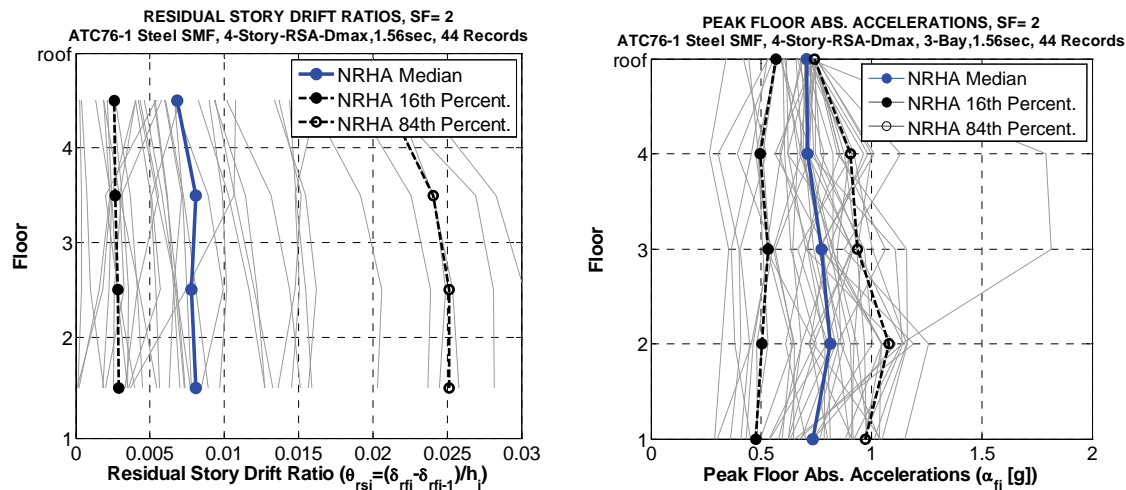


Figure 5-6 Residual story drift ratios (left) and peak floor accelerations (right) in the 4-story steel moment frame.

5.2.3 Single-Mode Nonlinear Static Analysis

Variations in single-mode nonlinear static analysis included the use of different component models (ASCE41, Analyt.M1, and Analyt.M3) and methods of predicting the target displacement. Target displacements were estimated using the coefficient method (ASCE41) and the equivalent single-degree-of-freedom method (EqSDOF). Collapse potential (i.e., lateral dynamic instability) was estimated using the ASCE/SEI 41-06 equation for R_{max} and the FEMA P-440A equation for R_{di} . In all cases, an invariant first mode load pattern was used to determine the pushover response.

A comparison between global pushover curves for the 4-story steel moment frame using ASCE41 and Analyt.M1 component models is shown in Figure 5-7. The ASCE41 pushover curves exhibit a slightly more negative post-yield stiffness and less deformation capacity than the Analyt.M1 pushover curves, indicating that ASCE41 yields a more conservative component model. The ASCE41 component model parameters are conservative estimates based on the state of knowledge in the early 1990s, while AnalytM1 component model parameters are best estimates based on regression analysis of recently collected comprehensive experimental data. These differences are important only when the target displacement is in the negative tangent stiffness region of the curves.

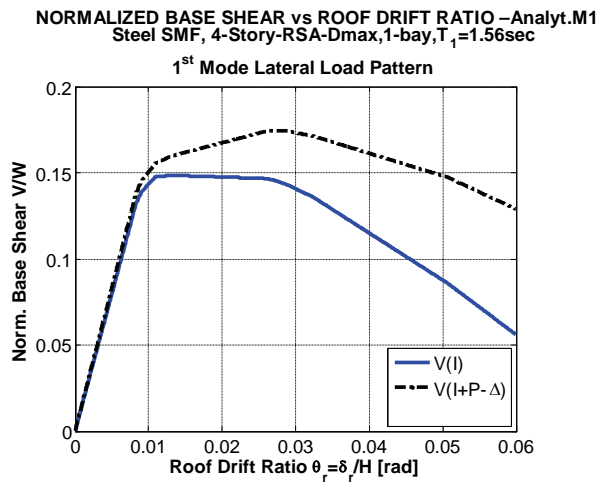
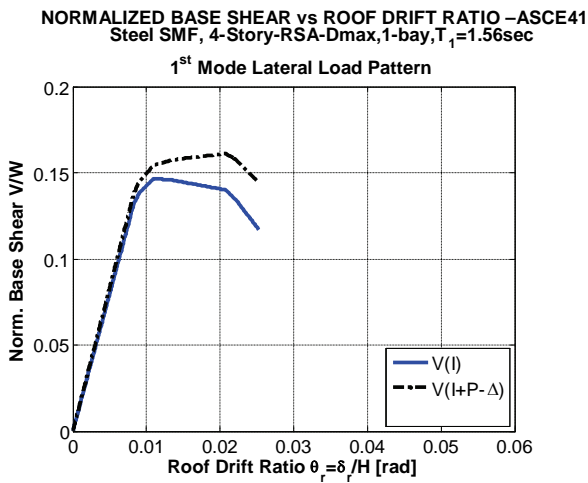


Figure 5-7 Comparison of global pushover curves for the 4-story steel moment frame, using ASCE41 (left) and Analyt.M1 (right) component models.

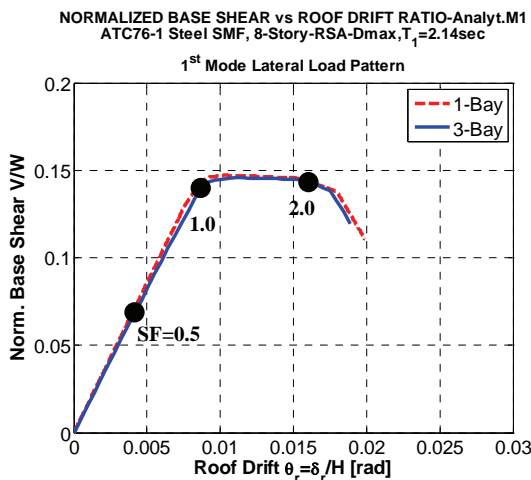
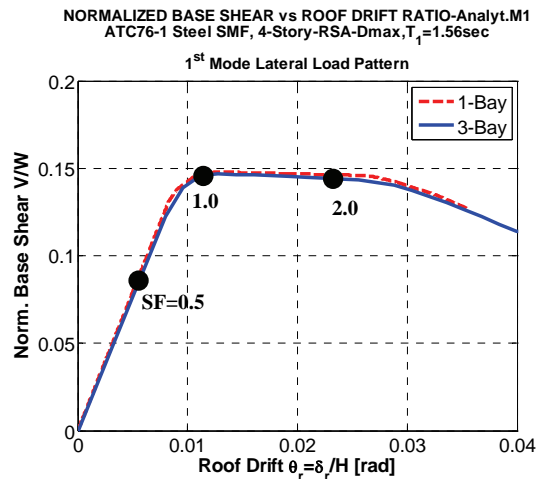
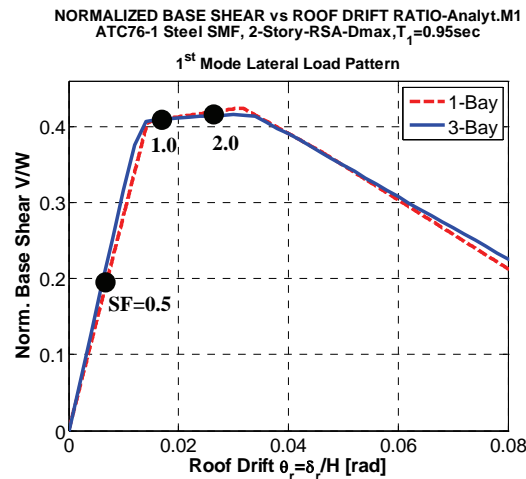


Figure 5-8 Comparison of global pushover curves for 1- and 3-bay models of 2-, 4-, and 8-story steel moment frames showing target displacements for SF= 0.5, 1 and 2.

A comparison between global pushover curves for 2-, 4-, and 8-story steel moment frames using the Analyt.M1 component model is shown in Figure 5-8. The shape of

the curves is essentially trilinear, consisting of a large elastic range, a range of nearly constant post-yield stiffness (which depends on the importance of P-delta effects), and a post-capping stiffness that leads to relatively rapid deterioration in strength (negative tangent stiffness). The slope of the post-capping stiffness is determined by a combination of P-delta effects and strength and stiffness deterioration in individual structural components. The taller the structure, the smaller the roof drift at which the tangent stiffness becomes clearly negative.

Selected comparisons between nonlinear response history analysis results (median and dispersion) and results from various single-mode nonlinear static analysis options for 2-, 4-, and 8-story steel moment frames are shown in Figures 5-9, 5-10, and 5-11. The legend in each figure identifies the component model and target displacement combination (e.g., ASCE41-EqSDOF) that was used in the nonlinear static analysis model that generated the curve. More complete results are presented in Appendix A.

General Observations – Single-Mode Nonlinear Static Analysis

Comparisons between single-mode nonlinear static analysis and nonlinear response history analysis results for steel moment frame structures led to the observations listed below. Any conclusions from these observations should be qualified by the characteristics of the regular steel moment frame prototypes in the study, which include: (a) clear elastic stiffness maintained until story or global yielding occurs, followed by a rapid decrease in stiffness; and (b) relatively long first mode periods of 0.95 sec, 1.56 sec, and 2.14 sec, respectively, for the 2-, 4-, and 8-story structures.

Long first mode periods are important because it keeps first mode response out of the short period range and produces inelastic response that is generally consistent with the equal displacement approximation for nonlinear single-degree-of-freedom systems. Long first mode periods for the low-rise steel moment frames, however, cause spectral accelerations at higher mode periods that are relatively large in comparison with the first mode, which can amplify higher mode effects. This could explain why demands predicted from a single-mode nonlinear static analysis are inferior in many cases to those obtained for shorter period systems, such as reinforced concrete moment frames (Section 5.3).

1. The accuracy of nonlinear static analysis response predictions depends strongly on the lateral load pattern used in the analysis. Utilization of the elastic first mode load pattern locks in the relative magnitude of story shears in the individual stories and does not permit consideration of dynamic redistribution.
2. In structures 4 or more stories in height, single-mode nonlinear static analysis predictions underestimate story demand parameters. This is particularly true for story shears and overturning moments, and much more so for demands in upper stories than in lower stories.

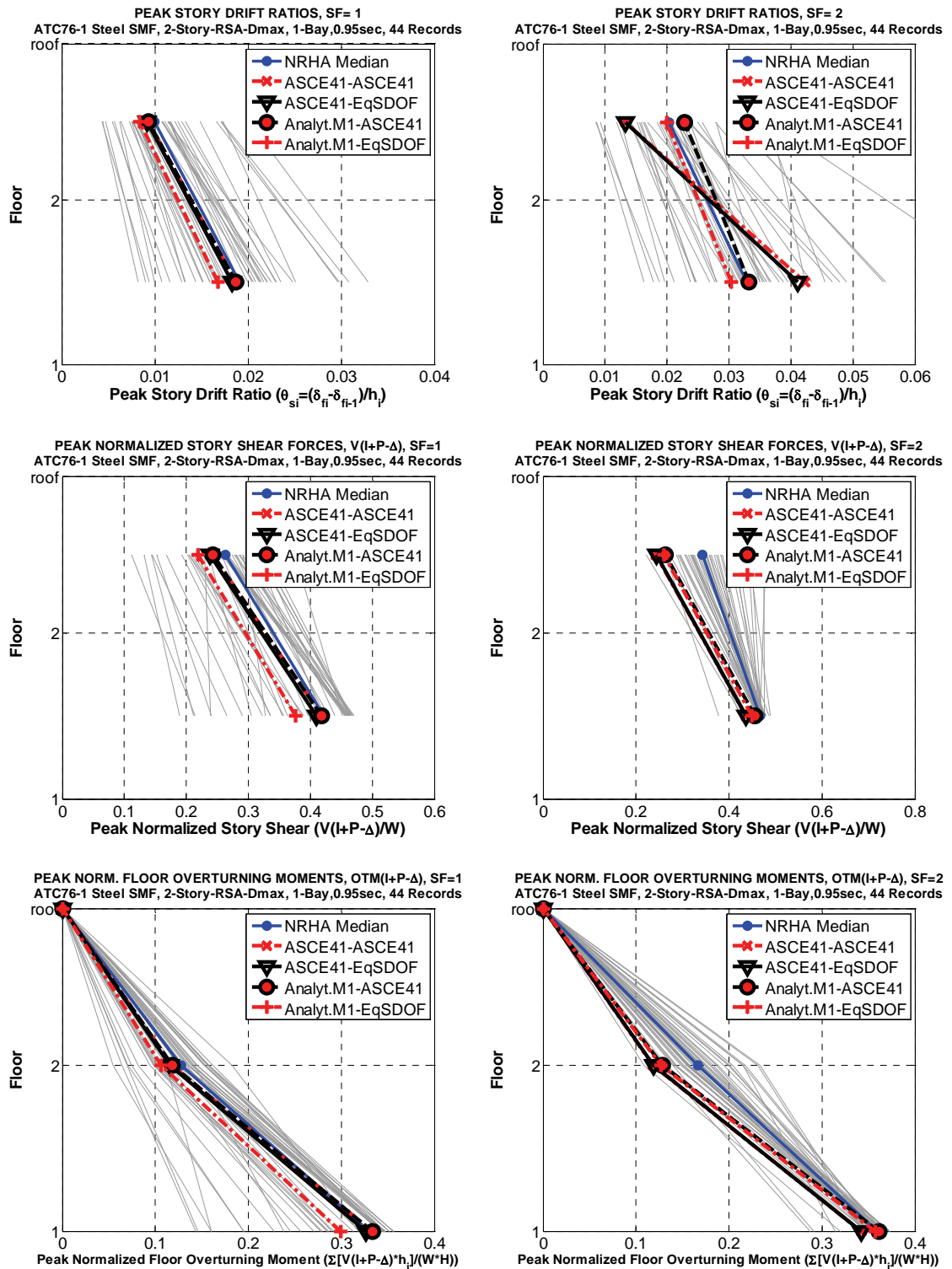


Figure 5-9 Comparison of nonlinear response history analysis results (median and dispersion) to results from various single-mode nonlinear static analysis options for 2-story steel moment frames.

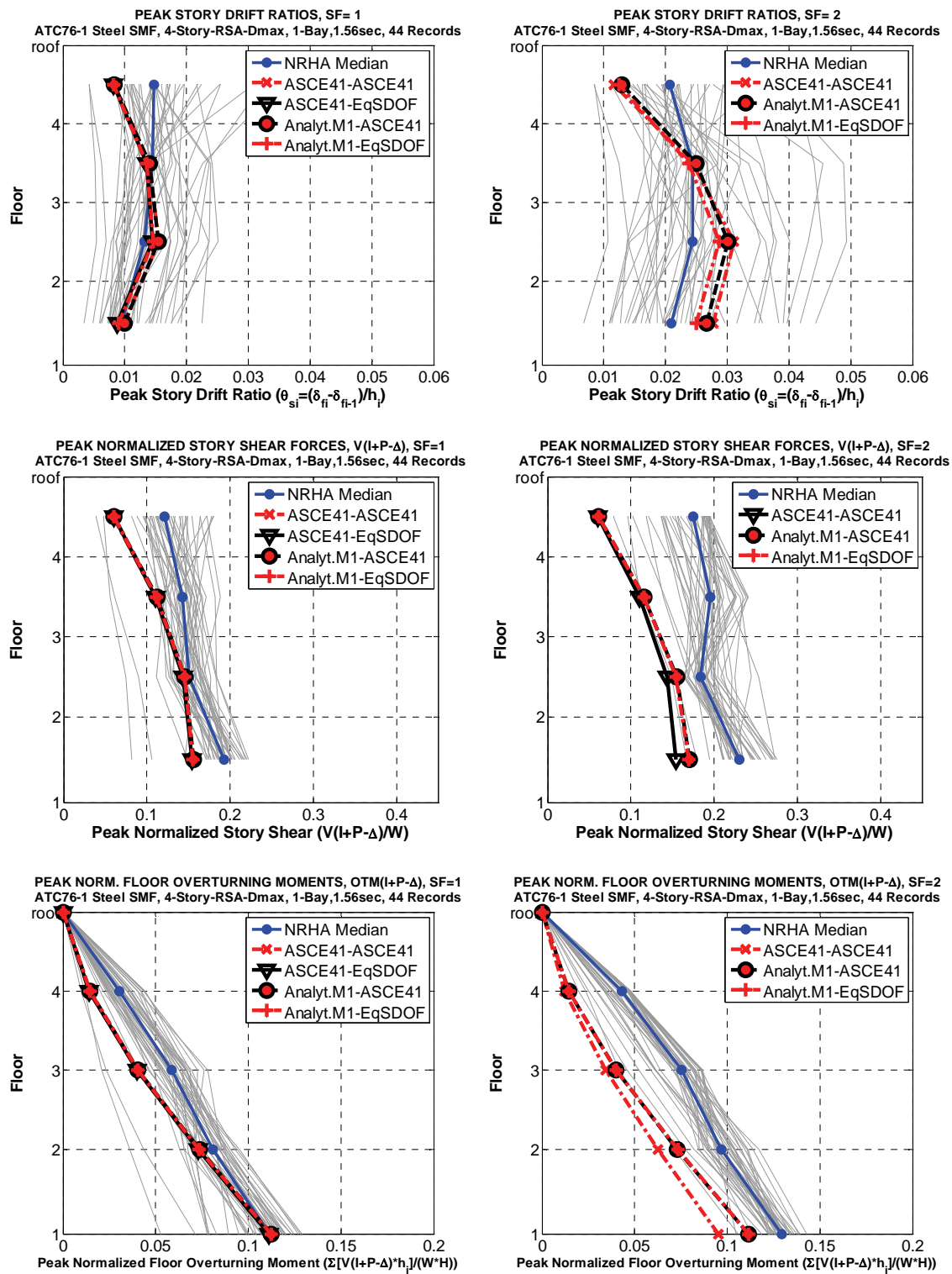


Figure 5-10 Comparison of nonlinear response history analysis results (median and dispersion) to results from various single-mode nonlinear static analysis options for 4-story steel moment frames.

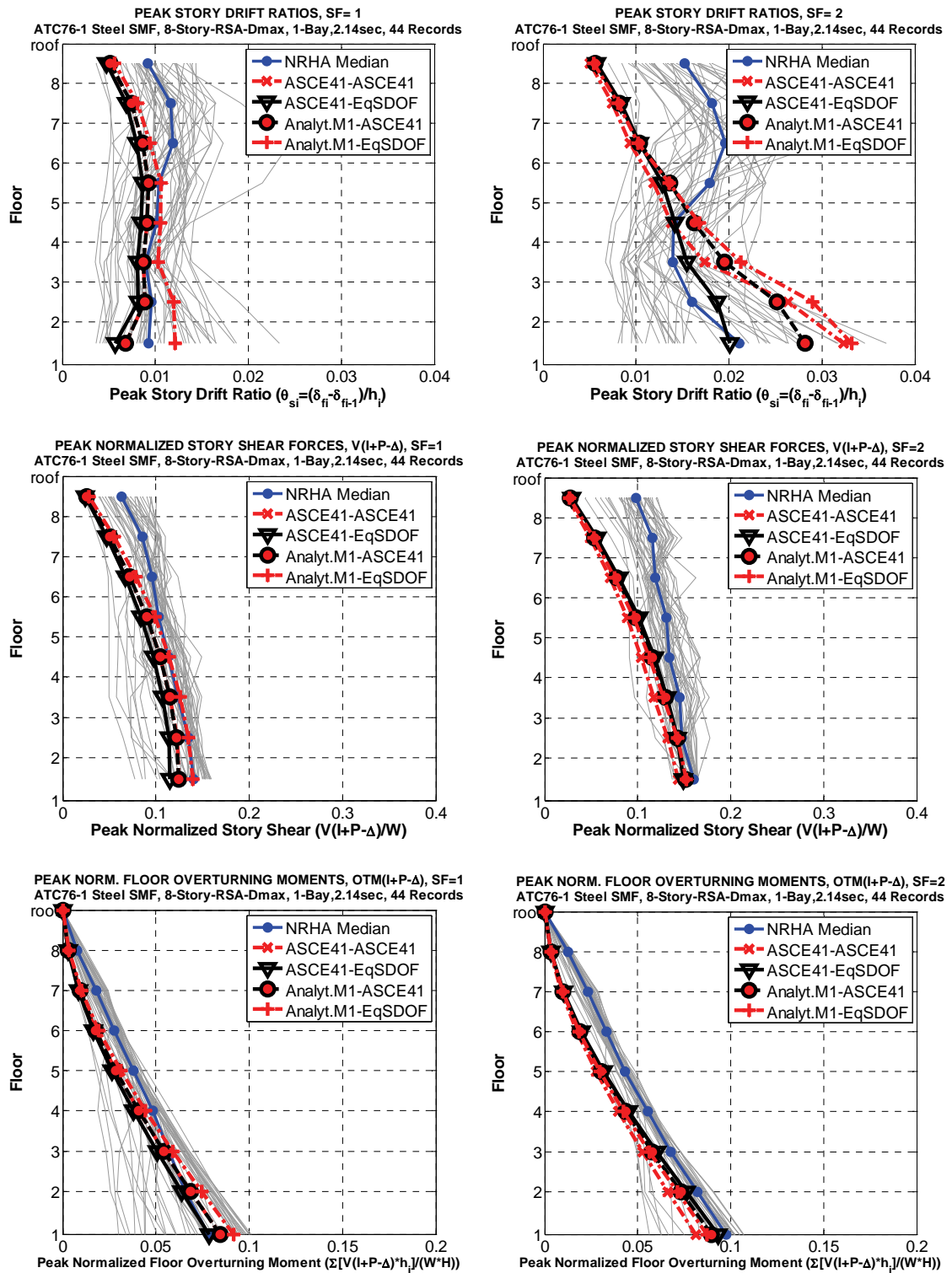


Figure 5-11 Comparison of nonlinear response history analysis results (median and dispersion) to results from various single-mode nonlinear static analysis options for 8-story steel moment frames.

3. In the first story, single-mode nonlinear static procedures can underestimate base shear demand. The reason for this is dynamic amplification due to redistribution of inelastic deformations to adjacent stories, which cannot be accounted for in a first mode invariant load pattern. This is illustrated by the first story shear demand in the 4-story steel moment frame for a scale factor of 2.0, in which the median result from nonlinear response history analysis is about 50% higher than results predicted by the different single-mode static procedures.
4. The process of assessing collapse potential using the ASCE/SEI 41-06 equation for R_{max} or the FEMA P-440A equation for R_{di} was not necessarily conservative. Pushover curves using Analyt.M1 component models passed both the R_{max} and R_{di} criteria for ground motion scale factors as high as 3.0, but nonlinear response history analysis of the equivalent single-degree-of-freedom system resulted in dynamic instability (i.e., collapse) in 38 out of 44 ground motions.

Component Models – Single-Mode Nonlinear Static Analysis

1. The accuracy of single-mode nonlinear static analysis predictions is not very sensitive to the choice of component model (ASCE41 or Analyt.M1). Exceptions include: (a) when the target displacement is in the negative tangent stiffness region of the global pushover curve; and (b) when different component models result in post-yield tangent stiffness slopes of opposite sign.
2. Use of ASCE41 component models underestimate post-yield strength and deformation capacities of steel moment frame structures in comparison with the Analyt.M1 component models. The consequences for prediction of target displacement increase at higher scale factors.
3. In limited study, use of the Analyt.M3 component model was found to produce pushover curves that are close to the ASCE41 component curves and that single-mode nonlinear static predictions based on Analyt.M3 and ASCE41 pushover curves are similar.

Target Displacement – Single-Mode Nonlinear Static Analysis

1. In most cases, the accuracy of single-mode nonlinear static analysis predictions does not depend strongly on the method used to predict the target displacement. Results were similar when the coefficient method (ASCE41) and the equivalent SDOF method (EqSDOF) were used because both methods predicted roof drifts that were close to the median roof drift obtained from nonlinear response history analysis. Exceptions include: (a) when the target displacement is in the negative tangent stiffness region of the global pushover curve; and (b) for short period structures in which the roof drift can be very sensitive to the degree of nonlinearity.
2. In most cases, all single-mode nonlinear static analysis optional combinations of component model and target displacement produced similar response quantity

predictions (similarly accurate or inaccurate). Again, this observation should not be extended beyond the range of structures discussed here. The exceptions noted in (1) apply here as well.

- When the target displacement is in the negative tangent stiffness region of the global pushover curve, the coefficient method is unreliable, and prediction of target displacement should be performed using an equivalent single-degree-of-freedom nonlinear response history analysis method (e.g., the Equivalent SDOF or SPO2IDA methods).

Summary Results – Single-Mode Nonlinear Static Analysis

Table 5-2 summarizes ratios of response quantity estimates to median results from nonlinear response history analysis for steel moment frame structures subjected to single-mode nonlinear static analysis. Values shown represent maximum deviations over the height of the structure. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis.

Table 5-2 Ratios of Response Quantity Estimates for Single-Mode Nonlinear Static Analysis of Steel Moment Frame Structures

	Scale Factor	Response Quantity Ratios ¹		
		Story Drift	Story Shear	Overturning Moment
2-story	1.0	0.9 to 1.0	0.9 to 1.0	0.9 to 1.0
	2.0	1.0 to 1.1	0.8 to 1.0	0.8 to 1.0
4-story	1.0	0.6 to 1.1	0.5 to 0.9	0.5 to 1.0
	2.0	0.6 to 1.3	0.3 to 0.8	0.3 to 1.0
8-story	1.0	0.6 to 1.0	0.4 to 0.9	0.4 to 1.1
	2.0	0.4 to 1.6	0.3 to 1.0	0.3 to 1.0

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

Variation in the ratios of peak response quantity estimates over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story steel moment frame structures subjected to different intensities of ground motion (SF = 1.0 and 2.0) is shown in Figure 5-12. For these scale factors, the target displacement is safely below negative tangent stiffness region of the global pushover curve. Results in the figure are for the combination of Analyt.M1 component model and the coefficient method for predicting target displacement (Analyt.M1-ASCE41).

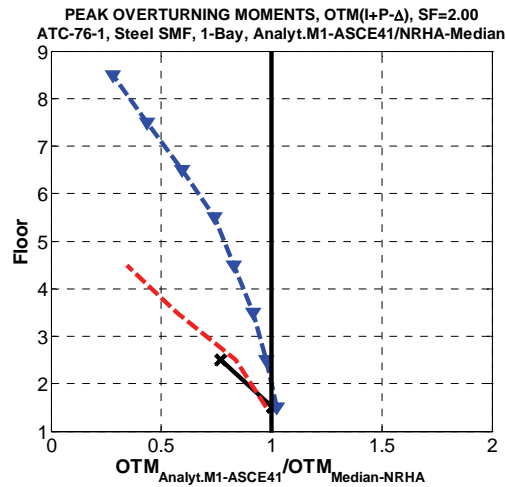
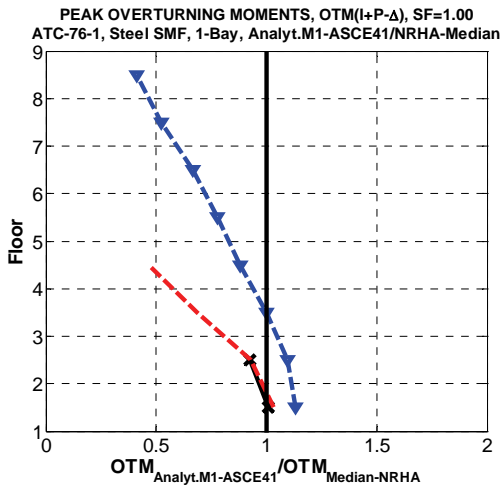
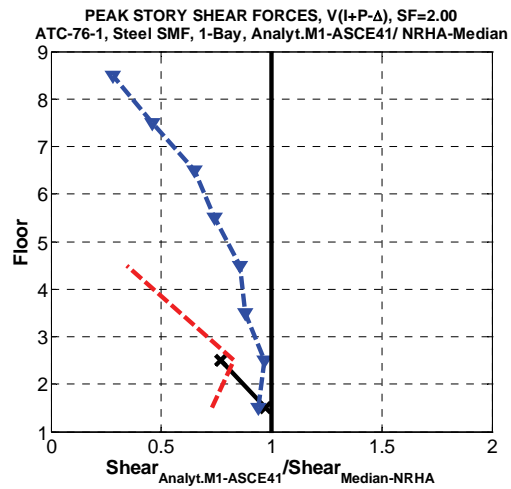
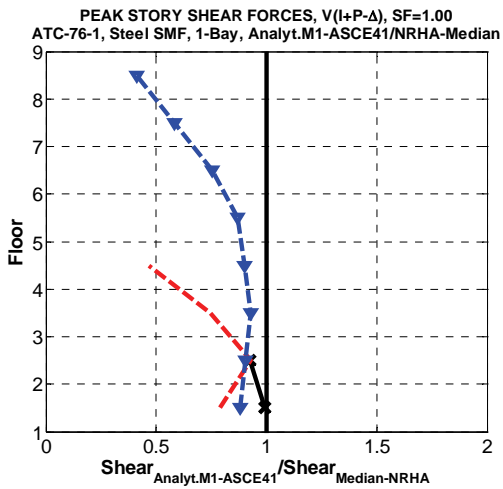
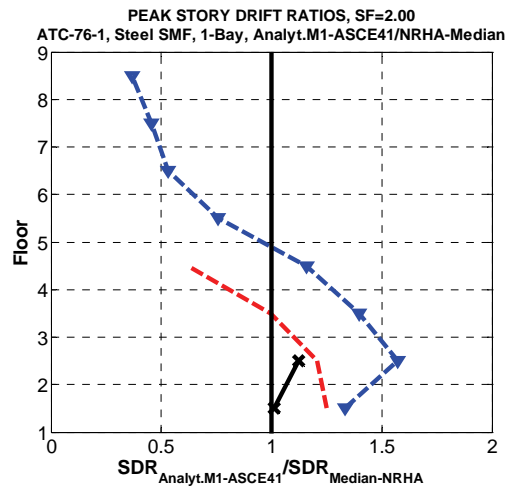
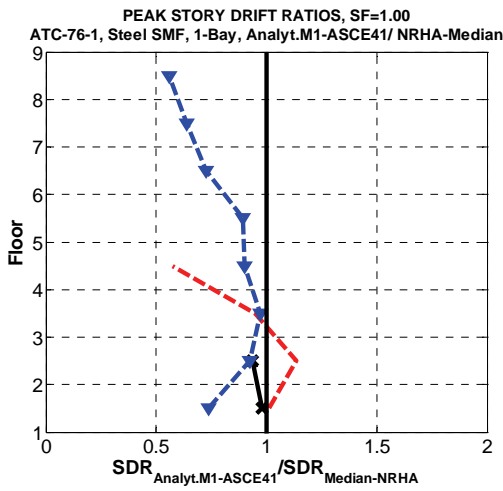


Figure 5-12 Ratios of response quantity estimates to median nonlinear response history analysis results over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story steel moment frame structures at scale factors SF=1.0 (left) and SF=2.0 (right).

5.2.4 Multi-Mode Nonlinear Static Analysis

Steel moment frame structures were investigated using the Modal Pushover Analysis (MPA) procedure developed by Chopra and Goel (2001), but without the extension of the method intended to improve estimation of local member forces developed by Chopra and Goel (2004). The MPA procedure, as implemented, follows the more common application among practitioners of determining all response quantities in a single analysis phase. Analyses were conducted using the Analyt.M1 component model and the equivalent SDOF method of estimating target displacement. Figure 5-13 shows first and second mode pushover curves and the corresponding equivalent SDOF system representations for the 4-story steel moment frame.

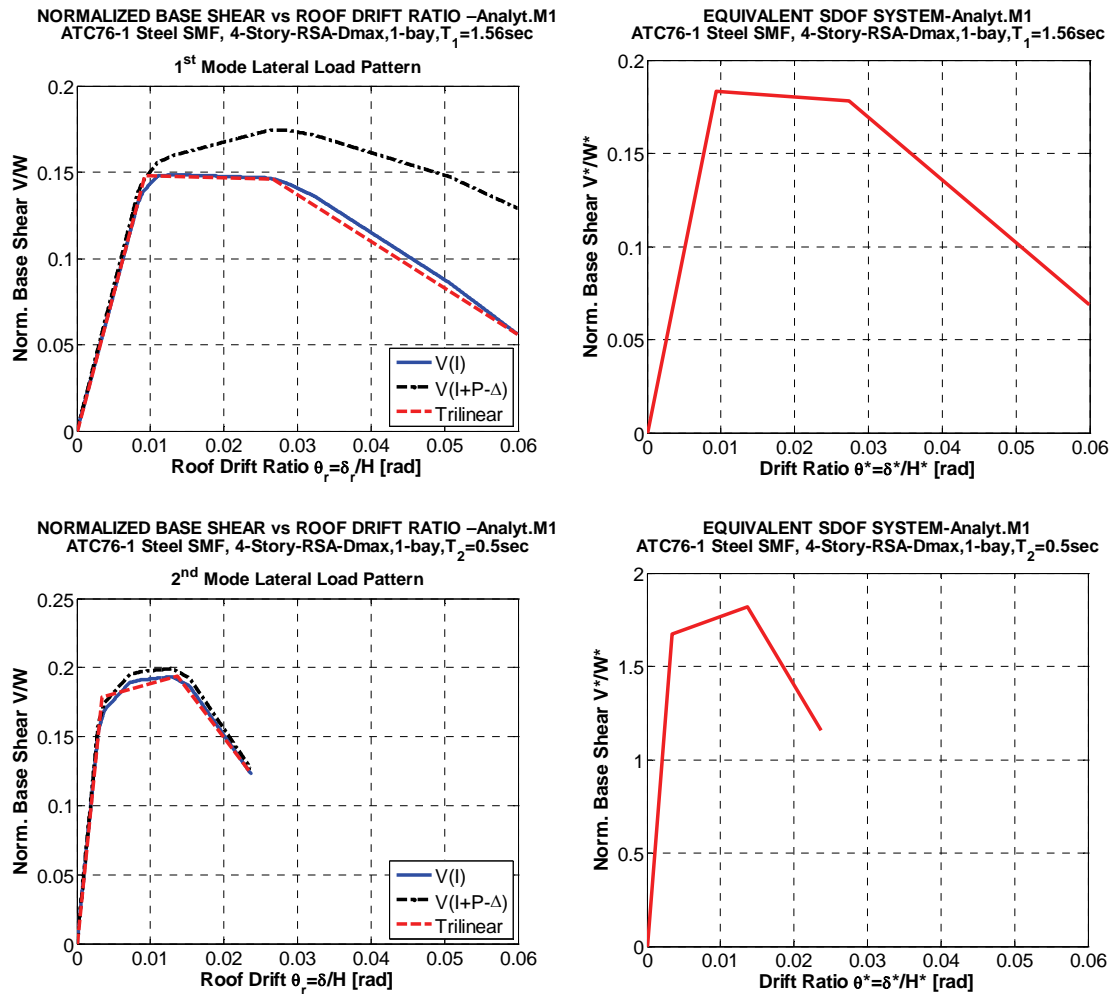


Figure 5-13 First mode and second mode pushover curves and equivalent SDOF systems for the 4-story steel moment frame.

Selected comparisons between nonlinear response history analysis results (median and dispersion) and results from Modal Pushover Analysis of 4-story and 8-story steel moment frames are shown in Figure 5-14. More complete results are presented in Appendix A.

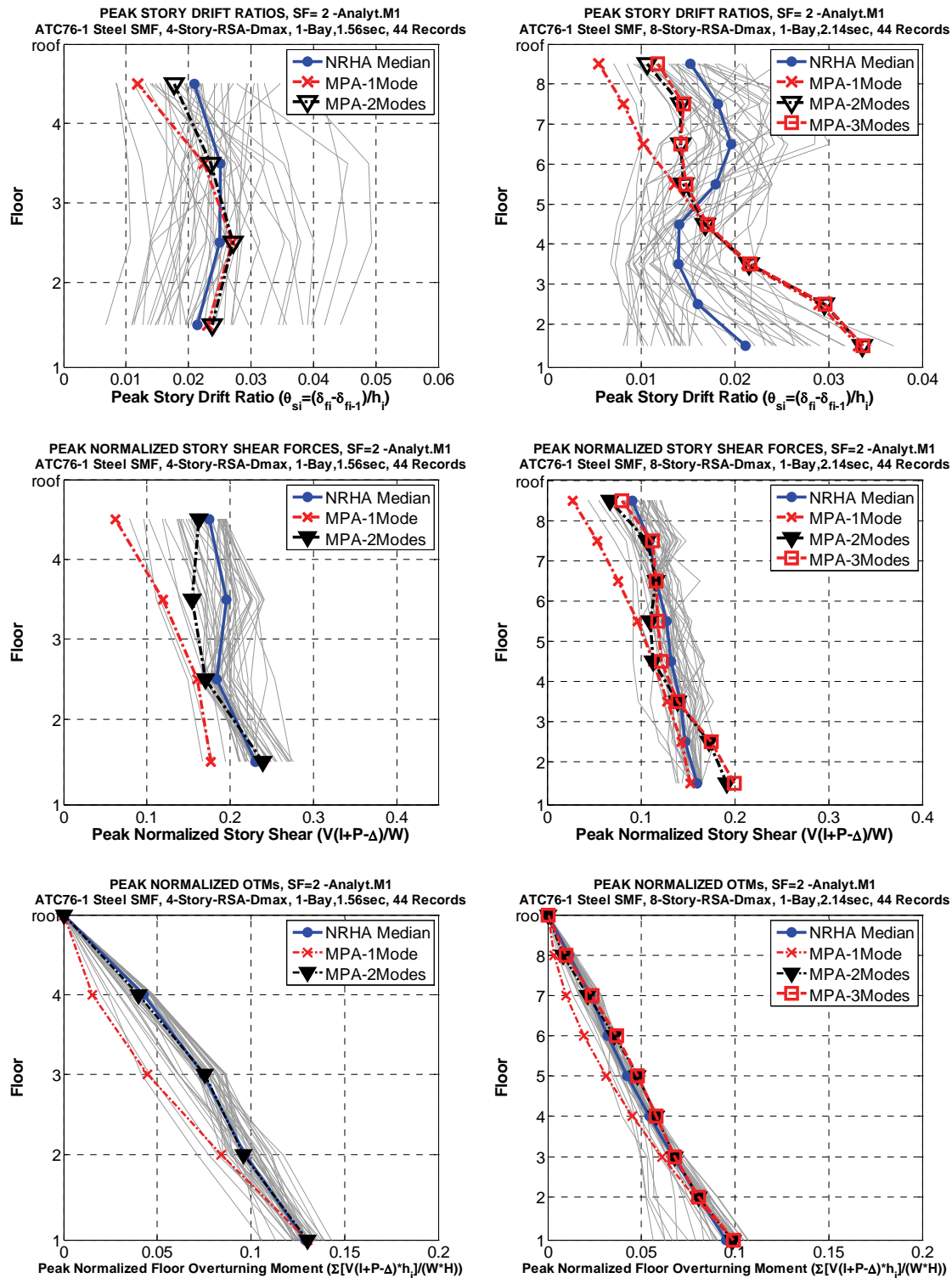


Figure 5-14 Comparison of nonlinear response history analysis results (median and dispersion) to results from Modal Pushover Analysis of the 4-story steel moment frame (left) and 8-story steel moment frame (right).

In many cases, particularly for low-rise regular structures, higher mode target displacements are smaller than the yield displacement, which implies that higher mode responses are elastic. Where this occurs, response quantities are modal

combinations of inelastic first mode contributions and elastic higher mode contributions. This simplifies the modal combination and avoids ambiguities that can be caused by displacement reversals sometimes observed in inelastic higher mode pushover analyses.

Table 5-3 summarizes ratios of response quantity estimates to median results from nonlinear response history analysis for steel moment frame structures subjected to Modal Pushover Analysis. Values shown represent maximum deviations over the height of the structure. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis.

Table 5-3 Ratios of Response Quantity Estimates for Modal Pushover Analysis of Steel Moment Frame Structures

	Scale Factor	Response Quantity Ratios ¹		
		Story Drift	Story Shear	Overturning Moment
4-story	1.0	0.7 to 1.1	0.8 to 1.0	0.8 to 1.0
	2.0	0.8 to 1.1	0.8 to 1.0	1.0 to 1.0
8-story	1.0	0.8 to 1.2	0.7 to 1.1	0.8 to 1.2
	2.0	0.7 to 1.9	0.9 to 1.2	1.0 to 1.1

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

The following summary observations are made regarding Modal Pushover Analysis of steel moment frame structures:

1. In all cases investigated, Modal Pushover Analysis led to improved prediction of response quantities compared to the single-mode nonlinear static analysis.
2. Consideration of second mode response led to considerable improvement in prediction of response quantities. Consideration of the third mode did not change results much, even for the 8-story steel moment frame system.
3. For the 4-story regular steel moment frame ($T_1 = 1.56$ sec), the improvement over single-mode nonlinear static analysis for all story-level demand parameters is substantial.
4. For the 8-story regular steel moment frame, Modal Pushover Analysis significantly improves story drift, story shear force, and overturning moment predictions in the upper stories. However, prediction of story drift in the lower stories is off by more than 50% at a scale factor of 2.0. The reason for this can be seen in Figure 5-15, which shows large amplification of story drifts in the

lower stories for a scale factor of 2.0. This is an indication of sensitivity to the pushover load pattern.

5. Load pattern sensitivity is also an issue if the system can develop story mechanisms, as illustrated for structures with special characteristics in Section 5.5.

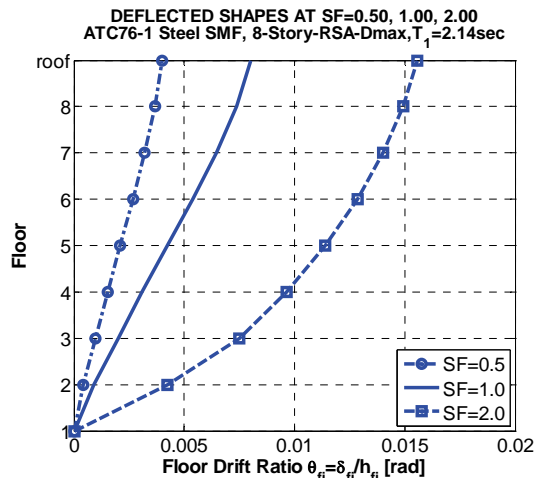


Figure 5-15 Deflected shapes of the 8-story steel moment frame structure.

6. Modal Pushover Analysis, however, is no more sensitive to load pattern than single-mode nonlinear static procedures.
7. Modal Pushover Analysis is not recommended when displacement reversals occur in higher mode pushover analyses.

5.2.5 Elastic Modal Response Spectrum Analysis

Elastic modal response spectrum analysis was tested as an alternative to nonlinear static analysis for predication of demand parameters on the 4-story steel moment frame structure. Table 5-4 summarizes ratios of response quantity estimates to median nonlinear response history analysis results. Figure 5-16 shows the variation over height between nonlinear response history analysis results (median and dispersion) and results from elastic modal response spectrum analysis.

Table 5-4 Ratios of Response Quantity Estimates for Elastic Modal Response Spectrum Analysis of Steel Moment Frame Structures

	Scale Factor	Response Quantity Ratios ¹		
		Story Drift	Story Shear	Overtuning Moment
4-story	0.5	0.8 to 1.0	0.8 to 1.0	0.8 to 1.1
	2.0	1.0 to 1.2	1.4 to 2.0	1.2 to 2.4

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

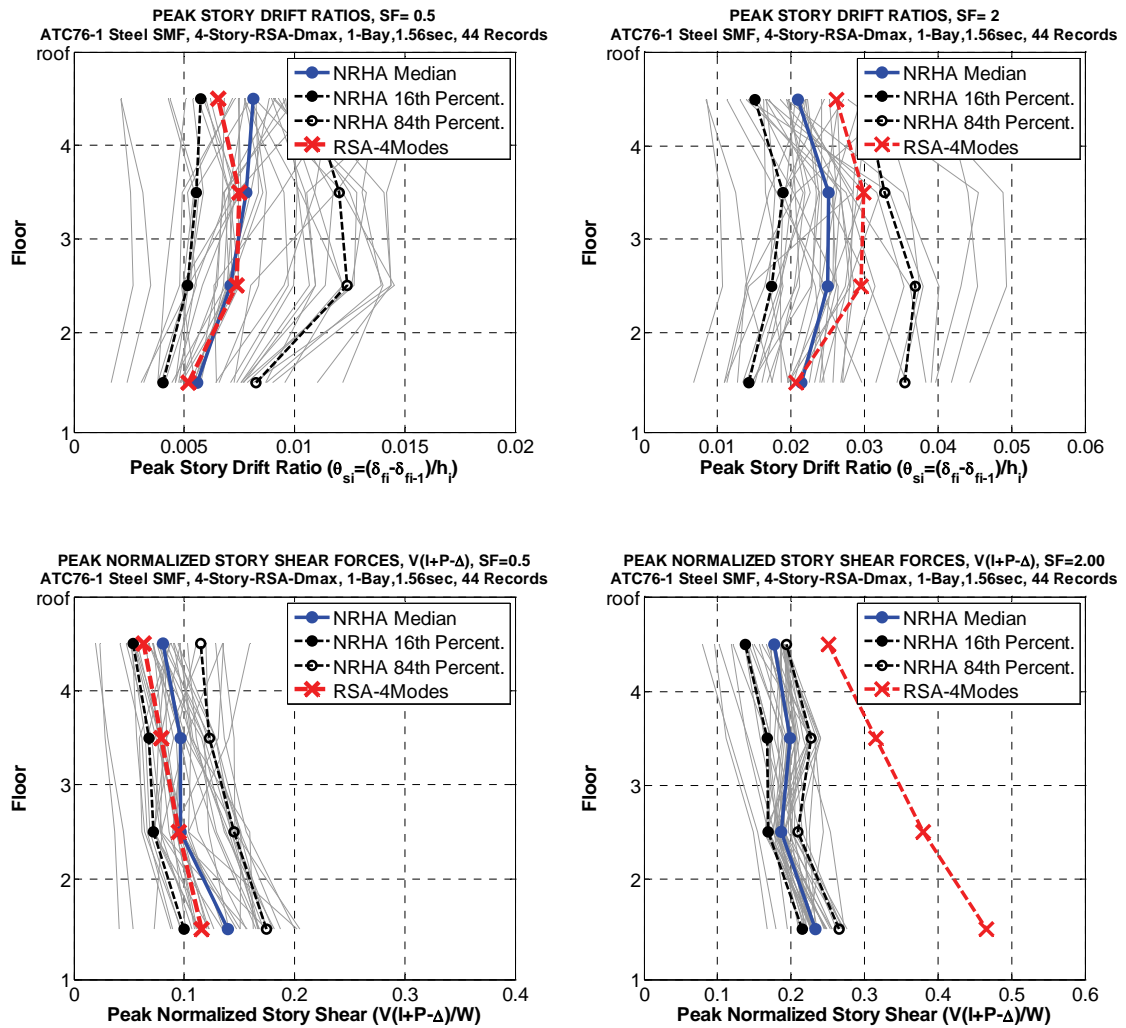


Figure 5-16 Comparison of nonlinear response history analysis results (median and dispersion) to results from elastic modal response spectrum analysis of the 4-story steel moment frame at scale factors 0.5 (left) and 2.0 (right).

The following summary observations are made regarding elastic modal response spectrum analysis of steel moment frame structures:

1. As might be expected, elastic modal response spectrum analysis demand parameters are reasonably accurate where the response is elastic or near-elastic for a scale factor of 0.5.
2. When response is highly nonlinear (SF = 2.0), elastic modal response spectrum analysis can produce reasonable predictions of story drift, provided the structure is regular and not subject to severe strength or stiffness deterioration.
3. Elastic modal response spectrum analysis can significantly overpredict force quantities in yielding structures.

5.3 Reinforced Concrete Moment Frame Studies

This section summarizes results of problem-focused studies on nonlinear response of reinforced concrete moment frame prototypes. Nonlinear response history analysis was performed using the FEMA P-695 far-field ground motion set, and various single-mode and multiple-mode nonlinear static analysis procedures were evaluated for their ability to predict peak values of floor displacement, story drift ratio, story shear force, and floor overturning moment. Detailed results are presented in Appendix B.

5.3.1 Reinforced Concrete Moment Frame Prototypes

Reinforced concrete moment frame prototypes consist of a subset of the reinforced concrete moment frame (RCMF) archetypes designed and analyzed in the FEMA P-695 report, *Quantification of Building Seismic Performance Factors* (FEMA, 2009b). The subset consists of three structures designed by means of equivalent lateral force (ELF) analysis for seismic design category D_{\max} ($S_{DS} = 1.0g$ and $S_{DI} = 0.60g$), designated as follows:

- 2-story RCMF (archetype ID 1001)
- 4-story RCMF (archetype ID 1010)
- 8-story RCMF (archetype ID 1012)

The structures consist of 3-bay distributed moment-resisting space frames with the plan view and elevation shown in Figure 5-17. As designed, the frames resist all seismic design forces and receive tributary gravity loads as indicated in the shaded portion of the figure.

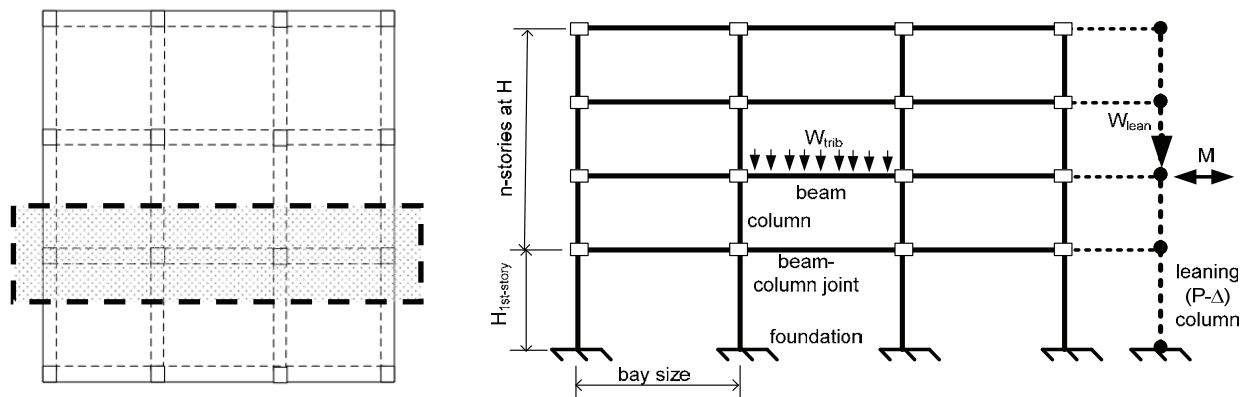


Figure 5-17 Plan and elevation of reinforced concrete moment frame prototypes (FEMA, 2009).

All frames consist of special reinforced concrete moment-resisting elements, designed in accordance with the 2003 *International Building Code* (ICC, 2003). Analytical models are based on the properties of the bare concrete moment-resisting frames. Nonlinear component models are capable of capturing severe strength and

stiffness deterioration, and have been calibrated based on a database of recent testing, as described in FEMA P-695. A leaning column was used to capture P-Delta effects.

5.3.2 *Nonlinear Response History Analysis*

Selected results from nonlinear response history analysis of 2-, 4-, and 8-story reinforced concrete moment frames are shown in Figures 5-18 through 5-20, which show the variation in peak values of the response quantities over height for each ground motion record along with statistical quantities (median, mean, and mean \pm 1 standard deviation) of the results. Figure 5-21 shows a comparison of median results for the 4-story and 8-story reinforced concrete moment frames at scale factors of 0.5, 1.0, and 2.0.

Nonlinear response history analysis of concrete moment frame structures led to the following observations regarding prediction of response quantities:

1. Inspection of responses to each ground motion record indicates that higher modes generally have little influence on peak displacements and an appreciable effect on story drifts and story shears.
2. As was the case for steel moment frames, story shears are not consistent with the lateral load pattern for which the structure was designed or the first mode load pattern in a pushover analysis. Story shears in the upper stories are significantly larger than would be expected from these load patterns and increase disproportionately with an increase in scale factor. Base shears developed in the nonlinear response history analyses are larger than the capacities indicated by a first-mode pushover analysis.
3. Peak overturning moments tend to follow a concave pattern that is consistent with the lateral load pattern used in design or a first-mode pushover load pattern, and dispersion in overturning moments is relatively small.
4. Median peak story drifts tend to concentrate in the lower stories with increasing scale factor.
5. Dispersion in response quantities is substantial and is related to the dispersion inherent in the suite of ground motion records used in the analyses. Dispersion in peak story shears and overturning moments reduces with increasing scale factor, as yielding of plastic hinges constrains the forces and moments that can be developed.
6. It appears that dispersion in peak story drifts may increase with increasing scale factor.
7. The distribution of component-level demand parameters (e.g., beam plastic hinge rotation, column shear force, and column axial force) correlates well with story-level demand parameters (e.g., story drift, story shear, and overturning moment).

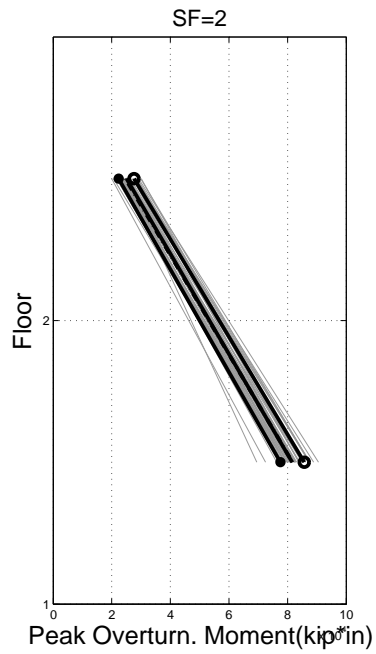
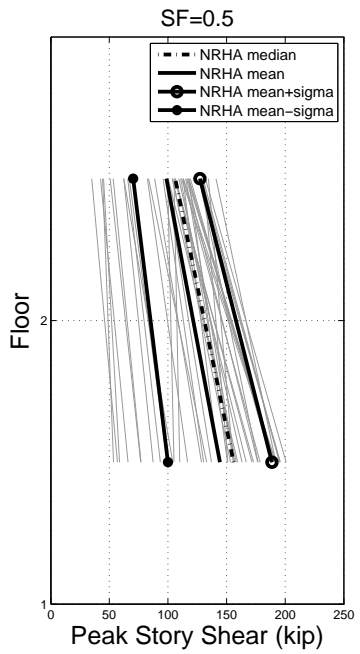
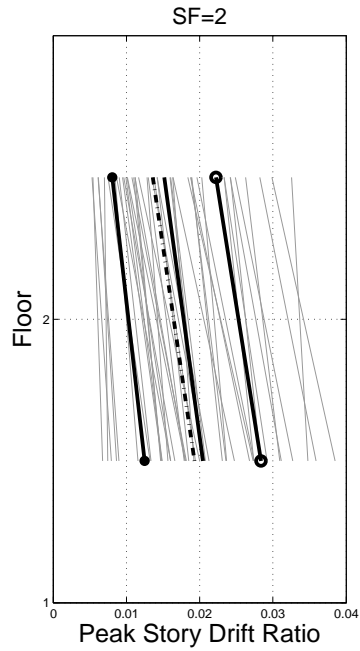
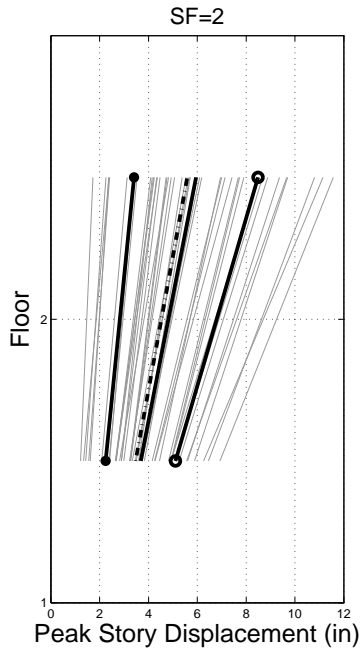


Figure 5-18 Selected results for nonlinear response history analysis of the 2-story reinforced concrete moment frame.

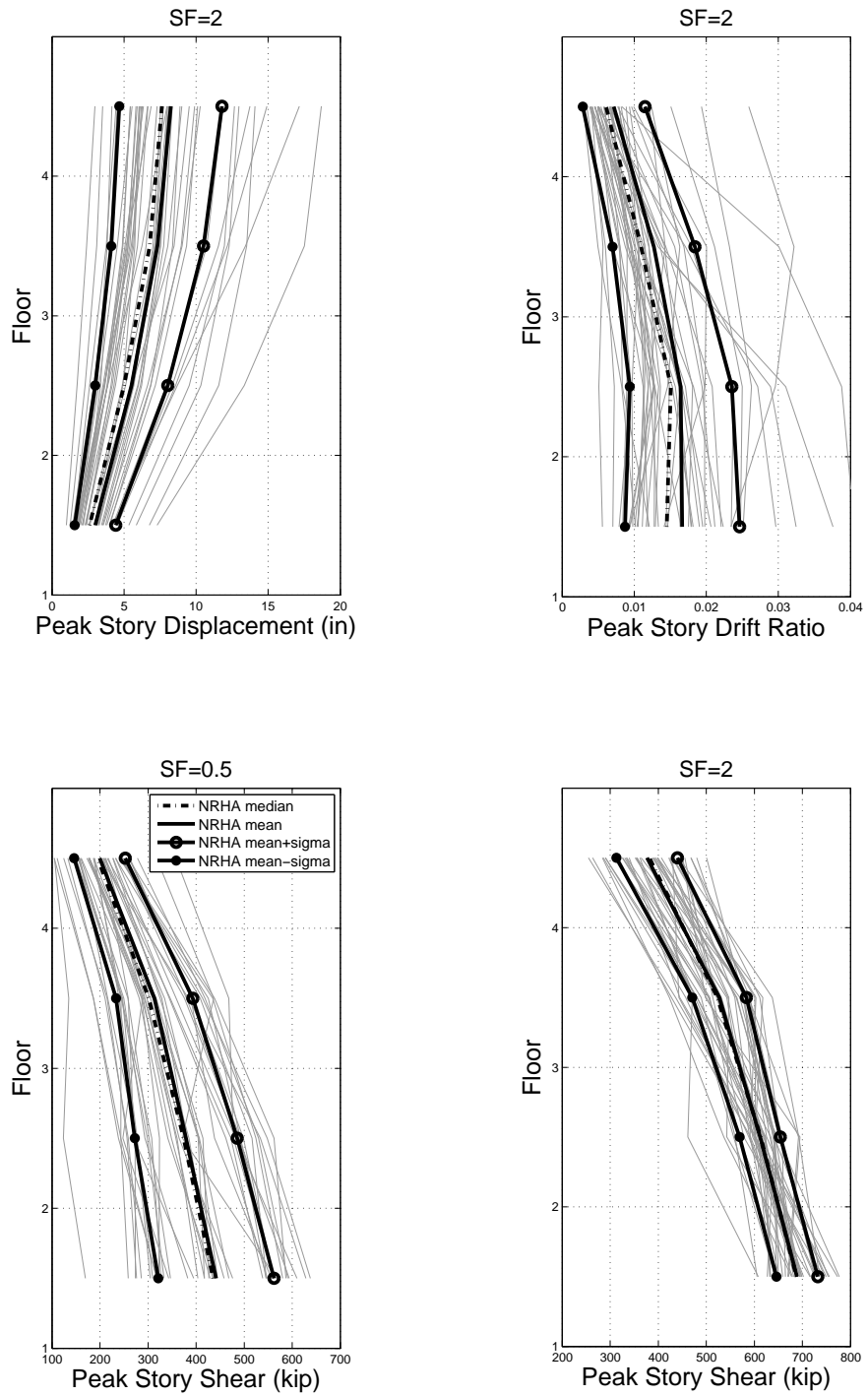


Figure 5-19 Selected results for nonlinear response history analysis of the 4-story reinforced concrete moment frame.

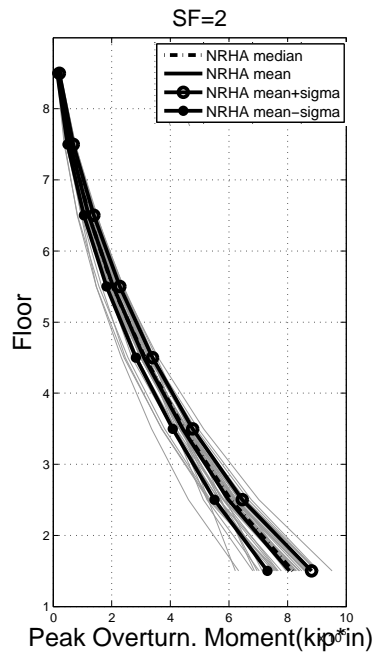
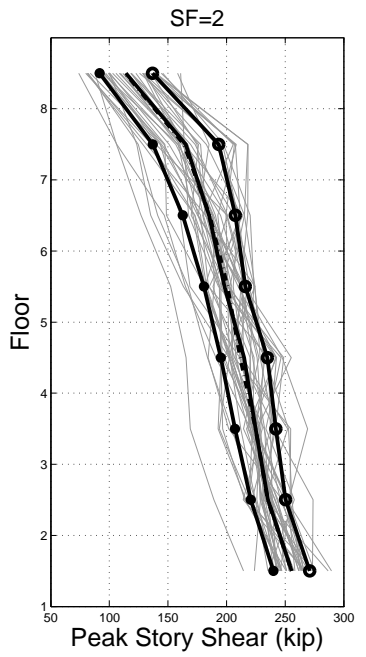
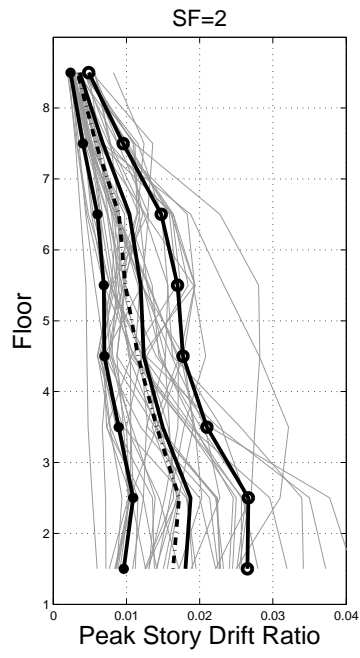
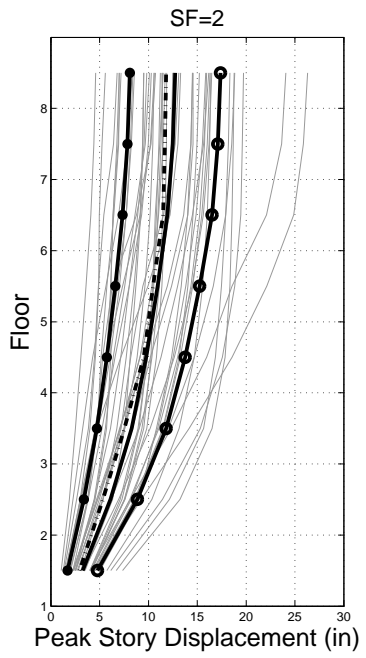


Figure 5-20 Selected results for nonlinear response history analysis of the 8-story reinforced concrete moment frame.

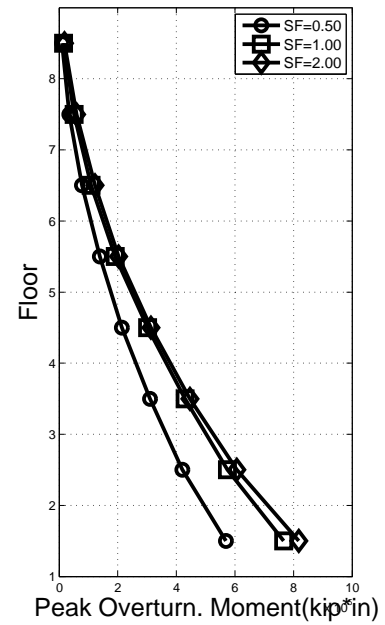
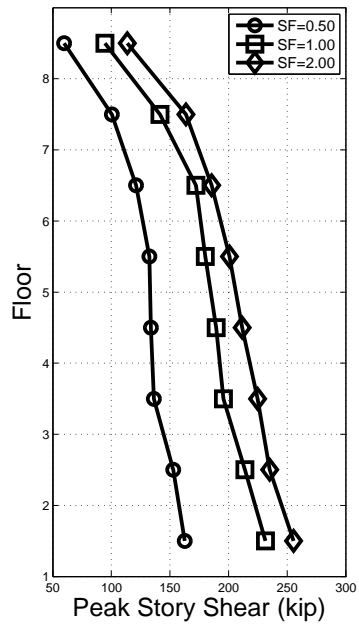
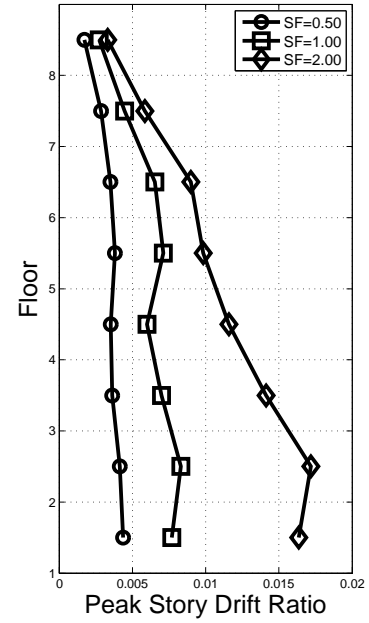
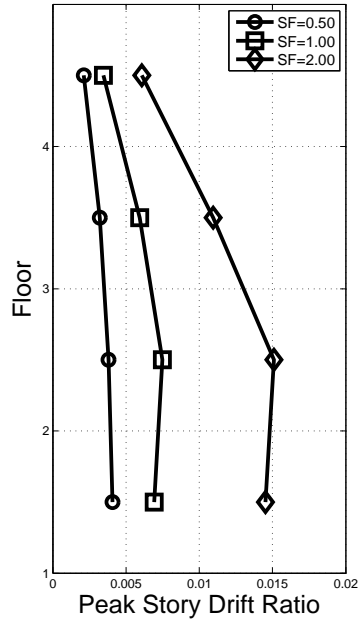


Figure 5-21 Median nonlinear response history analysis results at scale factors of 0.5, 1.0, and 2.0 for the 4-story (upper left) and 8-story (remaining) reinforced concrete moment frames.

5.3.3 Single-Mode Nonlinear Static Analysis

Variations in single-mode nonlinear static analysis of reinforced concrete moment frames included the use of different methods of predicting the target displacement. Target displacements were estimated using the coefficient method (ASCE41) and the N2 method in Eurocode 8 (N2/EC8). In all cases an invariant first mode load pattern was used to determine the pushover response.

A comparison between global pushover curves for 2-, 4-, and 8-story reinforced concrete moment frames is shown in Figure 5-22. Also shown are the target displacements at scale factors of 0.5, 1.0, and 2.0.

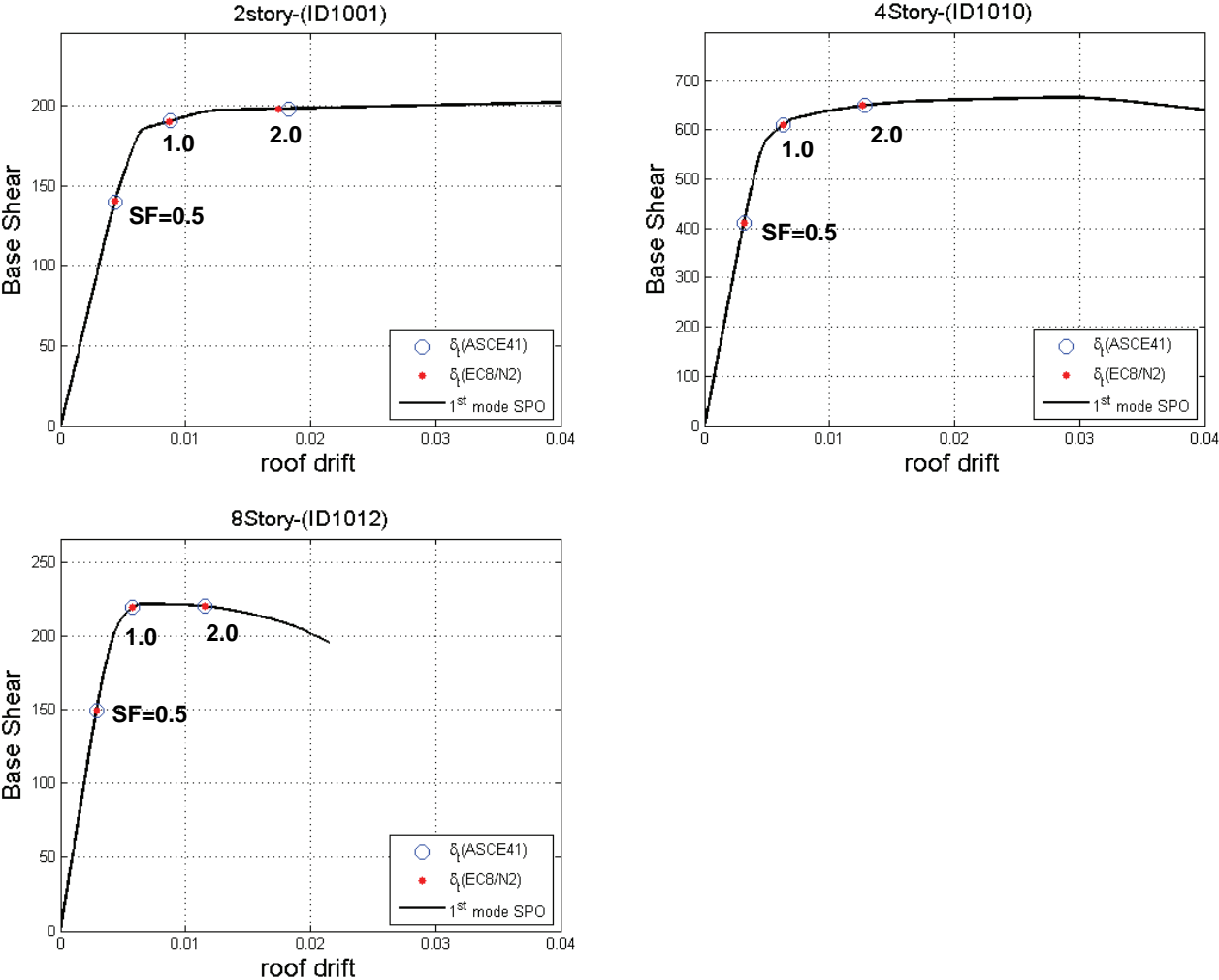


Figure 5-22 Comparison of global pushover curves for 2-, 4-, and 8-story reinforced concrete moment frames showing target displacements at scale factors 0.5, 1.0 and 2.0.

Yield occurs at a drift of about 0.6% of the height. Target displacements for scale factor of 0.5 are in the elastic region, nearly elastic for scale factor of 1.0, and moderately inelastic for scale factor of 2.0. Since target displacements obtained using the ASCE41 and N2/EC8 methods were nearly identical, the ASCE41 method

was used to estimate response quantities in the remaining single-mode nonlinear static analyses.

Selected comparisons between nonlinear response history analysis results (median, mean, and dispersion) and results from single-mode nonlinear static analysis of 2-, 4-, and 8-story reinforced concrete moment frames, are shown in Figures 5-23 through 5-26. The legend in each figure identifies the nonlinear response history statistical quantity and nonlinear static analysis target displacement method (e.g., ASCE41). Results are shown for scale factors 0.5 and 2.0. More complete results are presented in Appendix B.

Comparisons between single-mode nonlinear static analysis and nonlinear response history analysis results for reinforced concrete moment frame structures led to the following observations:

1. For the 2-story frame, peak displacements, story drifts, and overturning moments were estimated with reasonable accuracy at all scale factors. While peak story shears were estimated accurately for a scale factor of 0.5, they were underestimated at higher scale factors.

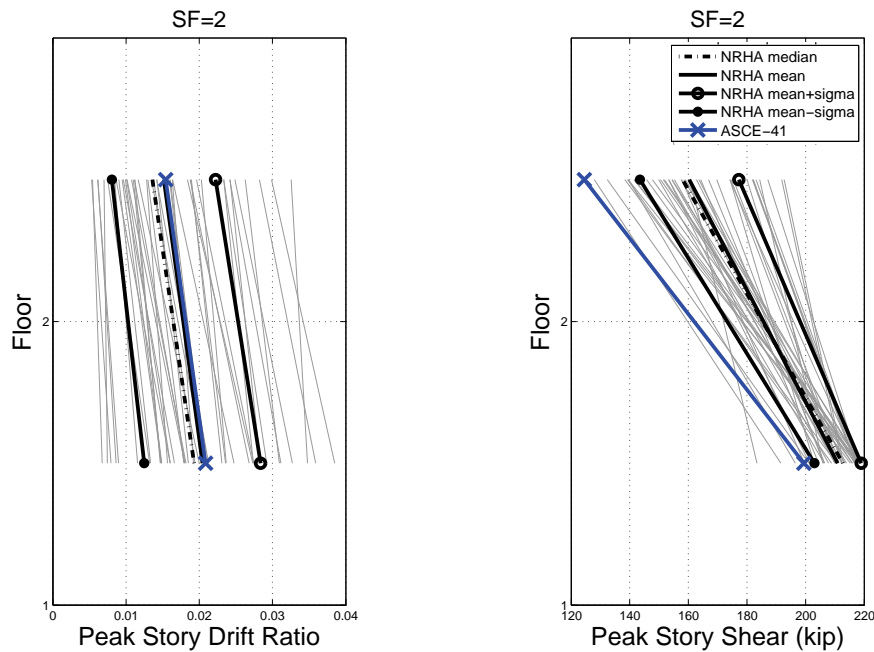


Figure 5-23 Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factor 2.0.

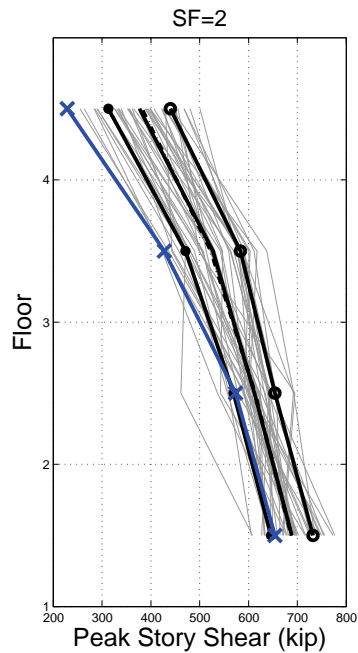
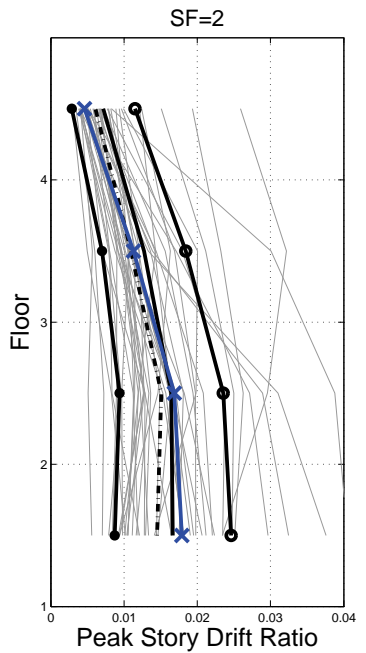
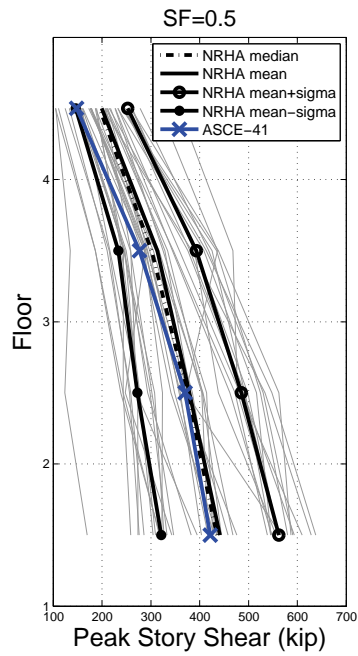
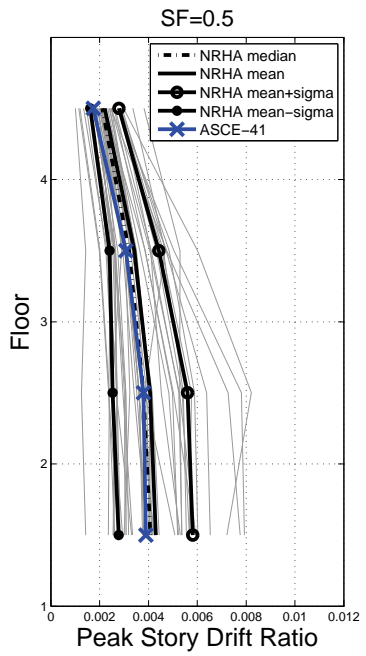


Figure 5-24 Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 4-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0.

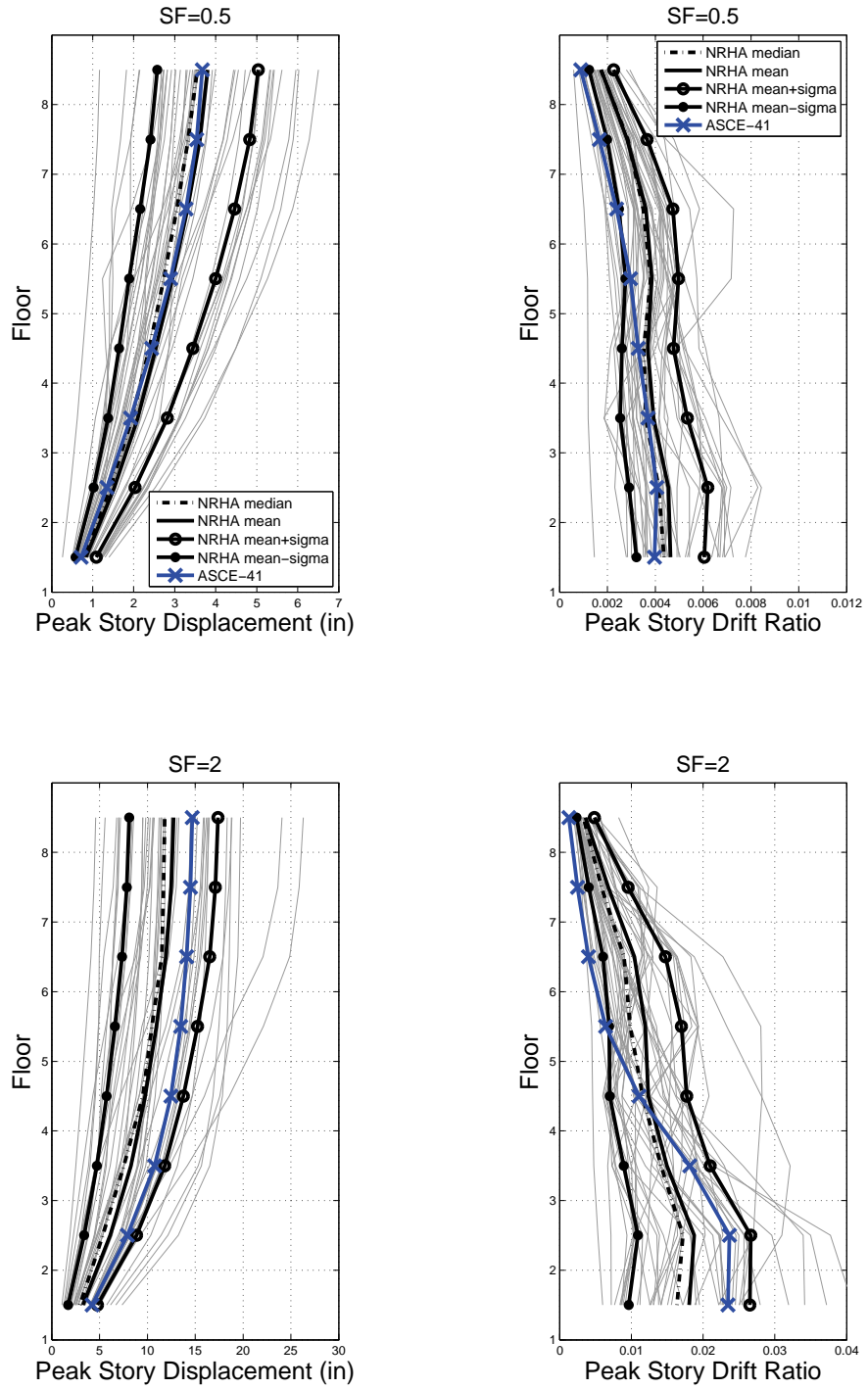


Figure 5-25 Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 8-story reinforced concrete moment frame for displacement response quantities at scale factors 0.5 and 2.0.

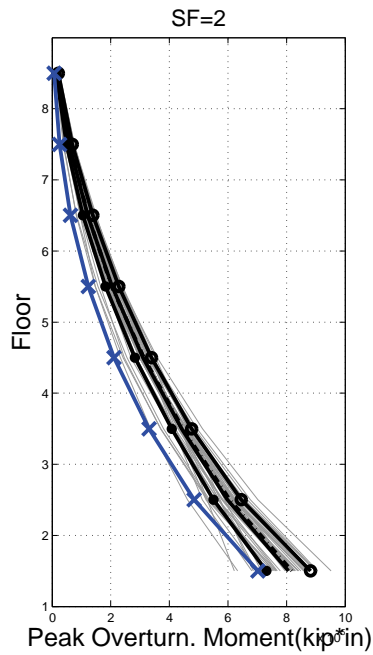
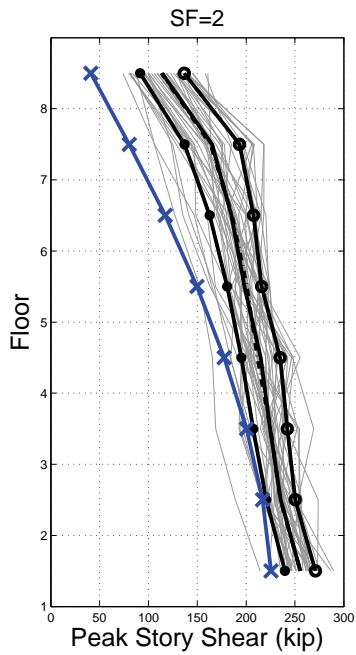
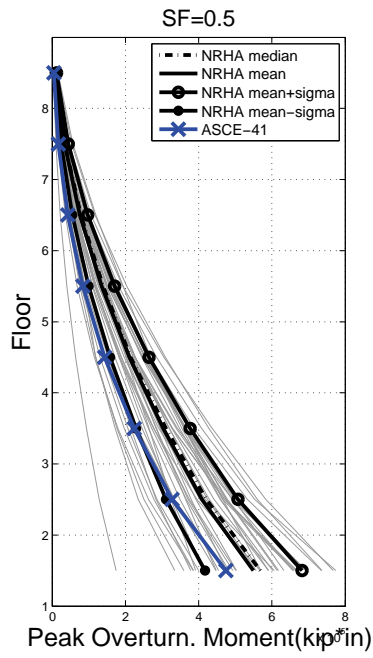
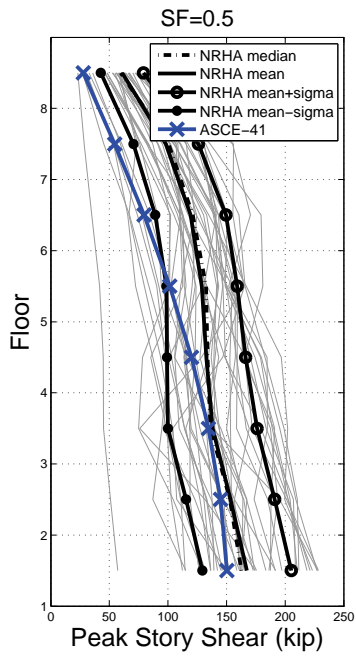


Figure 5-26 Comparison of nonlinear response history analysis to single-mode nonlinear static analysis of the 8-story reinforced concrete moment frame for force response quantities at scale factors 0.5 and 2.0.

2. For the 4-story frame, peak displacements and overturning moments were estimated with reasonable accuracy at all scale factors. Story drift and story shear estimates were reasonably accurate at low scale factors, but the accuracy degraded slightly as the scale factor increased from 0.5 to 2.0.
3. For the 8-story frame, the accuracy of displacement response estimates (peak displacement and peak drift ratio) degraded as the scale factor increased from 0.5 to 2.0. The tendency of equivalent SDOF systems to overestimate peak displacement response of MDOF systems with increasing degree of nonlinearity has been recognized in previous studies.
4. For the 8-story frame, force response estimates (peak story shear and peak overturning moment) were relatively inaccurate at all scale factors, especially in the upper stories, and the accuracy degraded as the scale factor increased from 0.5 to 2.0.

Table 5-5 summarizes ratios of response quantity estimates to median nonlinear response history analysis results for single-mode nonlinear static analysis of reinforced concrete moment frame structures. Values shown represent maximum deviations over the height of the structure. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis. Figure 5-27 shows how the ratios vary over height for 2-, 4-, and 8-story structures at scale factors of 0.5 and 2.0.

Table 5-5 Ratios of Response Quantity Estimates for Single-Mode Nonlinear Static Analysis of Reinforced Concrete Moment Frame Structures

		Response Quantity Ratios ¹				
		Scale Factor	Displacement	Story Drift	Story Shear	Overturning Moment
2-story	0.5		0.9 to 0.9	0.9 to 0.9	0.8 to 0.9	0.8 to 0.9
	1.0		1.1 to 1.1	1.0 to 1.1	0.9 to 1.0	0.9 to 1.0
	2.0		1.1 to 1.1	1.1 to 1.1	0.8 to 0.9	0.8 to 1.0
4-story	0.5		1.0 to 1.0	0.8 to 1.0	0.8 to 1.0	0.8 to 1.0
	1.0		1.1 to 1.2	0.8 to 1.2	0.7 to 1.0	0.7 to 1.0
	2.0		1.1 to 1.2	0.8 to 1.2	0.6 to 1.0	0.6 to 1.0
8-story	0.5		0.9 to 1.1	0.5 to 1.0	0.5 to 1.0	0.5 to 0.8
	1.0		1.1 to 1.2	0.5 to 1.3	0.4 to 1.0	0.4 to 0.9
	2.0		1.2 to 1.5	0.4 to 1.4	0.4 to 0.9	0.4 to 0.9

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

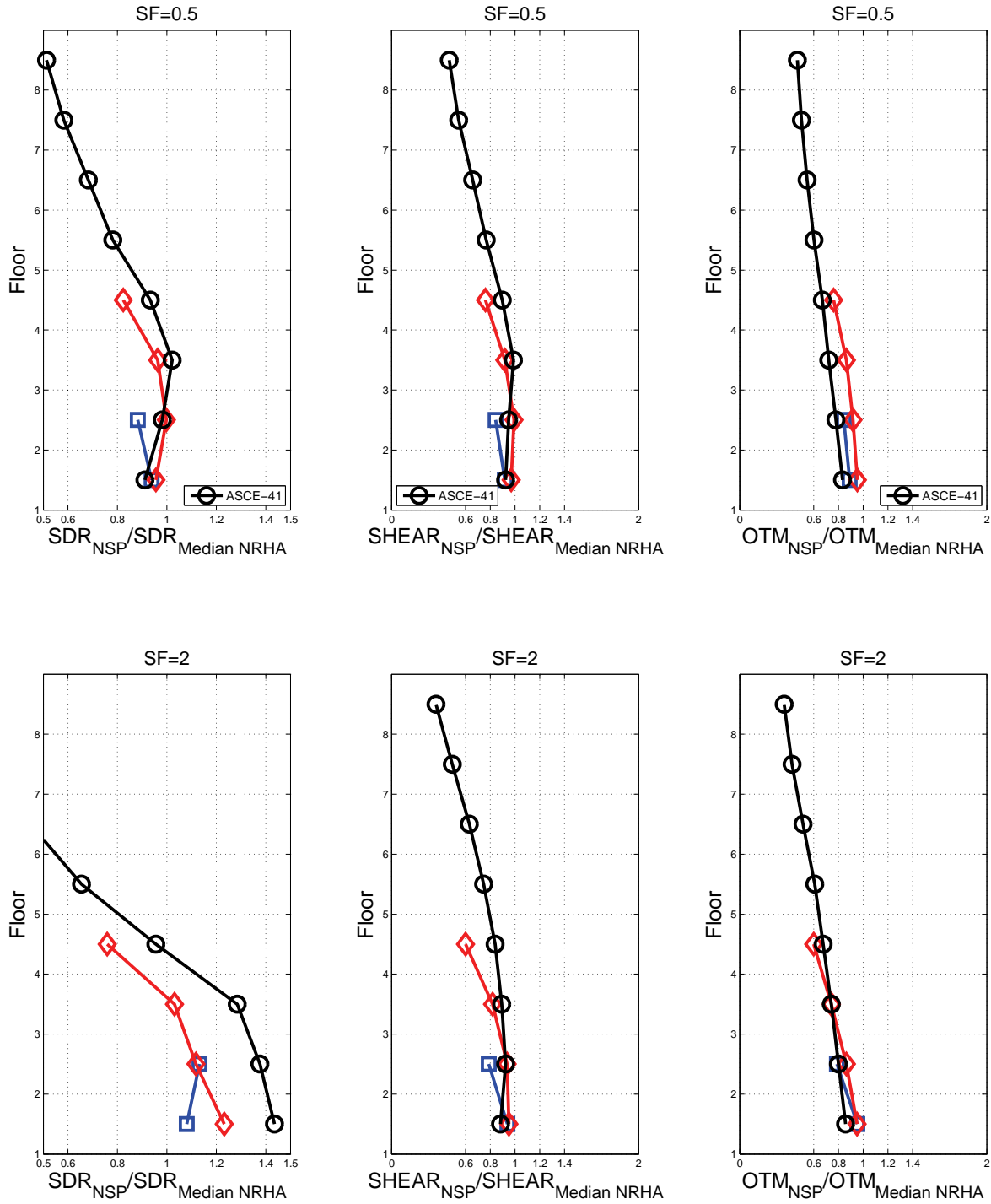


Figure 5-27 Ratios of response quantity estimates to median nonlinear response history analysis results over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story reinforced concrete moment frame structures for story drift ratio (left), story shear (middle), and overturning moment (right) at scale factors 0.5 and 2.0.

5.3.4 Multi-Mode Nonlinear Static Analysis

Reinforced concrete moment frame structures were investigated using the Modal Pushover Analysis (MPA) procedure developed by Chopra and Goel (2001) and the Consecutive Modal Pushover (CMP) analysis procedure developed by Poursha et al. (2009). As was the case for steel moment frame structures, the MPA procedure, as implemented, followed the more common application among practitioners of determining all response quantities in a single analysis phase and excluded the extension of a second analysis phase to better estimate element forces developed by Chopra and Goel (2004). Target displacements for the first mode pushover analyses were determined using the coefficient method in ASCE/SEI 41-06.

Modal Pushover Analysis

Figure 5-28 shows the modal pushover curves for the 2-, 4-, and 8-story reinforced concrete moment frames. Two modes were used for the 2-story frame, and three modes were used for the 4- and 8-story frames.

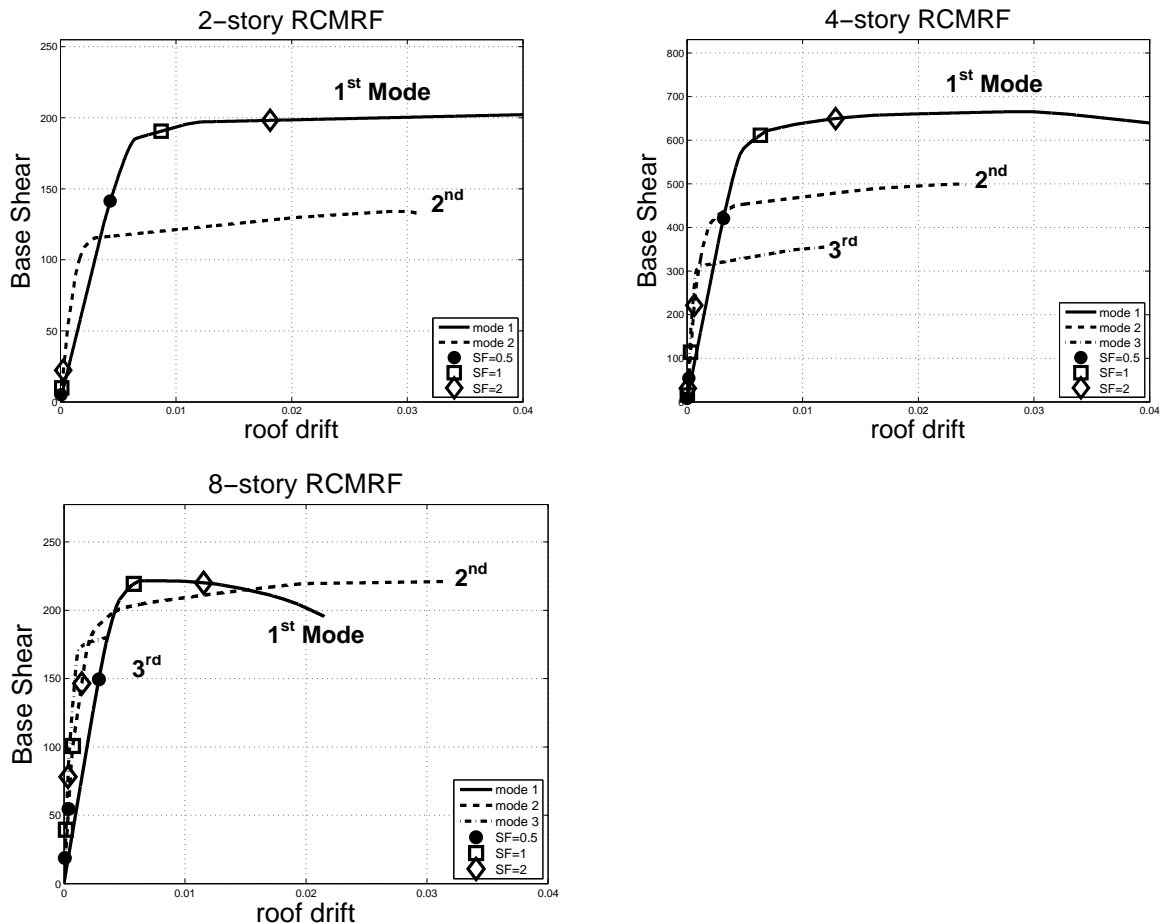


Figure 5-28 Modal pushover curves for the 2-, 4-, and 8-story reinforced concrete moment frames and first-mode target displacement estimates at scale factors 0.5, 1.0, and 2.0.

Selected comparisons between nonlinear response history analysis results (median, mean, and dispersion) and results from Modal Pushover Analysis of 2-, 4-, and 8-story reinforced concrete moment frames, are shown in Figures 5-29 through 5-31. Selected results are shown for scale factors 0.5 and 2.0. More complete results are presented in Appendix B.

Comparisons between Modal Pushover Analysis and nonlinear response history analysis results for reinforced concrete moment frame structures led to the following observations:

1. For the 2-story frame, second mode contributions to floor displacements, story drifts, and overturning moments were negligible, and reasonably accurate estimates of these quantities could be obtained using the first mode response alone. While story shears were estimated with reasonable accuracy at a scale factor of 0.5, the accuracy degraded at higher scale factors, and inclusion of second mode contributions to response did not improve this result.

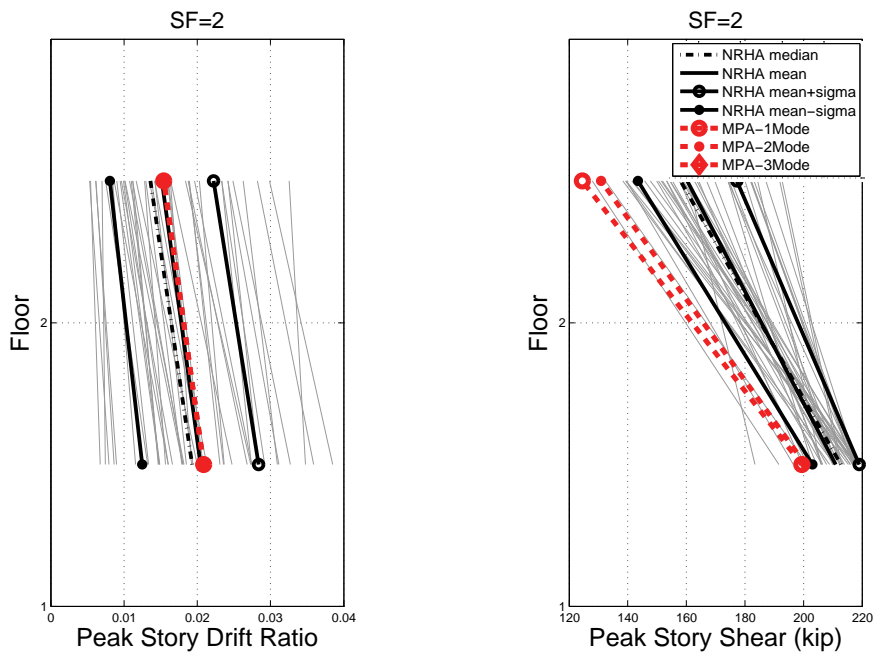


Figure 5-29 Comparison of nonlinear response history analysis to Modal Pushover Analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factor 2.0.

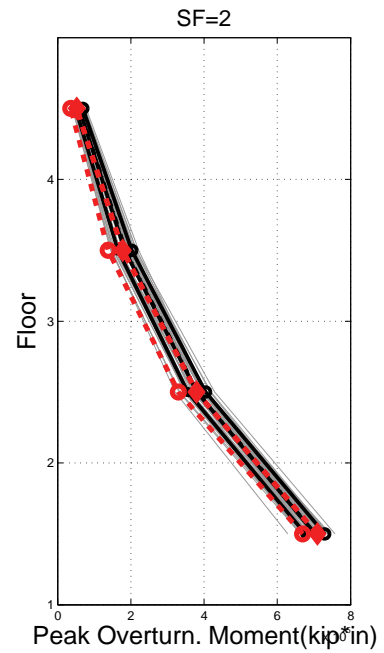
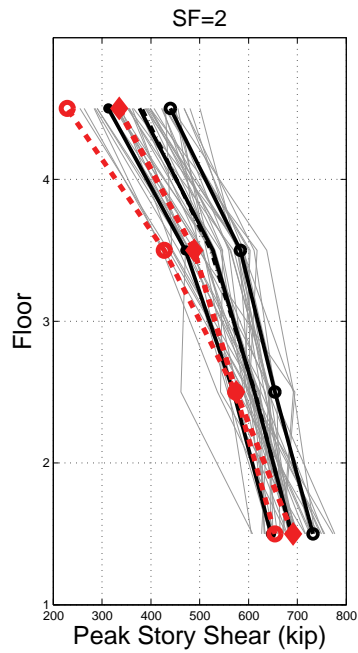
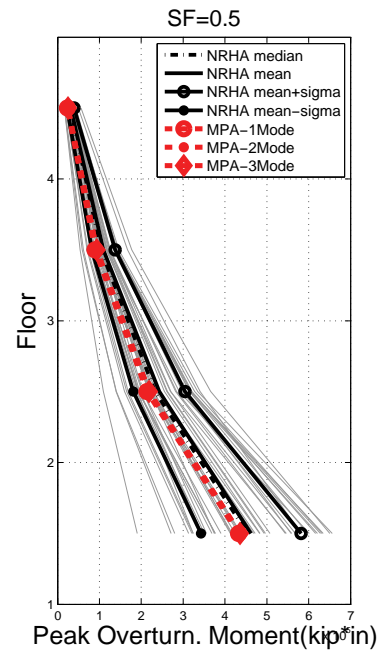
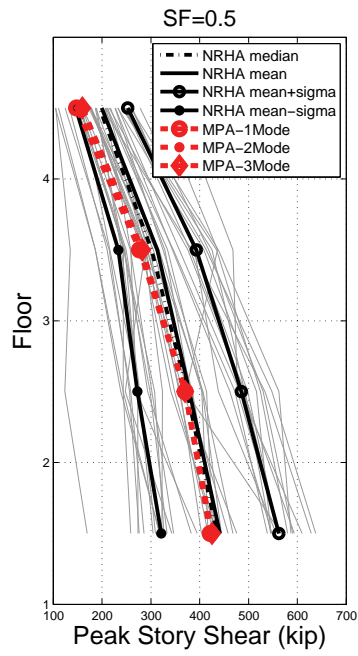


Figure 5-30 Comparison of nonlinear response history analysis to Modal Pushover Analysis of the 4-story reinforced concrete moment frame for peak story shear (left) and peak overturning moment (right) at scale factors 0.5 and 2.0.

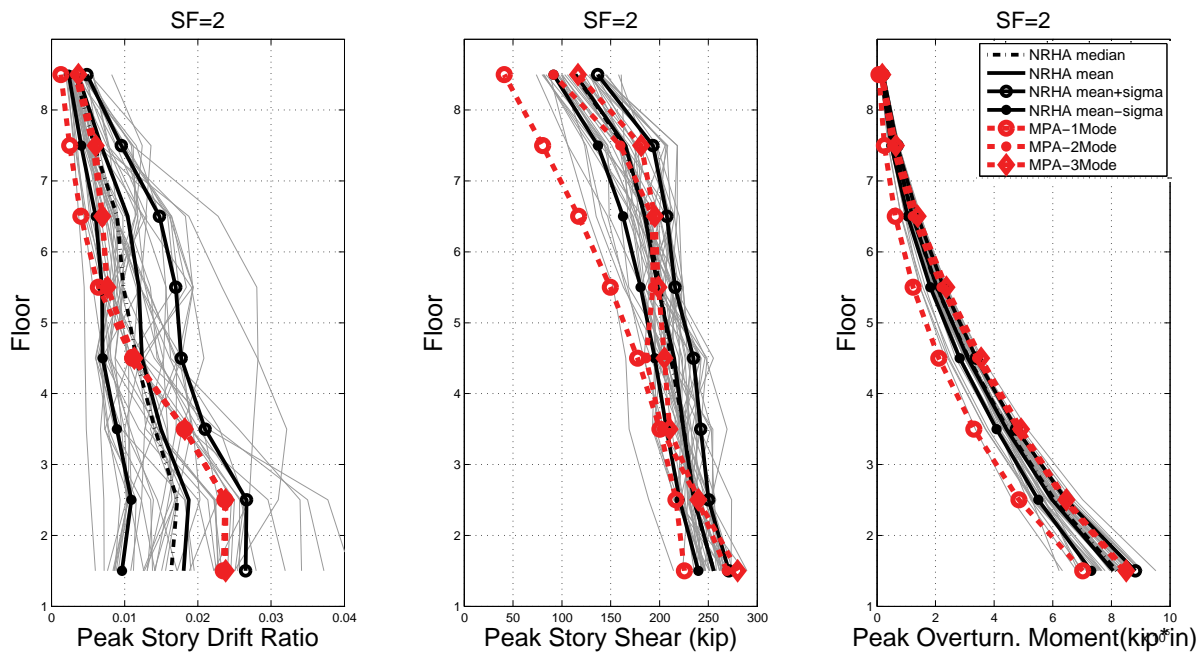


Figure 5-31 Comparison of nonlinear response history analysis to Modal Pushover Analysis of the 8-story reinforced concrete moment frame for peak story drift ratio (left), peak story shear (middle), and peak overturning moment (right) at scale factor 2.0.

2. For the 4-story frame, second and third mode contributions to floor displacement and story drift were negligible, and reasonably accurate estimates of these quantities could be obtained using first mode response alone. Accuracy of story shear estimates improved as the scale factor increased from 0.5 to 2.0, and inclusion of higher modes had a beneficial effect. Estimates of overturning moment were relatively accurate, and inclusion of higher modes had a beneficial effect.
3. For the 8-story frame displacement response, second and third mode contributions to floor displacement were negligible, and reasonably accurate estimates of this quantity could be obtained using first mode response alone. Accuracy of story drift estimates varied with location and scale factor, and inclusion of higher modes had a beneficial effect. Story drift estimates improved in the upper stories, but became less accurate in the lower stories, as the scale factor increased.
4. For the 8-story frame force response, the accuracy of story shear estimates varied with location and scale factor, and inclusion of higher modes had a significant effect. In the lower stories, story shear estimates were most accurate at a scale factor of 0.5, and were significantly overestimated at higher scale factors. In the upper stories, story shears were underestimated at a scale factor of 0.5, were reasonably accurate at a scale factor of 1.0, and were significantly overestimated at a scale factor of 2.0. Higher mode contributions to overturning moment were not negligible and improved the estimates, although overturning moments were

underestimated at a scale factor of 0.5, and were overestimated at a scale factor of 2.0.

Table 5-6 summarizes ratios of response quantity estimates to median nonlinear response history analysis results for Modal Pushover Analysis of reinforced concrete moment frame structures. Values shown represent maximum deviations over the height of the structure. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis. Figure 5-32 shows how the ratios vary over height for 2-, 4-, and 8-story structures at scale factors of 0.5 and 2.0.

Table 5-6 Ratios of Response Quantity Estimates for Modal Pushover Analysis of Reinforced Concrete Moment Frame Structures

		Response Quantity Ratios ¹			
		Scale Factor	Displacement	Story Drift	Story Shear
2-story	0.5	0.9 to 0.9	0.9 to 0.9	0.9 to 0.9	0.9 to 0.9
	1.0	1.1 to 1.1	1.0 to 1.1	0.9 to 1.0	0.9 to 1.0
	2.0	1.1 to 1.1	1.1 to 1.1	0.8 to 0.9	0.8 to 1.0
4-story	0.5	1.0 to 1.0	0.9 to 1.0	0.8 to 1.0	0.8 to 1.0
	1.0	1.1 to 1.2	0.9 to 1.2	0.9 to 1.0	0.9 to 1.0
	2.0	1.1 to 1.2	0.8 to 1.2	0.9 to 1.0	0.9 to 1.0
8-story	0.5	1.0 to 1.1	0.8 to 1.0	0.8 to 1.0	0.8 to 0.9
	1.0	1.1 to 1.2	0.7 to 1.3	0.8 to 1.1	0.8 to 1.0
	2.0	1.2 to 1.5	0.8 to 1.5	0.9 to 1.1	1.0 to 1.2

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

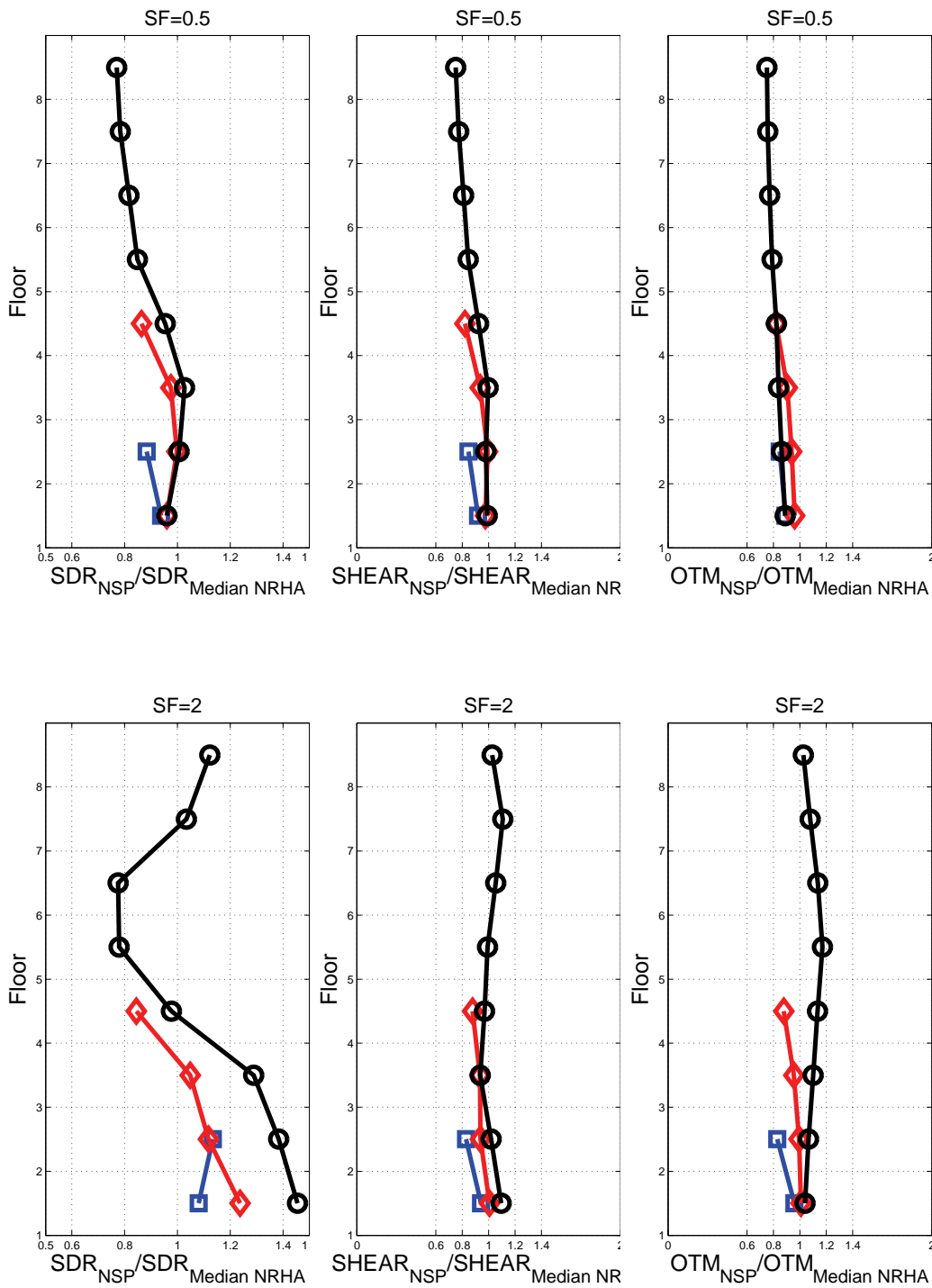


Figure 5-32 Ratios of response quantity estimates to median nonlinear response history analysis results over height for Modal Pushover Analysis of 2-, 4-, and 8-story reinforced concrete moment frame structures for story drift ratio (left), story shear (middle), and overturning moment (right) at scale factors 0.5 and 2.0.

Consecutive Modal Pushover

Selected comparisons between nonlinear response history analysis results (median, mean, and dispersion) and results from Consecutive Modal Pushover of 2-, 4-, and 8-story reinforced concrete moment frames, are shown in Figures 5-33 through 5-35. Selected results are shown for scale factors 0.5 and 2.0. More complete results are presented in Appendix B.

Comparisons between Consecutive Modal Pushover analysis and nonlinear response history analysis results for reinforced concrete moment frame structures led to the following observations:

1. For the 2-story frame, peak displacements and story drifts were estimated with reasonable accuracy at all scale factors. Story shears were significantly overestimated at a scale factor of 0.5, reasonably accurate at a scale factor of 1.0, and significantly underestimated at a scale factor of 2.0. Overturning moments tended to be underestimated.

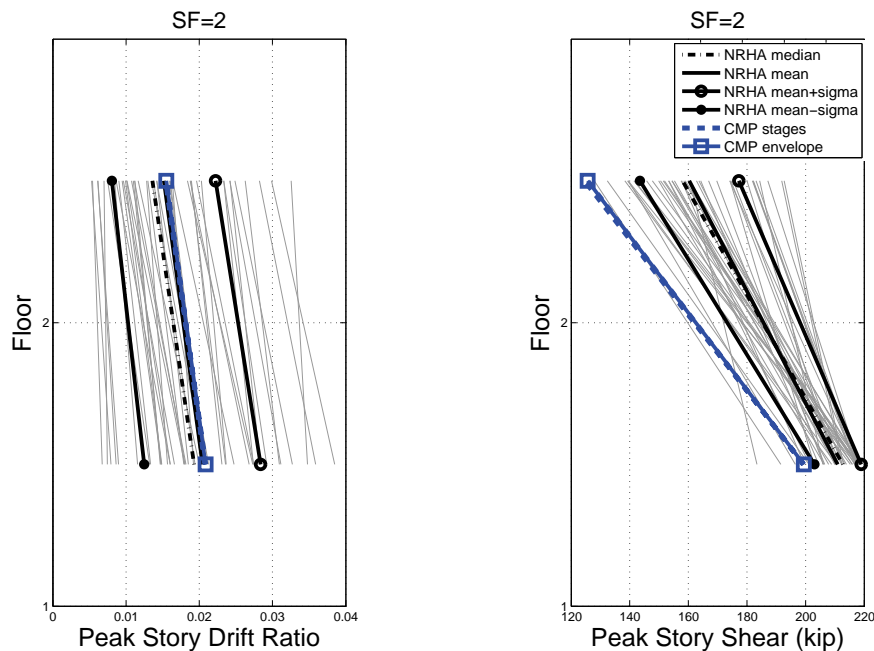


Figure 5-33 Comparison of nonlinear response history analysis to Consecutive Modal Pushover analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factor 2.0.

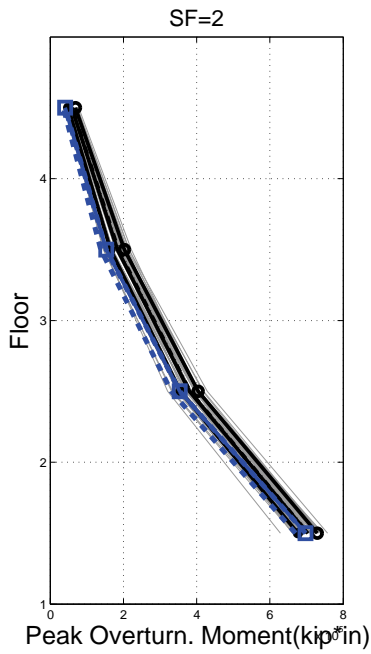
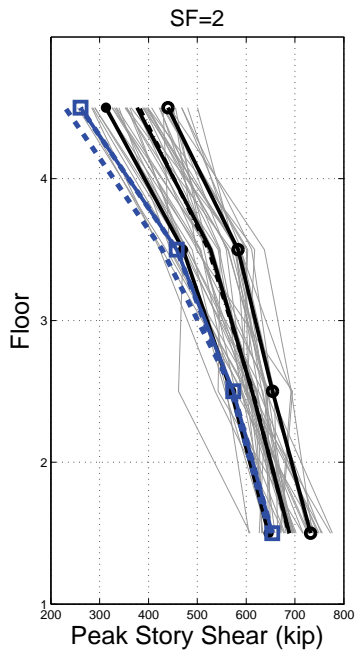
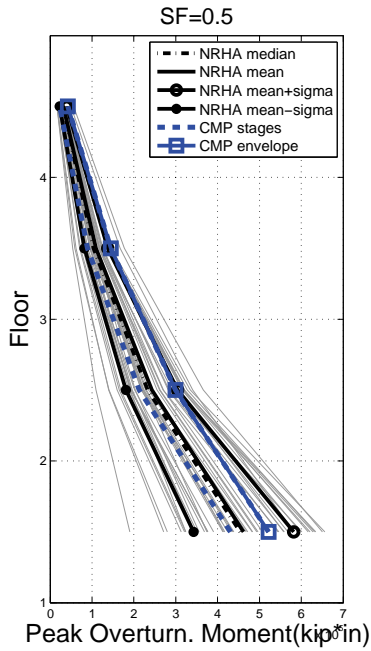
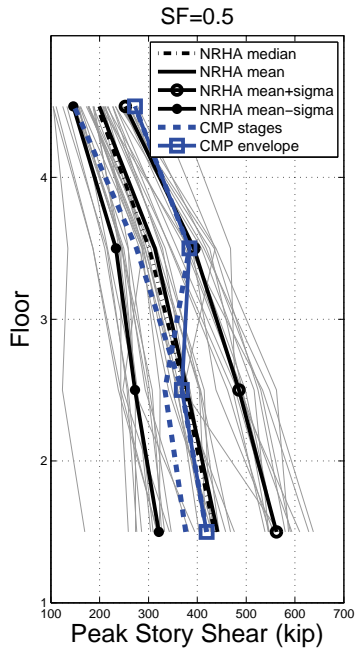


Figure 5-34 Comparison of nonlinear response history analysis to Consecutive Modal Pushover analysis of the 4-story reinforced concrete moment frame for peak story shear (left) and peak overturning moment (right) at scale factors 0.5 and 2.0.

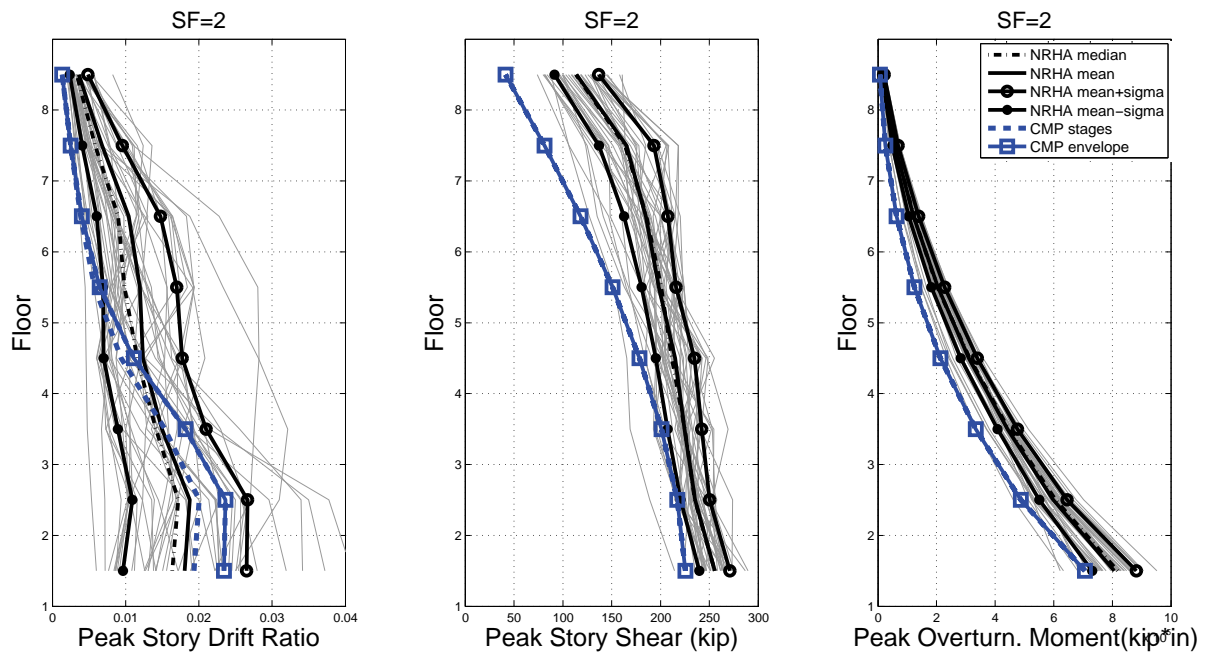


Figure 5-35 Comparison of nonlinear response history analysis to Consecutive Modal Pushover analysis of the 8-story reinforced concrete moment frame for peak story drift ratio (left), peak story shear (middle), and peak overturning moment (right) at scale factor 2.0.

2. For the 4-story frame, peak displacements were estimated with reasonable accuracy at all scale factors. The accuracy of story drift, story shear, and floor overturning moment estimates varied with location and scale factor. Story drifts in the upper stories were overestimated at a scale factor of 0.5, but reasonably accurate estimates of story drift were obtained over the height of the building at higher scale factors. Story shears in the upper stories were significantly overestimated at a scale factor of 0.5, while story shears over the height of the building were significantly underestimated at a scale factor of 2.0.
3. For the 8-story frame, peak displacements were overestimated at a scale factor of 2.0. The accuracy of story drift, story shear, and floor overturning moment estimates varied with location and scale factor.

Table 5-7 summarizes ratios of response quantity estimates to median nonlinear response history analysis results for Consecutive Modal Pushover analysis of reinforced concrete moment frame structures. Figure 5-36 shows how the ratios vary over height for 2-, 4-, and 8-story structures at scale factors of 0.5 and 2.0.

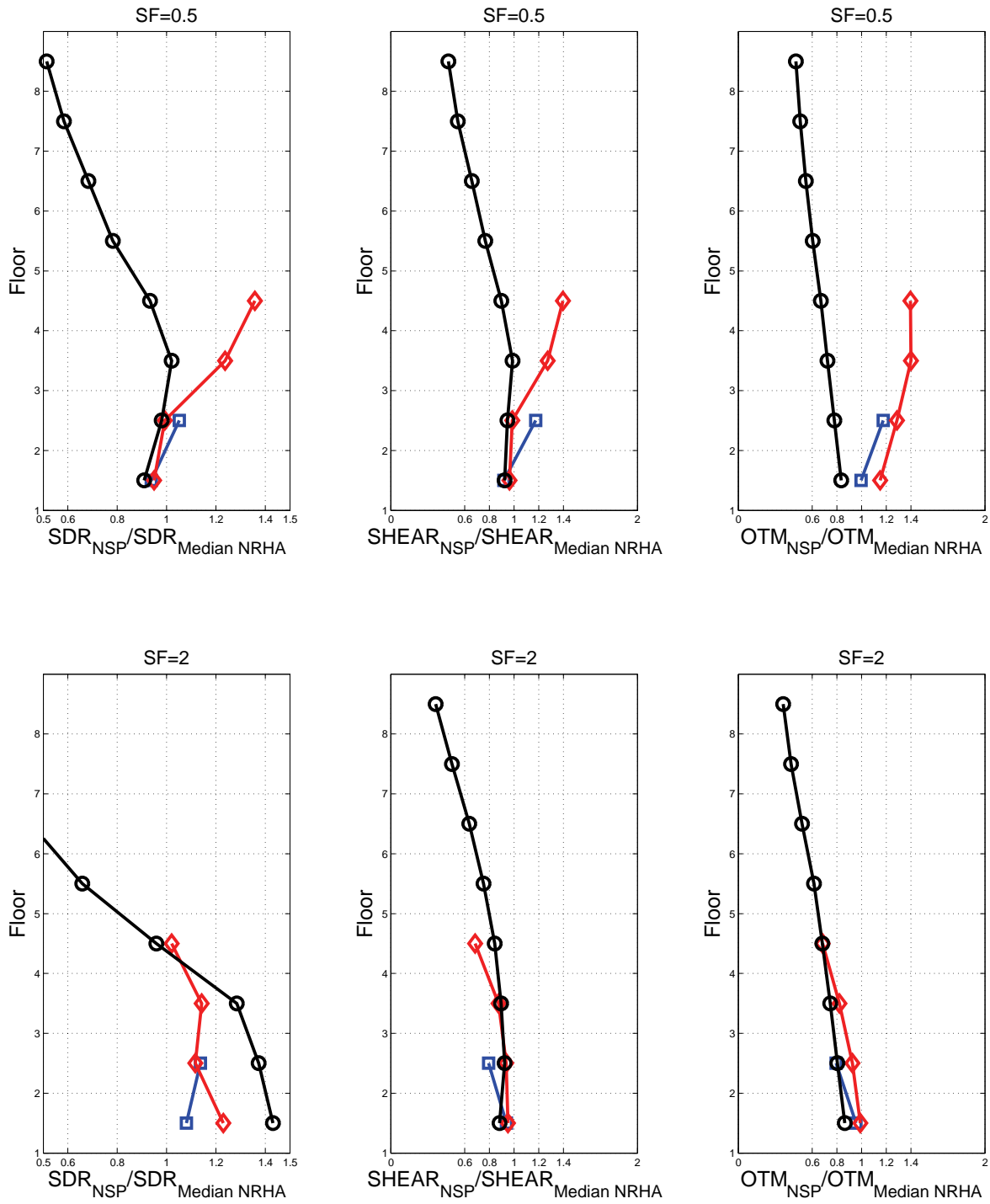


Figure 5-36 Ratios of response quantity estimates to median nonlinear response history analysis results over height for Consecutive Modal Pushover analysis of 2-, 4-, and 8-story reinforced concrete moment frame structures for story drift ratio (left), story shear (middle), and overturning moment (right) at scale factors 0.5 and 2.0.

Table 5-7 Ratios of Response Quantity Estimates for Consecutive Modal Pushover Analysis of Reinforced Concrete Moment Frame Structures

		Response Quantity Ratios ¹				
		Scale Factor	Displacement	Story Drift	Story Shear	Overturning Moment
2-story	0.5	0.9 to 0.9	0.9 to 1.1	0.9 to 1.2	1.0 to 1.2	
	1.0	1.1 to 1.1	1.1 to 1.1	1.0 to 1.0	1.0 to 1.0	
	2.0	1.1 to 1.1	1.1 to 1.1	0.8 to 1.0	0.8 to 1.0	
4-story	0.5	1.0 to 1.0	1.0 to 1.4	1.0 to 1.4	1.2 to 1.4	
	1.0	1.1 to 1.2	1.1 to 1.2	0.9 to 1.0	0.9 to 1.0	
	2.0	1.1 to 1.2	1.0 to 1.2	0.7 to 1.0	0.7 to 1.0	
8-story	0.5	0.9 to 1.1	0.5 to 1.0	0.5 to 1.0	0.5 to 0.8	
	1.0	1.1 to 1.2	0.5 to 1.3	0.4 to 1.0	0.4 to 0.9	
	2.0	1.2 to 1.5	0.4 to 1.4	0.4 to 0.9	0.4 to 0.9	

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

5.3.5 Elastic Modal Response Spectrum Analysis

Elastic modal response spectrum analysis was tested as an alternative to nonlinear static analysis for predication of demand parameters on the 2-, 4-, and 8-story reinforced concrete moment frame structures. Table 5-8 summarizes ratios of response quantity estimates to median nonlinear response history analysis results.

Table 5-8 Ratios of Response Quantity Estimates for Elastic Modal Response Spectrum Analysis of Reinforced Concrete Moment Frame Structures

		Response Quantity Ratios ¹				
		Scale Factor	Displacement	Story Drift	Story Shear	Overturning Moment
2-story	0.5	0.9 to 0.9	0.9 to 0.9	0.9 to 0.9	0.9 to 0.9	0.9 to 0.9
	1.0	1.1 to 1.1	1.1 to 1.1	1.3 to 1.5	1.3 to 1.4	
	2.0	1.0 to 1.1	1.0 to 1.1	2.3 to 2.7	2.3 to 2.7	
4-story	0.5	1.0 to 1.0	0.9 to 1.0	0.8 to 1.0	0.8 to 1.0	
	1.0	1.1 to 1.1	1.0 to 1.1	1.1 to 1.4	1.1 to 1.3	
	2.0	1.0 to 1.1	1.0 to 1.2	1.7 to 2.5	1.7 to 2.5	
8-story	0.5	1.0 to 1.1	0.8 to 1.0	0.8 to 1.0	0.8 to 0.9	
	1.0	1.1 to 1.1	0.9 to 1.1	1.0 to 1.4	1.0 to 1.3	
	2.0	1.0 to 1.3	1.0 to 1.6	1.6 to 2.6	1.6 to 2.5	

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

Comparisons between nonlinear response history analysis results (median, mean, and dispersion) and results from elastic modal response spectrum analysis of 2-, 4-, and 8-story reinforced concrete moment frames, are shown in Figures 5-37 through 5-39.

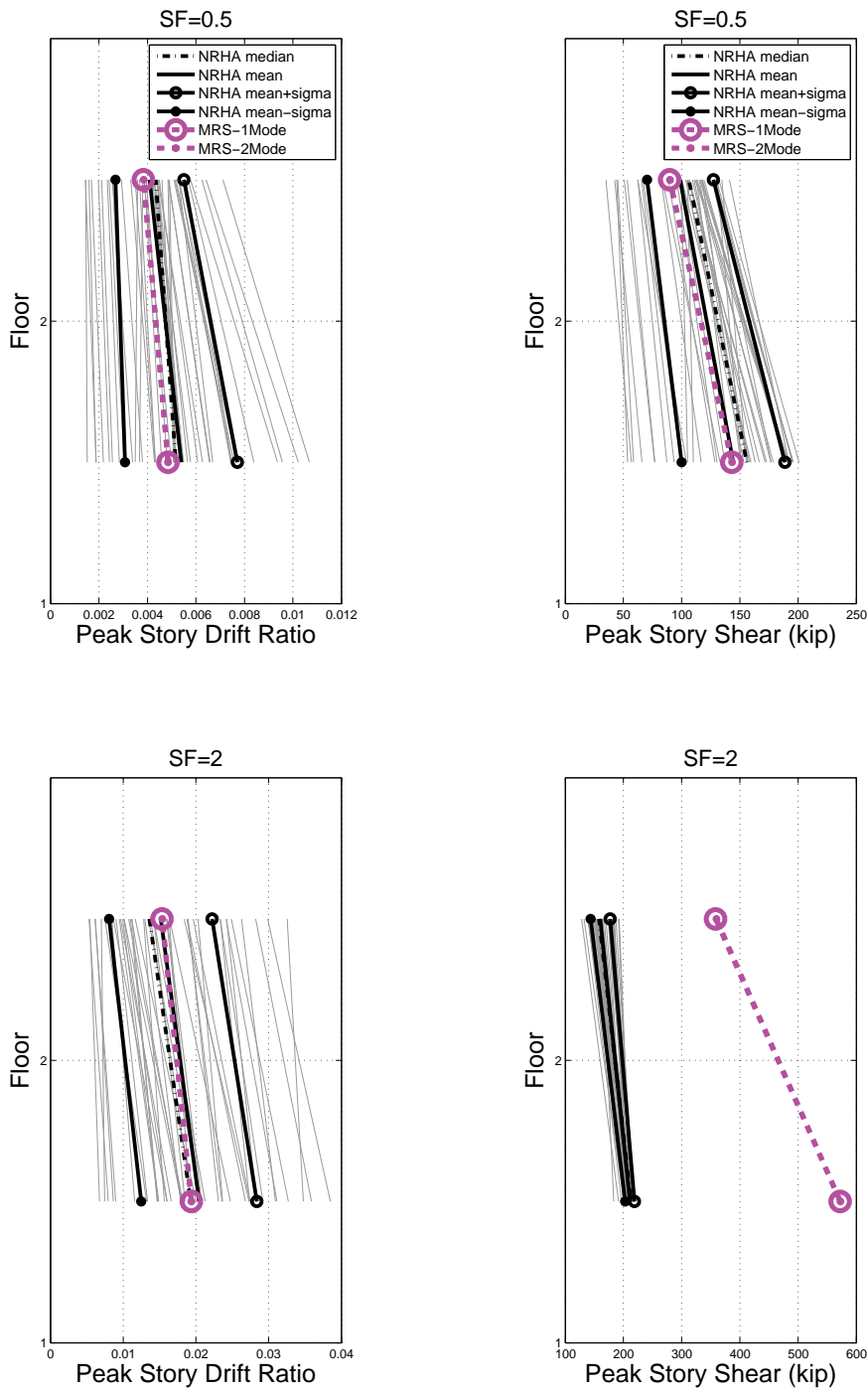


Figure 5-37 Comparison of nonlinear response history analysis to elastic modal response spectrum analysis of the 2-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0.

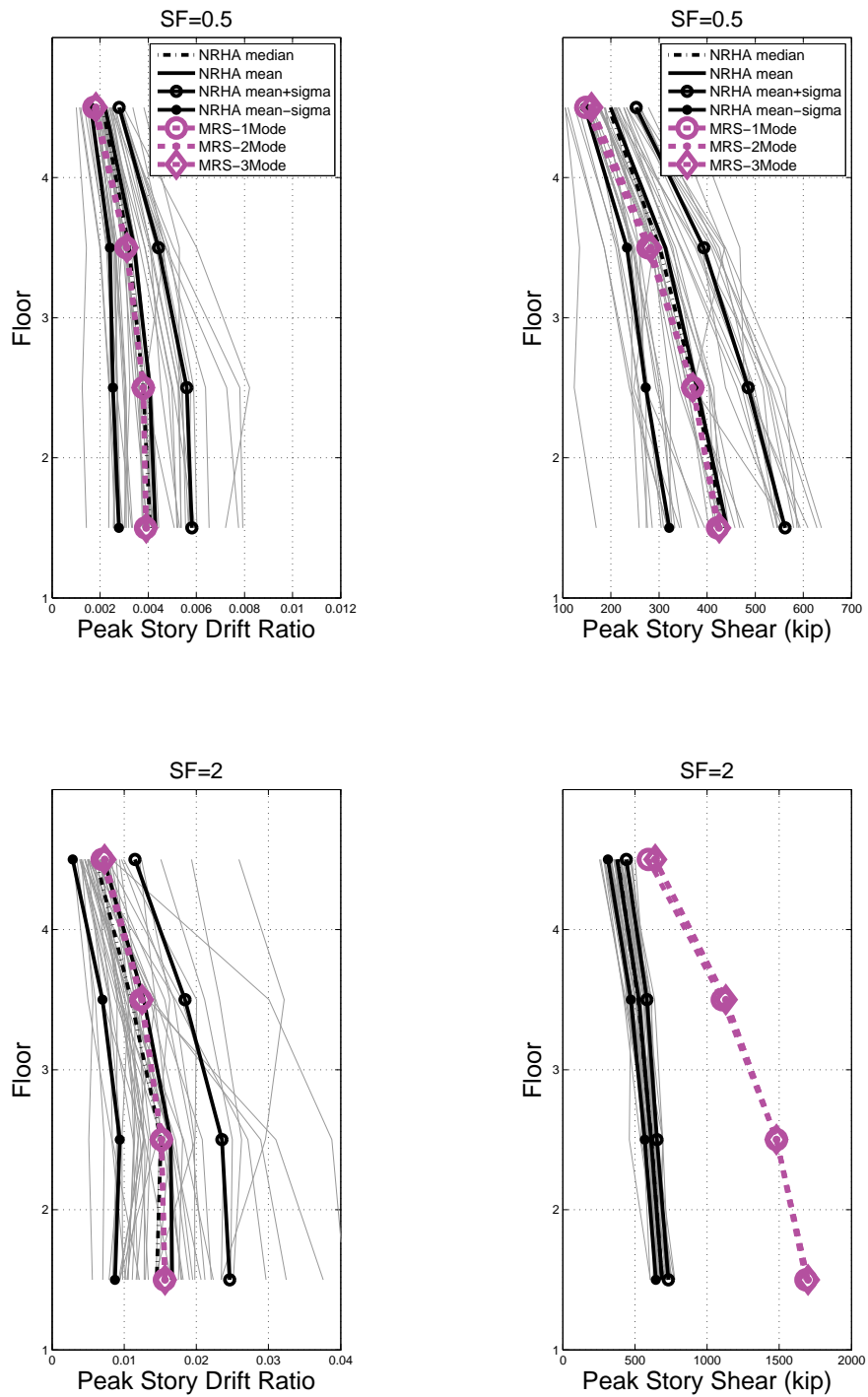


Figure 5-38 Comparison of nonlinear response history analysis to elastic modal response spectrum analysis of the 4-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0.

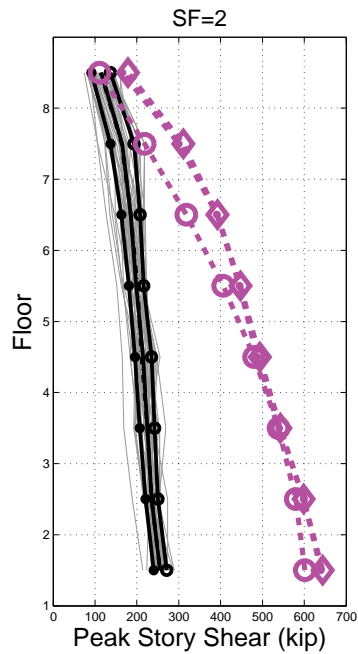
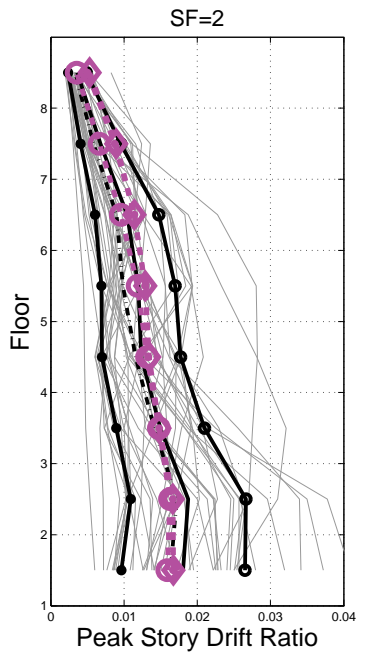
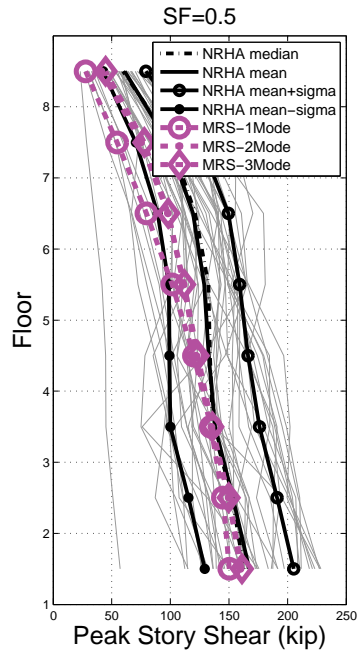
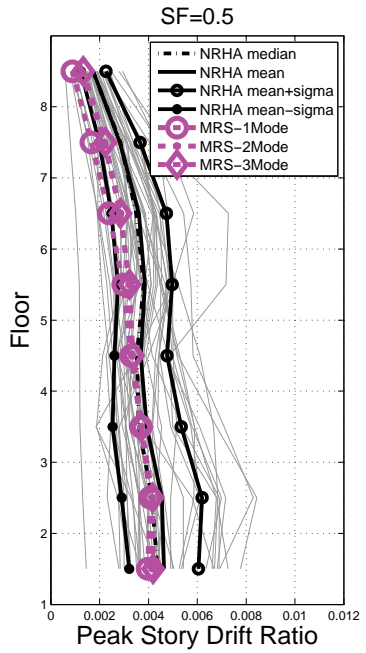


Figure 5-39 Comparison of nonlinear response history analysis to elastic modal response spectrum analysis of the 8-story reinforced concrete moment frame for peak story drift ratio (left) and peak story shear (right) at scale factors 0.5 and 2.0.

The following summary observations are made regarding elastic modal response spectrum analysis of reinforced concrete moment frame structures:

1. In general, drift profiles over height are proportional to the elastic distributions. In cases where drift patterns obtained in modal pushover analyses resemble elastic distributions, displacement and story drift estimates from elastic modal response spectrum analysis will resemble those obtained using modal pushover analysis. Story shear and overturning moment distributions at low scale factors (elastic or near-elastic response) will also resemble those obtained using modal pushover analysis. At high scale factors, force-related quantities are significantly overestimated in elastic analyses.
2. For the 2-story frame, peak displacement and story drift estimates were reasonably accurate. While story shears and overturning moments were estimated with reasonable accuracy at a scale factor of 0.5, the assumption of elastic response in every mode led to overestimation of story shears and overturning moments at higher scale factors.
3. For the 4-story frame, floor displacement and story drift estimates are reasonably accurate at all scale factors. Story shears are underestimated in the upper stories at a scale factor of 0.5 and overestimated at scale factors of 1.0 and 2.0. Overturning moments are slightly underestimated in the lower stories at a scale factor of 0.5 and overestimated at scale factors of 1.0 and 2.0.
4. For the 8-story frame, peak displacement estimates are reasonably accurate at a scale factor of 0.5 and are overestimated at a scale factor of 2.0. Story drifts are underestimated at a scale factor of 0.5, reasonably accurate at a scale factor of 1.0, and overestimated in the upper stories at a scale factor of 2.0. Story shears and overturning moments were generally underestimated at a scale factor of 0.5, and generally overestimated at scale factors of 1.0 and 2.0.

5.4 Reinforced Concrete Shear Wall Studies

This section summarizes results of studies on 2-, 4-, and 8-story reinforced concrete shear wall structures. Nonlinear response history analysis is performed using the FEMA P-695 far-field ground motion set, and various options of single-mode nonlinear static analysis and modal pushover analysis procedures are evaluated for their ability to predict peak values of story drift ratio, story shear force, and floor overturning moment. Detailed results are presented in Appendix C.

5.4.1 Reinforced Concrete Shear Wall Prototypes

Reinforced concrete shear wall (RCSW) prototypes consist of a subset of the reinforced concrete special shear wall archetypes designed and analyzed in the NIST GCR 10-917-8 report, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors* (NIST, 2010b). The subset

consists of three structures designed for seismic design category D_{max} ($S_{DS} = 1.0g$ and $S_{DI} = 0.60g$), designated as follows:

- 2-story RCSW (archetype ID 12)
- 4-story RCSW (archetype ID 13)
- 8-story RCSW (archetype ID 14)

Structures consisted of a rectangular plan configuration with two cantilever walls resisting lateral forces in each direction, as shown in Figure 5-40. All archetypes are special shear walls within building frame systems ($R = 6$), detailed in accordance with ACI 318-08, *Building Code Requirements for Structural Concrete* (ACI, 2008). Only rectangular walls without openings have been considered in this study.

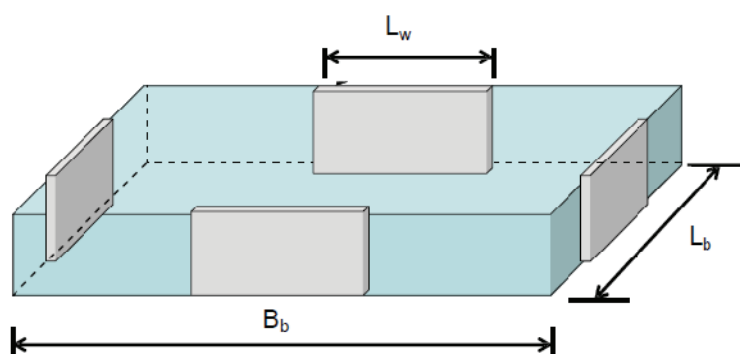


Figure 5-40 Configuration of reinforced concrete shear wall prototypes (NIST, 2010).

Nonlinear component models for shear wall elements consisted of fiber models (FM) and simplified spring models (SM). Fiber elements were used to model flexural behavior, and shear behavior was modeled with a translational spring per story. Fiber models result in behavior modes that are believed to account for localized failure such as concrete crushing and steel buckling and fracture. Results using fiber models are sensitive to assumptions made in analytical modeling and are sometimes difficult to explain with concepts that are used customarily in engineering practice.

For this reason simplified models were developed in which bending behavior was modeled with rotational springs. Use of simplified spring models permits explicit consideration of post-yield and post-capping behavior, using modeling parameters similar to those used for Analyt.M1 and Analyt.M3.

Plots showing nonlinear response history analysis results using fiber models and simplified spring models are presented in Figure 5-41 and Figure 5-42.

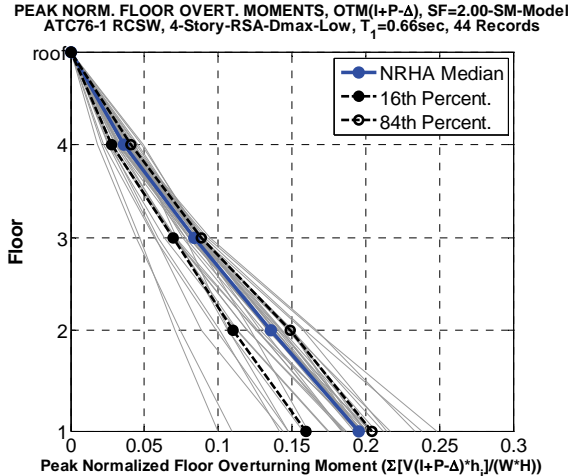
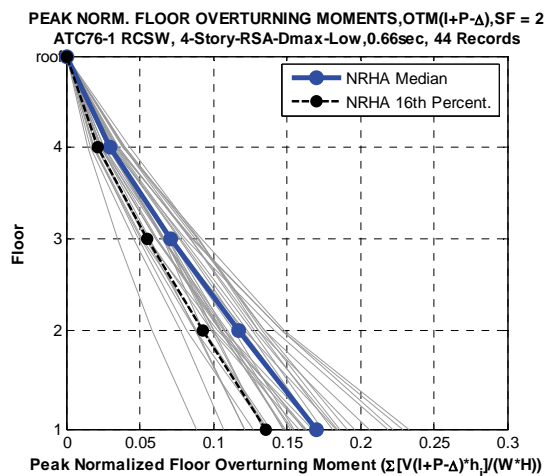
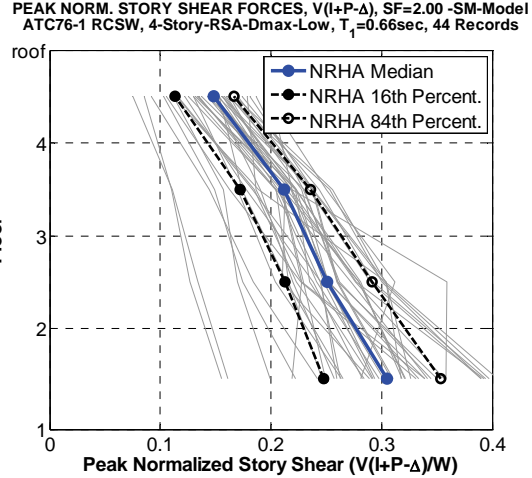
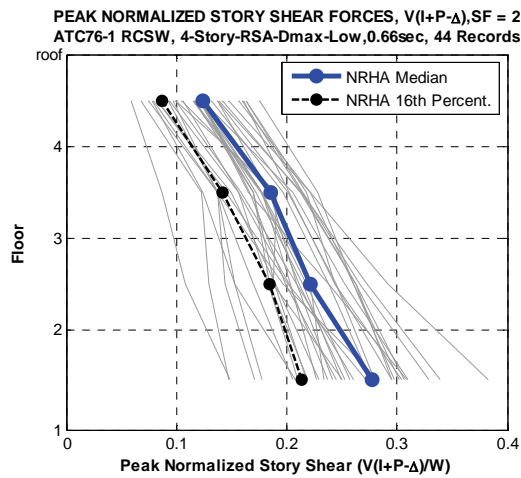
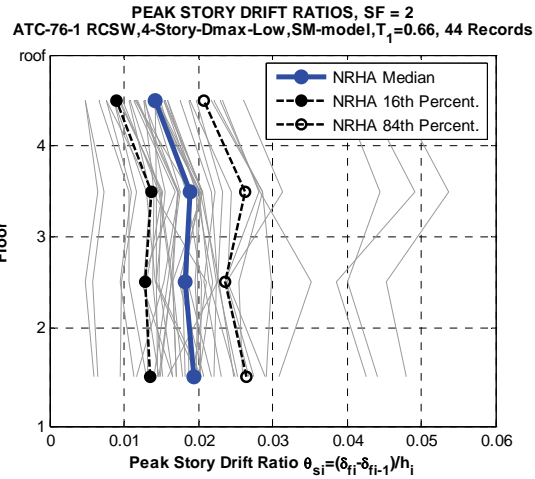
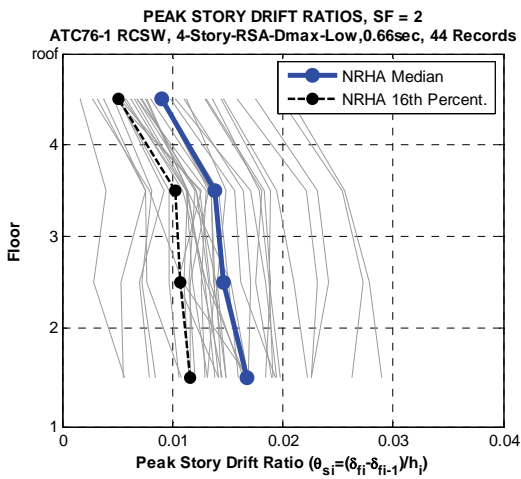


Figure 5-41 Peak story drift ratios, story shears, and floor overturning moments for the 4-story reinforced concrete shear wall structure using fiber models (left) and simplified spring models (right), with ground motion scale factor 2.0.

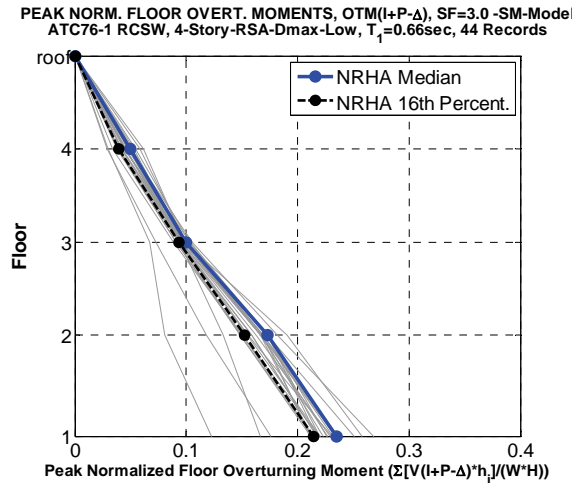
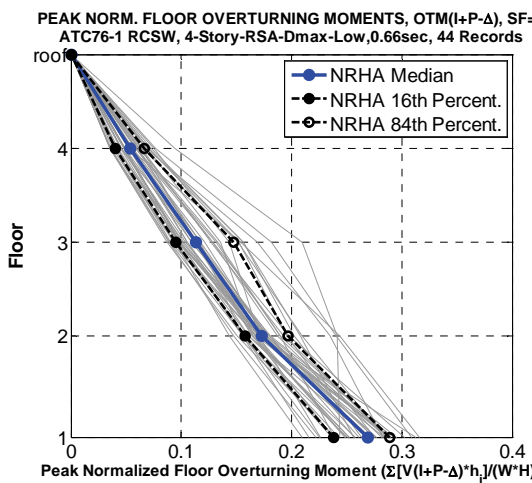
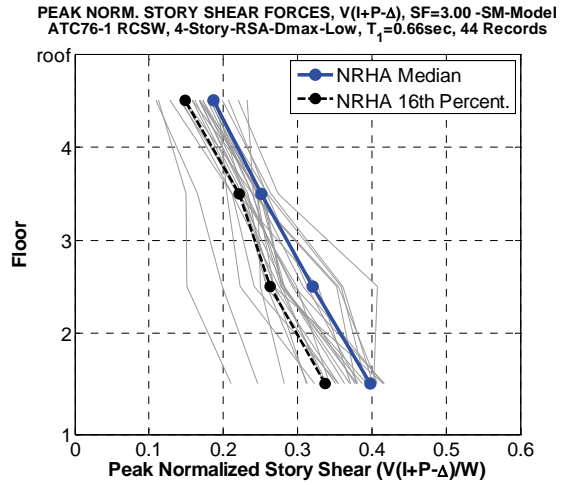
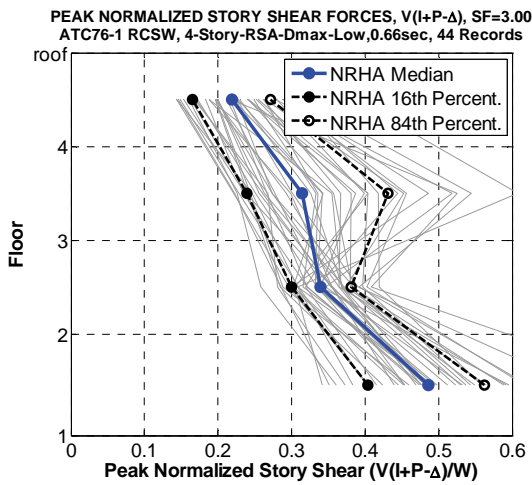
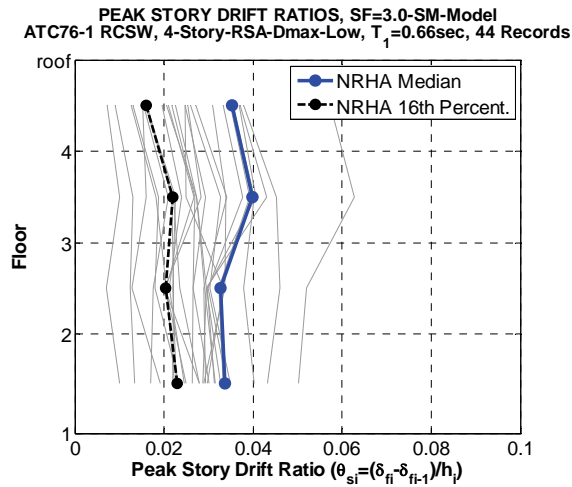
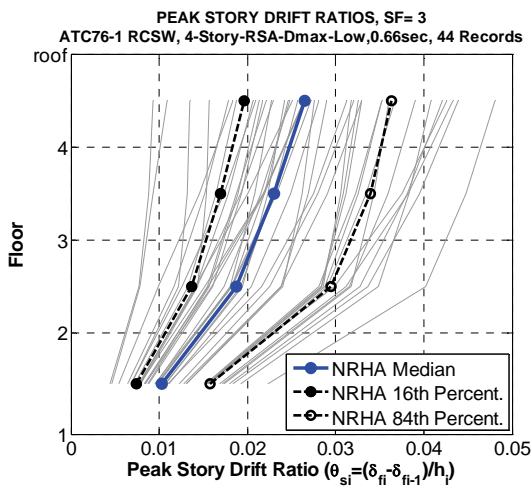


Figure 5-42 Peak story drift ratios, story shears, and floor overturning moments for the 4-story reinforced concrete shear wall structure using fiber models (left) and simplified spring models (right), with ground motion scale factor 3.0.

5.4.2 *Nonlinear Response History Analysis*

Nonlinear response history analysis of reinforced concrete shear wall structures led to the following observations regarding prediction of response quantities:

1. Fiber models provide information that cannot be obtained from simplified spring models, but it is difficult to establish the reliability and to assess the soundness of results obtained from fiber model analysis alone.
2. Simple models consisting of elastic elements and flexural as well as shear springs may be adequate to capture important behavior modes for regular shear walls without openings. Stiffness and strength properties up to yield can be captured adequately with spring models. Post-yield deformation capacities (e.g., plastic deformation before strength deterioration and post-capping negative tangent stiffness) control behavior of shear wall structures close to collapse but are difficult to quantify with currently available experimental data.
3. For a scale factor of 2.0, simplified models compared well with fiber models. Response quantities were about 10% to 20% larger for the simplified model, but the patterns were the same. This provides confidence that, given comparable component strength and deformation properties, both models predict consistent seismic demands for ground motion intensities that cause moderate nonlinearity.
4. For a scale factor of 3.0, significant differences were observed between fiber models and simplified models for all demand parameters. In particular, median base shear results for the fiber model significantly exceed the shear strength of walls. This is because the strength of the shear springs was not capped in the fiber model, assuming that the wall elements will always yield in bending. This observation highlights the importance of providing capping strengths for all possible yielding mechanisms because yielding in a dynamic environment may be different from what might be assumed based on static design assumptions.
5. The dispersion in response is largest for story drift ratios, smaller for story shears, and smallest for floor overturning moments. The smaller dispersion in story shears and floor overturning moments comes from saturation of strength capacities (bending and/or shear) as the structure yields.
6. Although yielding causes saturation of force quantities, dispersion occurs, even at very large scale factors because of dynamic redistribution once plastification occurs at a specific location.
7. Peak story shear demands and floor overturning moments obtained from nonlinear response history analysis resemble a pattern caused by uniform story forces. Neither pattern resembles a first mode load pattern.

5.4.3 Single-Mode Nonlinear Static Analysis

Variations in single-mode nonlinear static analysis included the use of different component models (ASCE41 and FM models) and methods of predicting the target displacement. Target displacements were estimated using the coefficient method (ASCE41) and the equivalent SDOF method (EqSDOF). In all cases an invariant first mode load pattern was used to determine the pushover response.

Global pushover curves for the 2-, 4-, and 8-story shear wall structures using the fiber model are shown in Figure 5-43. Also shown are target displacements obtained using the coefficient method (ASCE41) for scale factors of SF=0.5, 1.0, and 2.0. A ground motion scale factor of 2.0 is not sufficient to cause severe strength or stiffness deterioration in any of the systems modeled using fiber elements. Comparisons between nonlinear response history analysis results (median and dispersion) and results from various single-mode nonlinear static analysis options are shown in Figure 5-44.

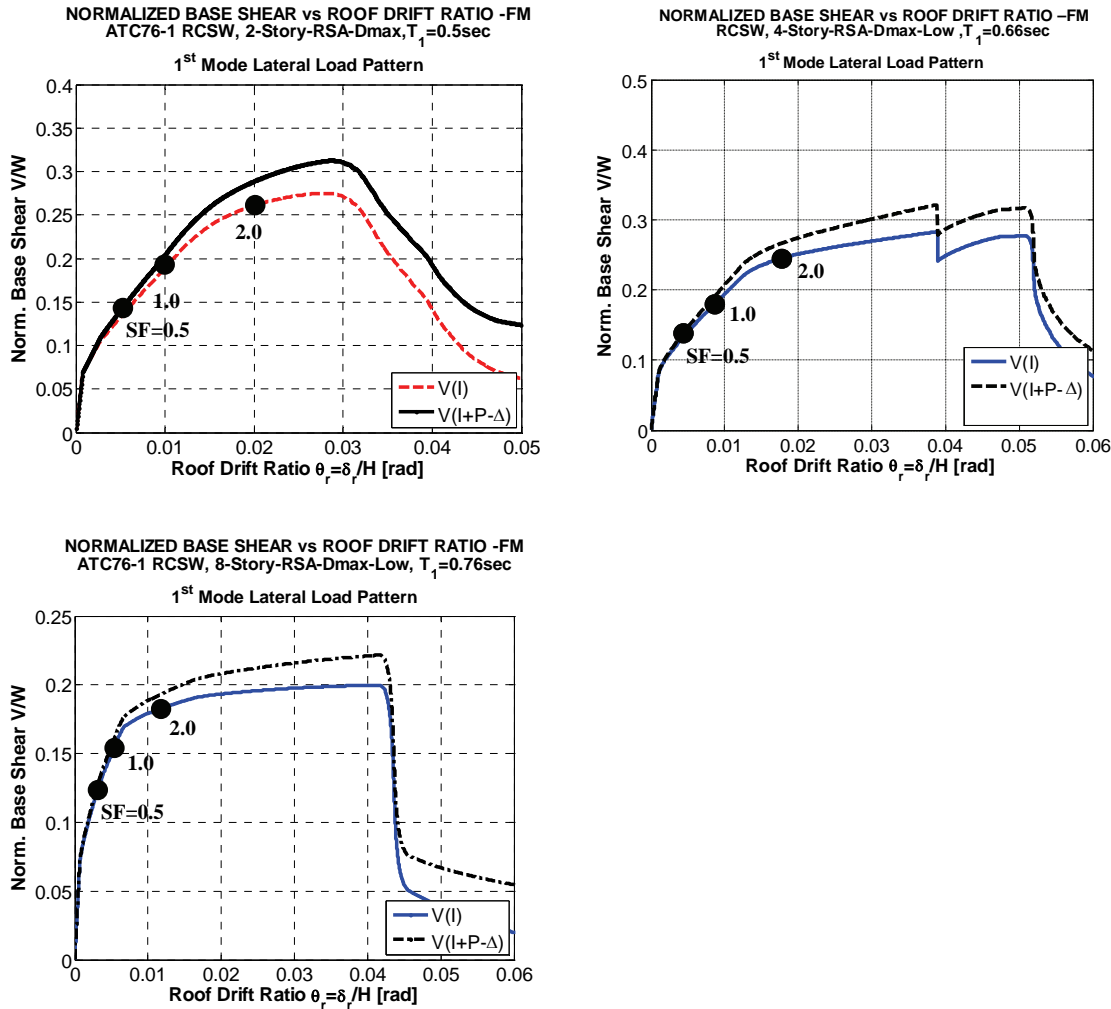


Figure 5-43 Comparison of global pushover curves for fiber models of 2-, 4-, and 8-story reinforced concrete shear wall structures showing target displacements for scale factors 0.5, 1.0 and 2.0.

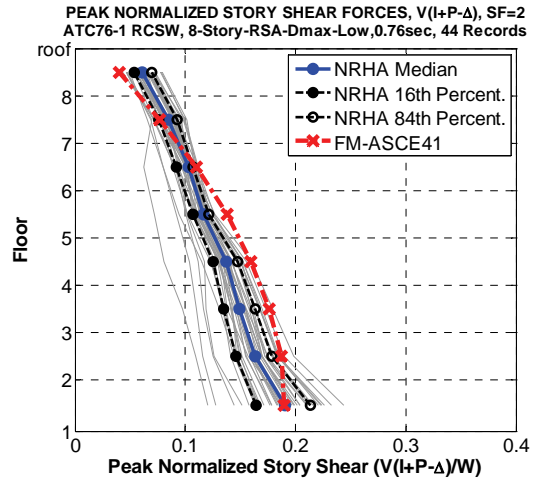
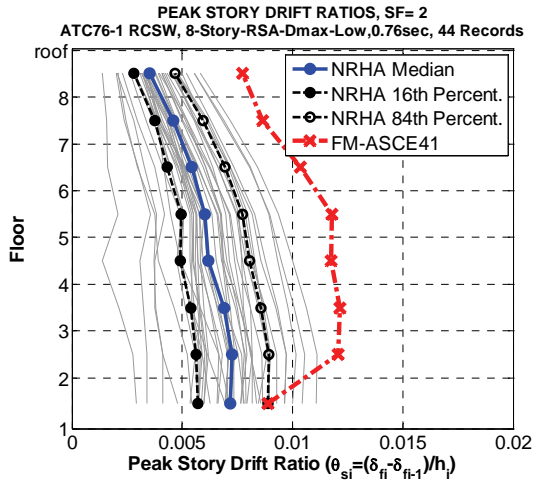
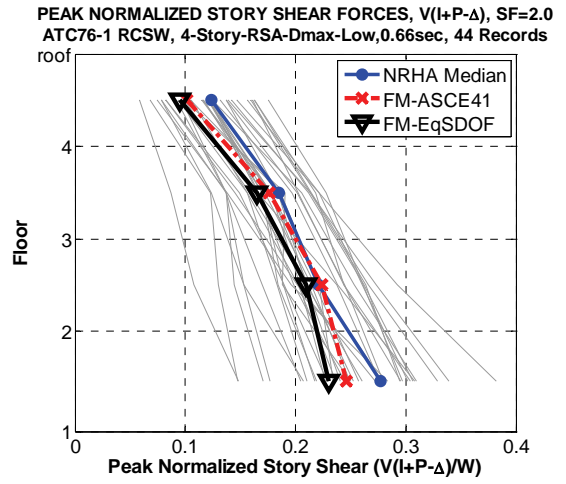
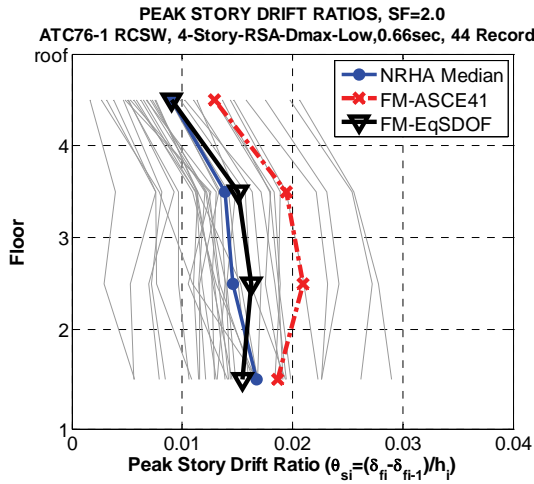
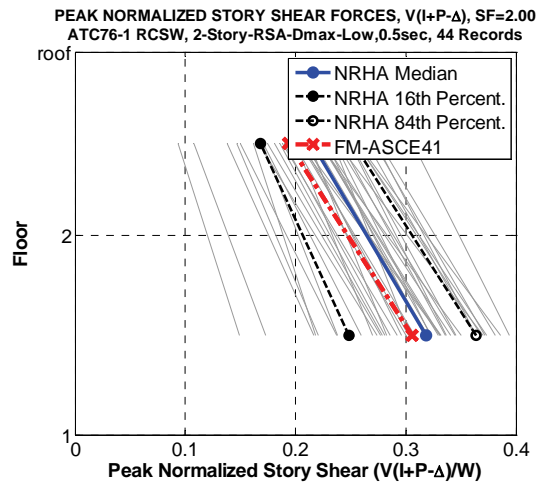
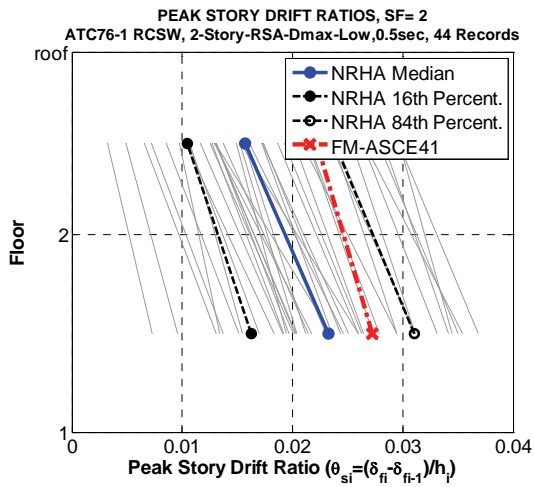


Figure 5-44 Comparison of nonlinear response history analysis to single-mode nonlinear static analysis for the 2-, 4, and 8-story reinforced concrete shear wall structures using fiber models.

General Observations – Single-Mode Nonlinear Static Analysis

Comparisons between single-mode nonlinear static analysis and nonlinear response history analysis results for reinforced concrete shear wall structures led to the observations listed below. Any conclusions from these observations should be qualified by the characteristics of the shear wall prototypes included in the study, which are considered to be representative of regular, low-rise construction without any significant irregularities in strength or stiffness.

1. In the shear wall structures investigated, the yielding mode in the pushover analysis, using the fiber model, was the same as the dominant yielding mode in the nonlinear response history analysis. In general, the dominant yielding mode was bending.
2. In general, single-mode nonlinear static analysis predictions for shear wall structures provided a better match to nonlinear response history analysis results than was the case for steel moment frame structures.

Component Models – Single-Mode Nonlinear Static Analysis

1. All fiber element model pushover curves exhibited a rapid deterioration in strength after the peak strength was attained. This is likely due to the formulation of the fiber model for bending behavior, but further investigation is needed. Such a loss in strength was not apparent in the nonlinear response history analyses.
2. All pushover curves had a large initial stiffness, but this stiffness degraded rapidly at relatively low load levels. In the 4- and 8-story structures, modeling of post-cracking shear behavior in the translational shear springs was mostly responsible for this stiffness deterioration. After “yielding” the stiffness remained close to constant until dropping rapidly due to flexural failure as defined by the material parameters used in the fiber model. This behavior indicates that simplified representation of pushover curves requires a multi-linear diagram.
3. Nonlinear static analysis predictions in the post-capping (negative tangent stiffness) range could not be explored because of the very rapid decrease in strength exhibited by the fiber model pushovers.
4. Nonlinear static analysis results varied significantly for different component models. The global pushover curve for the 4-story shear wall structure using the ASCE41 component model is shown in Figure 5-45. In comparison with the pushover curve for the fiber model (Figure 5-43), this curve indicates significantly less deformation capacity. Use of the ASCE41 component model resulted in intolerable drift ratios in nonlinear static analysis and lateral dynamic instabilities in nonlinear response history analysis at one-third of the ground motion intensities predicted using the fiber model.

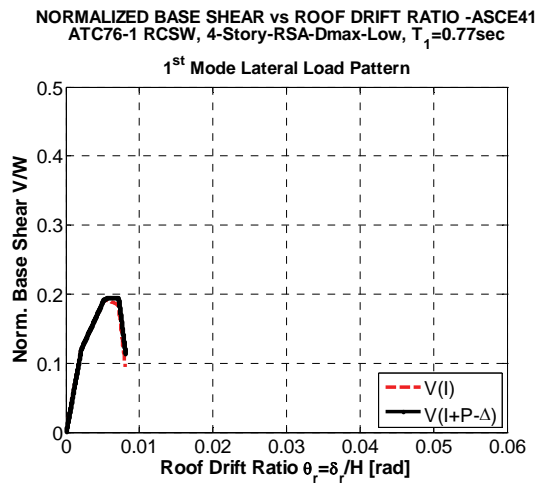


Figure 5-45 Global pushover curve for the 4-story reinforced concrete shear wall structure using ASCE41 component models.

Target Displacements – Single-Mode Nonlinear Static Analysis

1. Use of the ASCE/SEI 41-06 recommendations for idealizing pushover curve with the secant stiffness associated with $0.6V_y$, results in poor target displacement predictions, particularly when the structure is near-elastic. The importance of this idealization diminishes as the degree of nonlinearity increases.
2. The use of nonlinear response history analysis of a multi-linear equivalent SDOF system to predict of target displacement (EqSDOF) leads to considerable improvement in response predictions using single-mode nonlinear static analyses. Such a multi-linear idealization is illustrated in Figure 5-46 (right). This approach is most beneficial for structures in which the pushover curve exhibits a clear stiffness discontinuity due to pre- and post-cracking behavior.

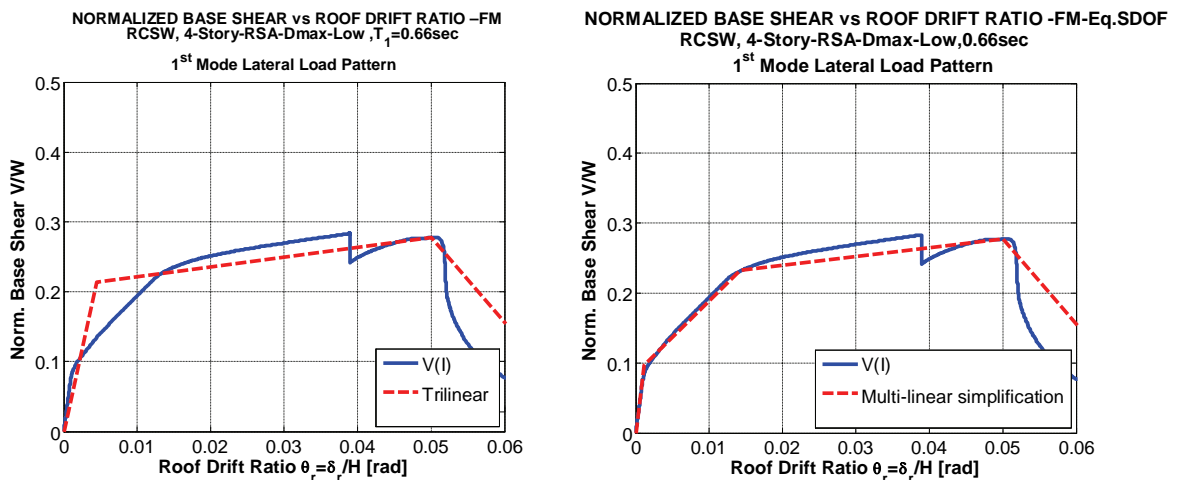


Figure 5-46 Different multi-linear equivalent single-degree-of-freedom approximations to global pushover curves.

- Utilizing the EqSDOF approach for estimation of target displacement improved response quantity predictions over the ASCE41 approach for the 4-story reinforced concrete shear wall structure in Figure 5-44 (middle).

Summary Results – Single-Mode Nonlinear Static Analysis

Table 5-9 summarizes ratios of response quantity estimates to median results from nonlinear response history analysis for reinforced concrete shear wall structures subjected to single-mode nonlinear static analysis. Values shown represent maximum deviations over the height of the structure. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis.

Variation in the ratios of peak response quantity estimates over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story reinforced concrete shear wall structures is shown in Figure 5-47. Results are shown for a scale factor of 2.0. Fiber model results at a scale factor of 3.0 could not be fully explained, so those results have been excluded from the summary observations. Results in Figure 5-47 show the improved prediction of response using the EqSDOF method of target displacement estimation.

Table 5-9 Ratios of Response Quantity Estimates for Single-Mode Nonlinear Static Analysis of Reinforced Concrete Shear Wall Structures

		Response Quantity Ratios ¹		
		Scale Factor	Story Drift	Story Shear
2-story	1.0	1.2 to 1.6	1.1 to 1.2	1.2 to 1.3
	2.0	1.2 to 1.4	0.9 to 1.0	0.9 to 1.1
4-story	1.0	1.4 to 1.8	1.1 to 1.3	1.1 to 1.4
	2.0	1.1 to 1.4	0.8 to 1.1	0.8 to 1.2
4-story (ESDOF)	1.0	0.9 to 1.1	0.9 to 1.0	0.9 to 1.1
	2.0	0.9 to 1.1	0.8 to 0.9	0.8 to 1.0
8-story	1.0	1.5 to 2.0	0.8 to 1.3	0.8 to 1.4
	2.0	1.3 to 2.5	0.7 to 1.2	0.7 to 1.2

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

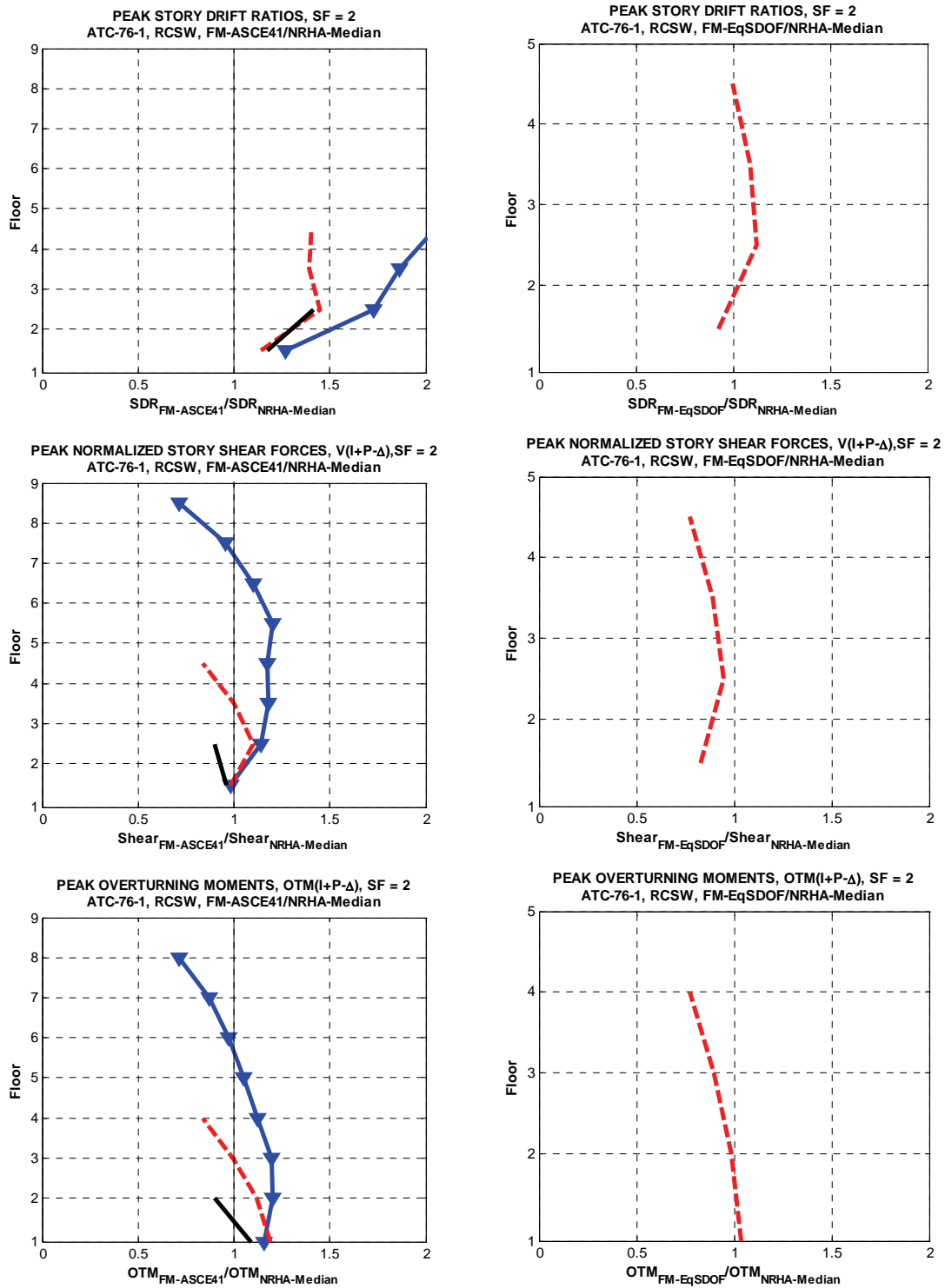


Figure 5-47 Ratios of response quantity estimates to median nonlinear response history analysis results over height for single-mode nonlinear static analysis of 2-, 4-, and 8-story reinforced concrete shear wall structures using different methods of target displacement estimation: ASCE41 (left) and EqSDOF (right), at scale factor 2.0.

5.4.4 Multi-Mode Nonlinear Static Analysis

Reinforced concrete shear wall structures were investigated using the Modal Pushover Analysis (MPA) procedure developed by Chopra and Goel (2001), but without the extension of the method intended to improve estimation of local member forces developed by Chopra and Goel (2004). The MPA procedure, as implemented, follows the more common application among practitioners of determining all response quantities in a single analysis phase.

Table 5-10 summarizes ratios of response quantity estimates to median results from nonlinear response history analysis for reinforced concrete shear wall structures subjected to Modal Pushover Analysis. Values shown represent maximum deviations over the height of the structure. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis.

Selected comparisons between nonlinear response history analysis results (median and dispersion) and results from Modal Pushover Analysis of 4-story and 8-story reinforced concrete shear wall structures are shown in Figure 5-48.

Table 5-10 Ratios of Response Quantity Estimates for Modal Pushover Analysis of Reinforced Concrete Shear Wall Structures

		Response Quantity Ratios ¹		
		Scale Factor	Story Drift	Story Shear
4-story	1.0	1.0 to 1.0	1.0 to 1.2	1.0 to 1.1
	2.0	1.1 to 1.6	1.1 to 1.3	1.1 to 1.8
8-story	1.0	1.0 to 1.3	0.8 to 1.1	0.9 to 1.2
	2.0	1.2 to 1.4	0.9 to 1.2	1.0 to 1.1

¹Values are the minimum and maximum ratios of estimated response quantity to median value from nonlinear response history analysis.

The following summary observations are made regarding Modal Pushover Analysis of reinforced concrete shear wall structures:

1. For the shear wall structures investigated, Modal Pushover Analysis appears to result in reasonably accurate, or conservative, predictions of response.
2. Modal Pushover Analysis predictions are good for low level (near-elastic) response, but the accuracy of predictions decreases as the degree of nonlinearity increases.
3. For the 4-story reinforced concrete shear wall structure, consideration of second mode response is somewhat detrimental (rather than beneficial) for some

response parameters at a scale factor of 2.0, but response predictions, in general, are conservative.

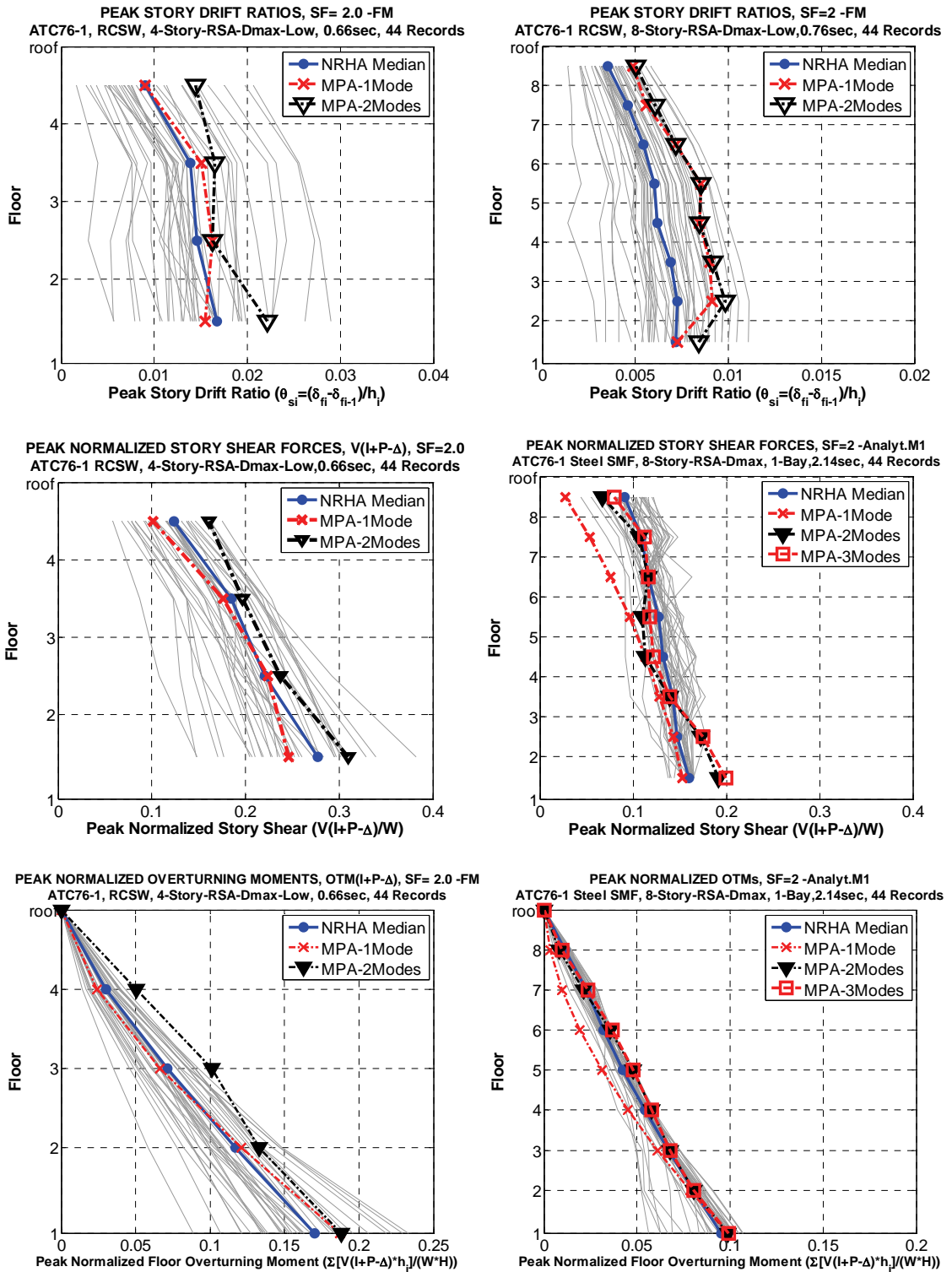


Figure 5-48 Comparison of nonlinear response history analysis results (median and dispersion) to results from Modal Pushover Analysis of the 4-story reinforced concrete shear wall (left) and 8-story reinforced concrete shear wall (right).

4. For the 8-story reinforced concrete shear wall structure, consideration of second mode contributions considerably improved predictions of story shear forces and overturning moments.
5. If the system response is sensitive to the yielding mode of the shear wall (e.g., flexure or shear), it is not clear whether Modal Pushover Analysis will capture changes in yielding that might occur in nonlinear response history analysis.
6. An essential starting point for Modal Pushover Analysis is a reasonable idealization of modal pushover curves for estimation of target displacements. For systems, like shear walls, that exhibit multi-linear characteristics due to pre- and post-cracking behavior, use of a multi-linear idealization as shown in Figure 5-46 (right) is recommended.

5.5 Results for Structural System Prototypes with Special Characteristics

5.5.1 Prototypes with Special Characteristics

Studies of 2-, 4-, and 8-story steel moment frames, reinforced concrete moment frames, and reinforced concrete shear walls described above were supplemented with analyses of other systems to expand the coverage provided by the basic systems that were selected, and to further investigate specific issues. The following additional structural systems were considered:

- 2-story steel single-bay “shear building” with load-pattern-sensitive story mechanisms (Appendix A)
- 4-story irregular (weak-story) steel single-bay “shear building” (Appendix A)
- 4-story reinforced concrete shear wall simplified model designed to fail in bending (Appendix C)

5.5.2 Response Sensitivity to Load Patterns and Strength Irregularities

Single bay steel “shear building” models were used to investigate response sensitivity to load patterns and strength irregularities and to illustrate a limitation on nonlinear static procedures that rely on an invariant load pattern. In shear building models, only the columns contribute to lateral flexibility. Beams are considered to be rigid and infinite in strength.

Figure 5-49 shows analytical results for two 2-story shear building systems. The difference between the two cases is an 8% change in the shear strength of the first story. While the nonlinear response history results are nearly identical in both cases, the pushover analysis results indicate a weak story irregularity that is highly sensitive to a minimal change in story strength.

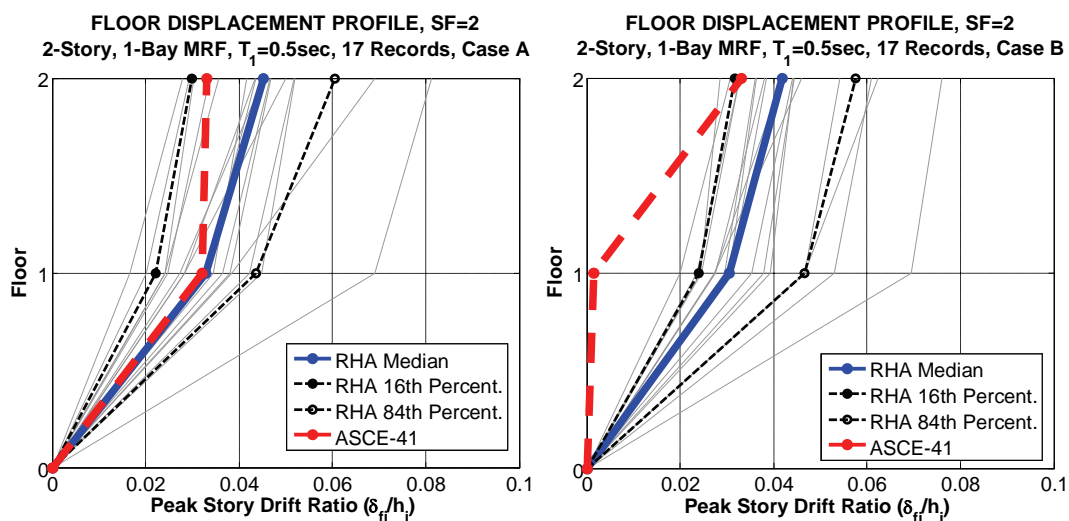


Figure 5-49 Comparison of pushover and nonlinear response history analysis results for a 2-story steel shear building with slightly different story shear capacities.

Two 4-story steel shear buildings were similarly investigated. In the “base case,” story shear strengths were tuned to the lateral load pattern. In the “irregular case,” a first-story strength irregularity was created by increasing the story shear strength above the first story by 50%. Results are presented in Figure 5-50 and Figure 5-51.

Figure 5-50 shows that the pushover curves and deflection profiles for the two structures were nearly identical. In both cases, the invariant load pattern detected yielding in the first story.

Figure 5-51 shows that story drift, story shear, and floor overturning moment predictions from single-mode nonlinear static analyses were also almost identical in both cases while large differences existed in the nonlinear response history analysis results. Median NHRA predictions of story drift ratio clearly show the weak-story behavior in the first story of the “irregular case.”

Summary observations on load pattern sensitivity and strength irregularities are as follows:

1. Pushover behavior of regular structures is sensitive to the selected load pattern because little or no redistribution of inelastic deformation between stories can take place. An invariant load pattern will dictate where the pushover will “detect” a weak story, whether or not such a weakness exists. This type of load pattern sensitivity is not resolved with the use of Modal Pushover Analysis.
2. If a pushover analysis indicates a concentration of drift in a single story, principles of mechanics should be employed to assess whether such an irregularity indeed exists.

3. If an irregularity exists, then story drift predictions obtained from a single-mode nonlinear static analysis are likely to be reasonable in the weak story but might be considerably off in other stories.
4. If irregularities exist in more than one story, it is unlikely that an invariant load pattern will be able to detect more than one irregularity. In such cases, nonlinear response history analysis is recommended.

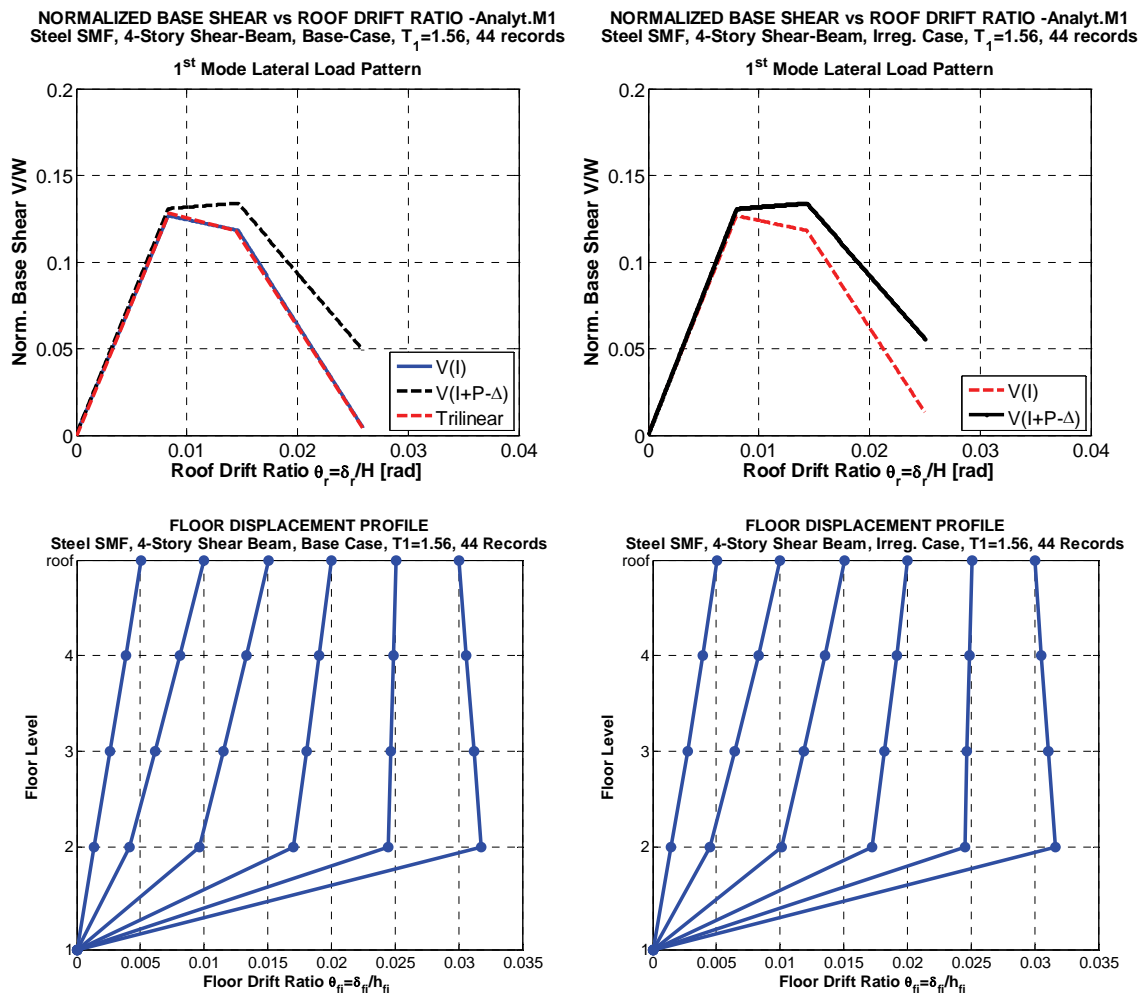


Figure 5-50 Pushover curves and deflection profiles for a 4-story steel shear building: base case (left) and irregular case (right).

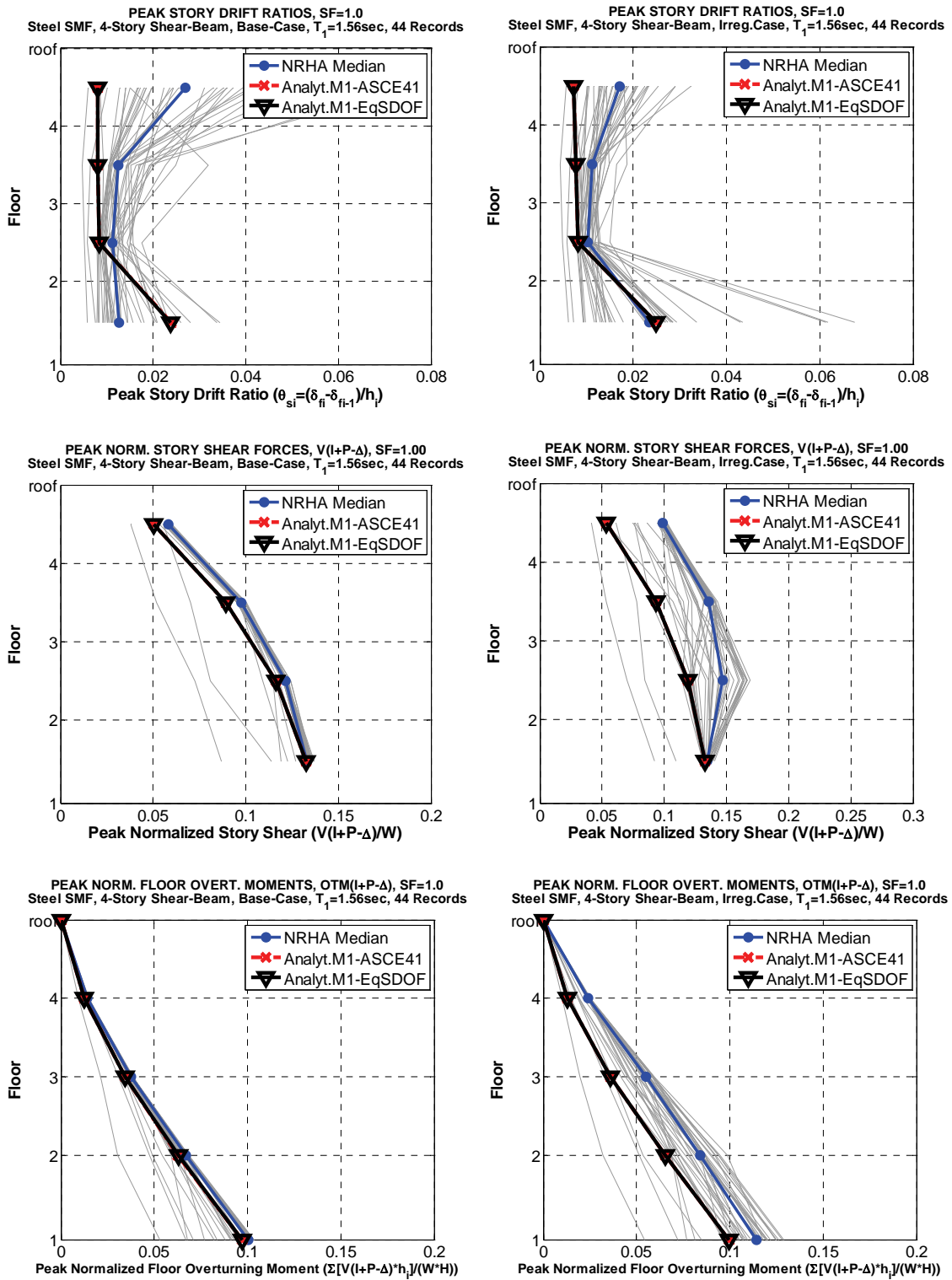


Figure 5-51 Comparison of nonlinear response history analysis results and single-mode nonlinear static analysis results for a 4-story steel shear building: base case (left) and irregular case (right).

5.5.3 Response Sensitivity to Changes in Yield Mode

Shear wall behavior depends strongly on the yield mode (i.e., flexure or shear). A simplified model of a 4-story reinforced concrete shear wall building, with walls designed to be flexure-controlled, was used to investigate response sensitivity to yield mode.

In design and in a nonlinear static analysis with an invariant load pattern, the relative magnitude of bending moment to shear force are locked to specific values in each story. Many nonlinear response history analysis studies have shown that the relative magnitude of bending moment to shear force (M/V ratio) can vary considerably during elastic and inelastic response, and the base shear force can increase significantly due to dynamic amplification. In taller buildings this increase can easily exceed a factor of two.

Figure 5-52 shows that nonlinear static analysis provides reasonably accurate predictions of story drift, but underestimates story shear demands because of dynamic amplification. The lack of dispersion in the nonlinear response history results for base shear clearly shows that almost every analysis attained maximum shear strength rather than bending strength in the first story. This result is the opposite of what would be concluded from a pushover analysis or expected to occur as a result of the design intent. Nonlinear response history analysis can “detect” a change in the yield mode from flexure to shear, which cannot be detected in a static analysis with an invariant load pattern.

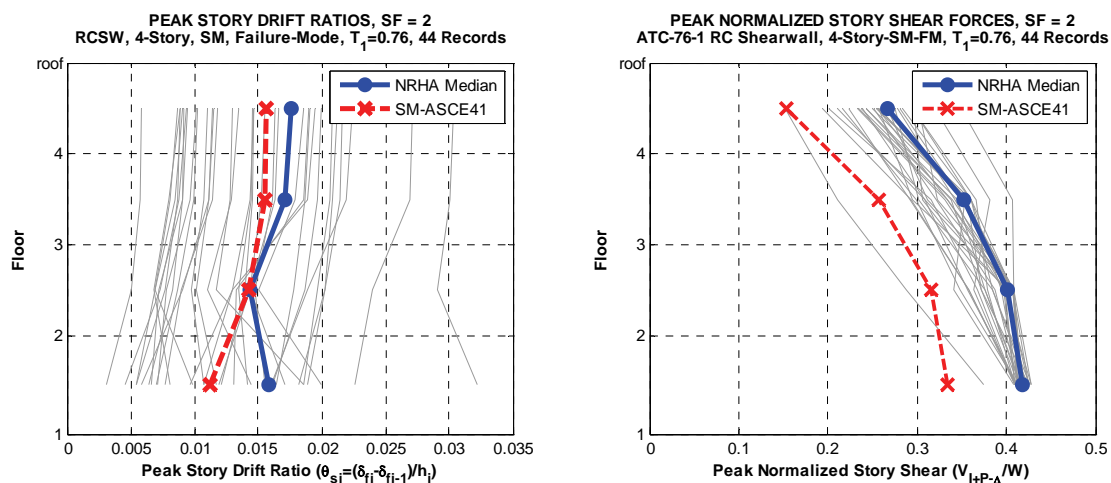


Figure 5-52 Comparison of nonlinear response history analysis results to nonlinear static analysis predictions of story drift ratio (left) and story shear force (right) for a 4-story reinforced concrete shear wall structure designed to be flexure-critical.

Results from Ancillary Studies

Ancillary studies were undertaken to supplement primary problem-focused studies, to expand the coverage provided by the basic systems that were selected, and to further investigate specific issues. This chapter summarizes the results of three ancillary studies, which are presented in detail in Appendices D, E, and F.

The first such ancillary study investigated the relationships among demand parameter dispersion, ground motion scaling method, and the size of ground motion data set, with the goal of finding more economical approaches for nonlinear response history analysis. The second ancillary study tested the practicality of directly determining target displacement for nonlinear static analysis. The third ancillary study, conducted by practicing structural engineers, applied the methods tested in primary studies (with some extensions) to models of special-case buildings encountered in practice using production software in common use among practitioners. The objective of this third study was to test the accuracy of the methods and to identify practical challenges to their implementation.

6.1 Ancillary Study on the Effects of Ground Motion Selection and Scaling on Response Quantity Dispersion

Codes, standards, and guidelines governing the application of nonlinear response history analysis (NRHA) prescribe, for example, the use of maximum of three or average of seven ground motions, as single components or in pairs. There is no statistical basis for this requirement, and the distinction between three (requiring the use of the maximum result) and seven (allowing the use of mean result) ground motions is simply an incentive to use more ground motion records in the analysis. Because this project aims to examine the relative accuracy of various analysis methods in the context of realistic variations in response, the blind application of such codified rules was considered inadequate, and alternatives were studied.

FEMA 440 (FEMA, 2005) established that results from a small number of nonlinear response history analyses, properly scaled using peak roof displacement as an intensity measure, provided more reliable predictions of demand than could be obtained using nonlinear static procedures. To further develop this potential, an ancillary study investigating the effects of ground motion scaling intensity measures and subset selection techniques on the central tendency and variability of response quantities was performed. The goal of this study was to identify methods that can be applied in practice to reliably characterize the central tendency and variability of

response parameters for use in current practice and explicit performance-based seismic design procedures. This study is presented in detail in Appendix D.

While ASCE/SEI 7-10 and ASCE/SEI 41-06 attempt to exercise some control over the characteristics of the suite of ground motion records, the basis of scaling (e.g., amplitude-only versus spectral matching), and the relative scaling of one ground motion with respect to another, is uncontrolled. This study uses the far-field ground motion record set introduced in FEMA P-695 (FEMA, 2009b). As developed, the set has been normalized by peak ground velocity to remove unwarranted variability between records due to differences in event magnitude, distance, source type, and site conditions, while still maintaining the inherent record-to-record variability present in the record set. Thus, this record suite provides a set of records for use in a consistent manner to evaluate collapse across all applicable Seismic Design Categories, located in any seismic region, and founded on any soil site classification (FEMA, 2009b).

6.1.1 Intensity Measure Selection

The choice of the intensity measure used as a basis for determining the strength of ground motions employed in nonlinear response history analysis influences the dispersion in the computed response quantities. Intensity measures that result in smaller dispersions are considered to be more efficient in that fewer nonlinear response history analyses are needed to characterize the distribution of the response quantity.

In this study, the following four scaling methods were used as a basis for establishing the intensity of ground motions used in nonlinear response history analyses:

- **FEMA P-695:** 22 pairs of orthogonal horizontal records were scaled to a common intensity that retains a degree of natural dispersion. This is functionally equivalent to using the geometric mean of the peak ground velocity of the two horizontal components of the ground motion, averaged over different orientations, as the intensity measure.
- $S_a(T_1)$: The records were scaled to the pseudo-spectral acceleration at the first mode period of the structure.
- $S_a(T_a, T_b)$: The records were scaled with a vector-valued approach where $S_a(T_a, T_b)$ is taken equal to the geometric mean of two elastic spectral values, $S_a(T_a)^{0.5} S_a(T_b)^{0.5}$ where T_a = the first mode period of the structure and T_b = the second mode period of the structure.
- $S_{di}(T_1)$: The records were scaled to the peak displacement of an equivalent single-degree-of-freedom oscillator.

Peak story drifts, floor accelerations, beam and column plastic hinge rotations, story shears, and overturning moments over the height of 2-, 4-, and 8-story reinforced

concrete moment-resisting frames were determined by nonlinear response history analysis using the FEMA P-695 ground motions scaled uniformly by scale factors of 0.5, 1.0, and 2.0. Maximum values of these quantities over the height of each frame were determined, and the dispersion and bias in computed response quantities were compared.

Observed Dispersion

No one intensity measure resulted in relatively small dispersions for all response quantities, frames, and scale factors considered. Use of $S_a(T_1)$ and $S_{di}(T_1)$ methods reduced dispersion for low intensity (elastic response). Use of FEMA P-695 and $S_a(T_a, T_b)$ methods reduced dispersion for floor accelerations. No one method was found to be clearly better than another for predicting story drifts at moderate and high intensities.

Observed Bias

The study was designed to establish nominally compatible intensities for the records used, with different scaling methods at a given scale factor. Bias resulting from differences in median values based on the choice of scaling method were caused by small differences in the hazard level represented by the scaling method. Each scaling method could bias some response quantities more than others, and greater dispersion was associated with greater apparent bias. It was observed that $S_a(T_1)$ and $S_{di}(T_1)$ methods are biased to produce slightly lower floor accelerations than those obtained from the FEMA P-695 method, and bias in story drift was inconsistent between the methods.

6.1.2 Record Subset Selection

Although the use of a large number of records (more than 30) is generally considered necessary to characterize the dispersion of a response quantity, there is some evidence (e.g., Azarbakht and Dolsek, 2007) that suggests relatively small subsets of ground motion records can be used to estimate median (50th percentile) and 84th percentile values of a response quantity. The effectiveness of three alternative approaches to record subset selection was investigated.

Subset Sizes and Selection Methods

Record subsets containing 1, 3, 5, 7, 9, 11, 13, 15, and 17 of the ground motion records comprising the 44-record set established in FEMA P-695 were determined using the following alternative approaches:

- Records were selected to match the median or 84th percentile spectra obtained for the full 44 record set, over specified period ranges.

- Records were first scaled to achieve a common $S_{di}(T_1)$ value, their elastic spectra plotted, and subsets selected to match the median or 84th percentile spectra of the full set.
- Records were scaled to achieve a common $S_a(T_1)$ value, their elastic spectra plotted, and subsets selected to match the median or 84th percentile spectra of the full set.

Each selection method was applied using scale factors of 0.5, 1.0, and 2.0. In each case, nonlinear response history analyses of the 2-, 4-, and 8-story reinforced concrete moment-resisting frames were conducted using the record subsets to determine estimates of median and 84th percentile values of individual and maximum story drifts, floor accelerations, story shears, floor overturning moments, beam plastic rotations, and column plastic rotations. These estimates were compared with the corresponding values obtained using the full set of 44 records. Errors in these estimates were determined as a function of subset size.

Estimation of Central Tendency

Each of the three approaches to record scaling and subset selection was successful at producing estimates of median and dispersion values with relatively small subset sizes. Error in the estimates generally decreased as the subset size increased, with relatively good estimates being obtained with five records. For estimates of the median using five records, mean absolute errors (for intermediate-level response quantities of story drift, floor acceleration, story shear, and overturning moment) ranged between approximately 3 and 10%, while maximum absolute errors ranged between approximately 20 and 30%. Mean and maximum errors for median component-level response quantities (e.g., beam and column plastic rotations) were much larger.

With respect to the relative accuracy of estimates of median floor accelerations and story drifts, the error resulting from the record subsets amplify the strengths and weakness observed for the scaling methods applied to the full set; that is, those scaling methods that resulted in greater dispersion in particular response quantities generally resulted in larger errors in estimates made using record subsets.

Subset selection was made over different period ranges. Records scaled in accordance with FEMA P-695 were selected within the period range $0.8T_2$ to $1.5T_1$. Those scaled to common values of S_{di} were selected within the period range $0.8T_2$ to $1.2T_2$ (since the initial scaling to S_{di} captures response at T_1). Records scaled to common values of $S_a(T_1)$ were selected within the range $0.8T_i$ to $1.5T_1$, where $i = \text{ceil}(\sqrt{N_{st}})$; *ceil* is the function that rounds up to the closest integer, and N_{st} is the number of stories. Thus, for the 8-story building, the range was $0.8T_3$ to $1.5T_1$.

Further study, with a broader set of buildings, would help identify the best period ranges for use in subset selection.

Even randomly selected record subsets would reduce error in median estimates with an increase in subset size. Random selections based on S_a and S_{di} scaling indicated that the subset selection techniques usually result in lower errors for a given subset size and a more monotonic reduction in error with increasing subset size.

Tests on the distribution of response quantities resulting from records scaled to common values of S_a and S_{di} indicate that lognormal distributions are better than normal distributions in characterizing the distributions of response quantities. Distributions on individual and maximum values of story drift ratio, floor acceleration, story shear, and overturning moment were considered. Of those, the distribution on floor acceleration exhibited the largest deviation from lognormal.

Estimation of Dispersion

An estimate of dispersion of a response quantity can be made by taking differences in the median response quantities obtained for the subsets matched to median and 84th percentile spectra of the full set. The dispersion estimate is given by $\ln(EDP_{84}) - \ln(EDP_{50})$, where EDP signifies the median result (engineering demand parameter) obtained for the particular record subset.

Use of a multipart estimation formula resulted in estimates of 84th percentile values that generally were about as accurate as the median estimates, when using S_{di} scaling. With this scaling method, the 84th percentile estimates were within 20% of actual values, with the exception of some plastic hinge rotations for the 8-story frame.

As an alternative, an attempt was made to estimate dispersion using the difference of median response quantities obtained for records representing different hazard levels. While there was some merit in such an approach, clear limitations were also identified.

6.2 Ancillary Study on Direct Determination of Target Displacement

In nonlinear static pushover analysis, target displacements have conventionally been determined by equivalent linearization or displacement coefficient approaches. These approaches make use of relationships between the peak displacement response of a yielding SDOF oscillator and that of an elastic counterpart, established on the basis of past computations. Appendix E describes an ancillary study to investigate the determination of target displacement by two alternative approaches.

In this study target displacements were estimated using the following:

- Direct computation

- Application of the open source software tool, Static Pushover 2 Incremental Dynamic Analysis, SPO2IDA (Vamvatsikos and Cornell, 2005b)

Direct computation of target displacement involves nonlinear response history analysis to determine peak inelastic displacement of an equivalent SDOF system. It allows (but also necessitates) the use of ground motion records suitable for the location and site characteristics of interest. The main reasons for computing the target displacement directly from an equivalent SDOF oscillator are that: (1) it permits use of hysteretic models appropriate for the structure under consideration (if sufficient justification exists to use a more complex model than a simple bilinear one); (2) it allows use of a multi-linear load-displacement backbone curve, which might have a significant effect on the target displacement for structures whose pushover curve exhibits clear multi-linear shear force versus roof displacement characteristics; (3) it may be used for different damping levels and damping models; and (4) it provides much more realistic target displacement predictions than the coefficient method outlined in ASCE/SEI 41-06 in cases where the target displacement is in the negative tangent stiffness region of the pushover curve.

The SPO2IDA tool makes use of previously computed results for a large variety of capacity boundaries to allow estimates of incremental dynamic analysis curves to be determined. Such curves describe peak displacement response at 16th, 50th, and 84th percentiles as a function of pseudo-spectral acceleration.

Both approaches were found to produce useful estimates of peak displacement and are recommended as replacements for the displacement coefficient and equivalent linearization approaches. In particular, they are highly recommended in cases where: (1) the target displacement is in the negative tangent stiffness region of the global pushover curve and (2) the pushover curve is not close to linear before global yielding occurs (e.g., stiffness changes that occur between the pre- and post-cracking regimes of reinforced concrete or masonry structures).

6.3 Ancillary Study on Practical Implementation of Analysis Methods

An ancillary study was conducted by practicing structural engineers on the practical implementation of analytical methods investigated in the focused analytical studies. The study consisted of applying the various methods (with some extensions) to models of special-case buildings encountered in practice using production software. The objectives of this study were to: (1) identify challenges and issues related to the practical implementation of nonlinear response history analysis; and (2) evaluate various existing and modified analysis techniques for their ability to capture higher mode response in detailed models of real structural systems. A detailed description of the study, as well as its findings, is provided in Appendix F.

6.3.1 Approach

Two special-case buildings, Building A and Building B, were studied with production software typically available to, and used by, practicing design professionals. The nonlinear models reflected assumptions and procedures typically followed by practitioners in performing nonlinear analysis.

The following analysis methods were used to analyze each structure:

- Nonlinear static pushover analysis
- Single and multimode response spectrum analysis
- Nonlinear pushover analysis plus elastic higher modes
- Modal Pushover Analysis (MPA)
- Consecutive Modal Pushover analysis (CMP)
- Extended Consecutive Modal Pushover analysis (Extended CMP)

Results from each of these analyses were compared to results taken from a suite of eight nonlinear response history analyses (NRHA). Ground motion scale factors of 0.5, 1.0, and 2.0 were considered, but only results for a scale factor of 2.0 are summarized here.

Damping was assumed to be 5% in all analyses. Only Rayleigh damping was used in the Building A model, and a mixture of Rayleigh and modal damping was used in the Building B model. An inherent damping value of 5% is higher than would typically be used for these building types; a value of 2% would be more common. As a result, predicted deformation demands could be 20% to 25% lower than would otherwise be expected. Force demands could also be higher, depending on the degree of nonlinearity in the analysis.

Ground Motions

Since this study was intended to capture the bi-directional and torsional effects, the ground motions were used as orthogonal component pairs instead of individual records. In order to accurately represent this application in practice, 8 ground motions pairs from the 22 pairs introduced in FEMA P-695 were chosen such that the average spectrum of their spectra combined with the square-root-sum-of-the-squares (SRSS) method was roughly equivalent to the average spectrum of the 22 pairs combined with the same method (see Figure 6-1). Depending on the target spectrum, this approach varies from the ASCE/SEI 41-06 provisions for ground motion scaling but is consistent with the more recent provisions in ASCE/SEI 7-10 for sites more than 5 km from a fault.

For these eight pairs, ground motion components from each pair were oriented along the principal axes as shown in the figure, such that the average spectrum of the ground motions in the X-direction was roughly equivalent to the average spectrum of the ground motions in the Y-direction.

The spectrum determined from the average of the eight ground motion components in each primary direction was then used as the input spectrum for all other analysis procedures.

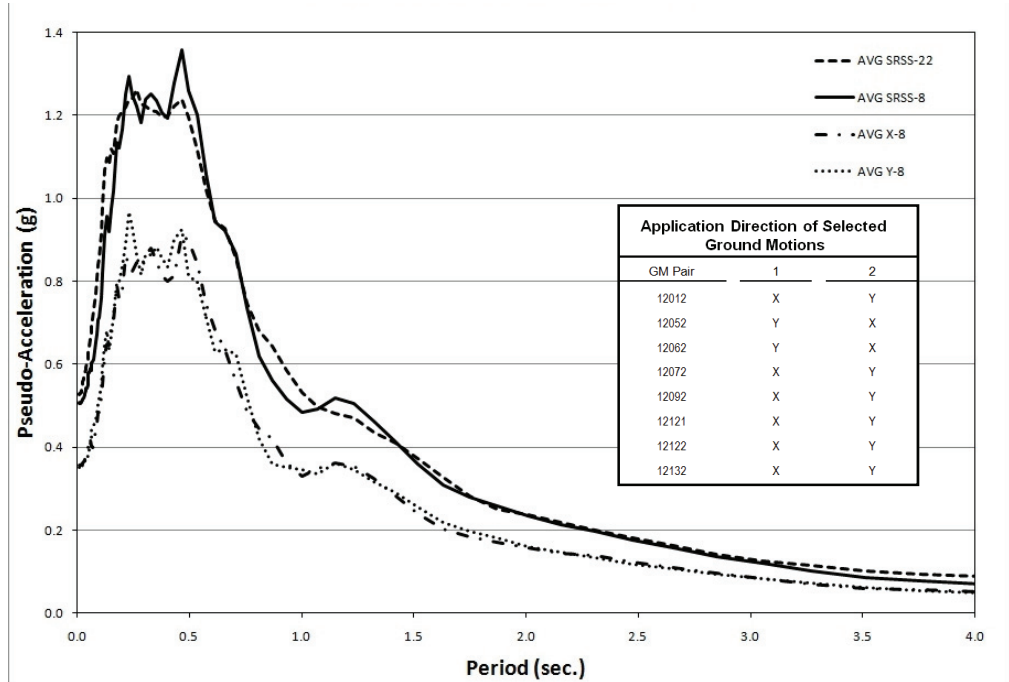


Figure 6-1 Average spectra for ground motions used in this study.

Study Building A

Building A is a 6-story steel frame structure with a moment-resisting frame system. The building is 111 feet by 260 feet in plan, with an overall building height of 82 feet. The moment frames are located along each transverse frame line and exterior longitudinal frame line. The system also includes two buckling-restrained braced frames. The diaphragms are concrete fill on metal deck. The typical floor plan with moment frame layout is shown in Figure 6-2.

The original three-dimensional analysis model for Building A is shown in Figure 6-3. It was developed using SAP 2000 to reflect the dimensions, members, and materials of the building. When nonlinear responses history analyses of the original three-dimensional model failed, modifications were made in an attempt to develop a reasonably stable analysis. Changes (and simplifications) included eliminating foundation springs and uplift elements, fixing column bases, replacing axial load-moment interaction hinges with flexural hinges, changing moment hinges to

eliminate fracture behavior, and changing all hinges to be bi-linear without degradation. The resulting final model was a two-dimensional frame model, also shown in Figure 6-3. The resulting modal properties of the Building A model are listed in Table 6-1. The mass participation in the first mode is 85%, indicating that the effects of higher modes should be modest.

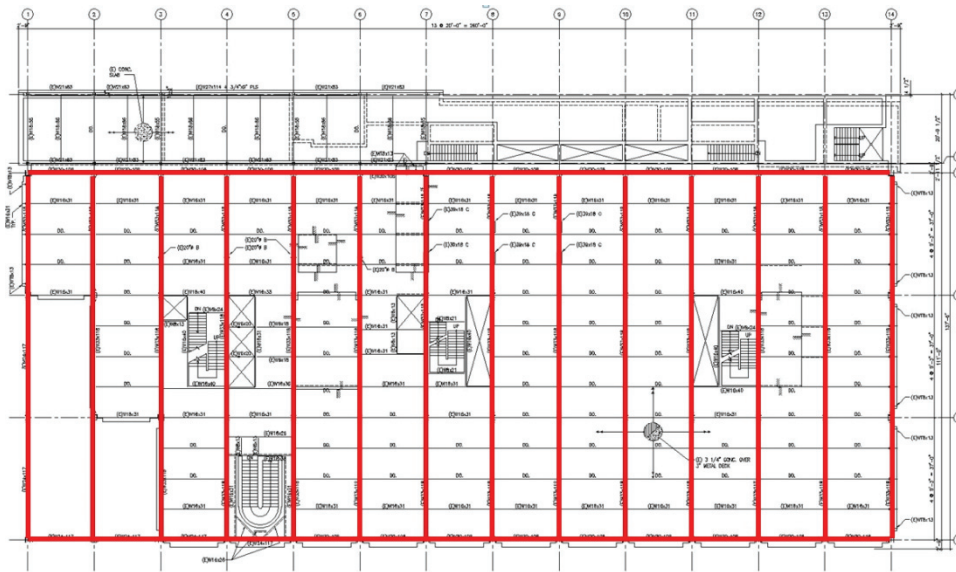


Figure 6-2 Typical floor plan of Building A.

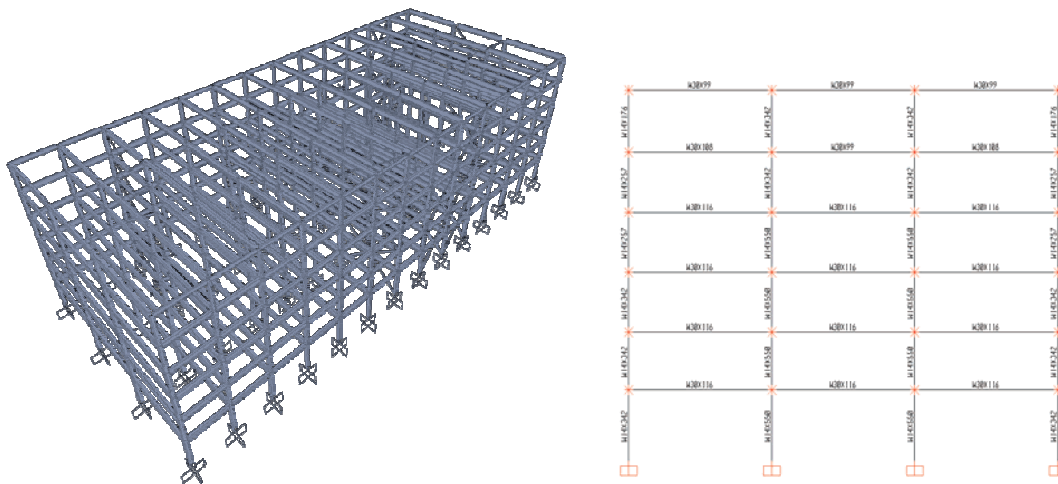


Figure 6-3 Three-dimensional SAP2000 model (left) and two-dimensional transverse frame model (right) for Building A.

Table 6-1 Modal Properties for Building A

	Transverse		Longitudinal	
	Mode 2	Mode 5	Mode 1	Mode 4
T_1 (s)	1.33	0.47	1.55	1.79
M_n^*/Weight	84.1%	11.1%	82.0%	12.1%

Study Building B

Building B is a 3-story steel braced frame structure. In the longitudinal direction, there are two chevron braced frames along each exterior line. In the transverse direction there are three braced frames: (i) two exterior, 3-story, 5-bay braced frames; and (ii) one interior, 2-story, 2-bay braced frame. The building is 168 feet by 210 feet in plan, with an overall building height of 46 feet. The two floor diaphragms are concrete fill on metal deck, and the roof diaphragm is bare metal deck. The final model, developed using PERFORM-3D, is shown in Figure 6-4.

The modal properties of Building B are summarized in Table 6-2. The transverse (H2) response of Building B includes a flexible roof diaphragm and a soft first story. This presents a challenging higher-mode problem that is quite different from the response of Building A. The flexible roof diaphragm mechanism is entirely different from the soft first story mechanism, yet both occur in the building as a result of different earthquake records, as can be seen in Figure 6-7a.

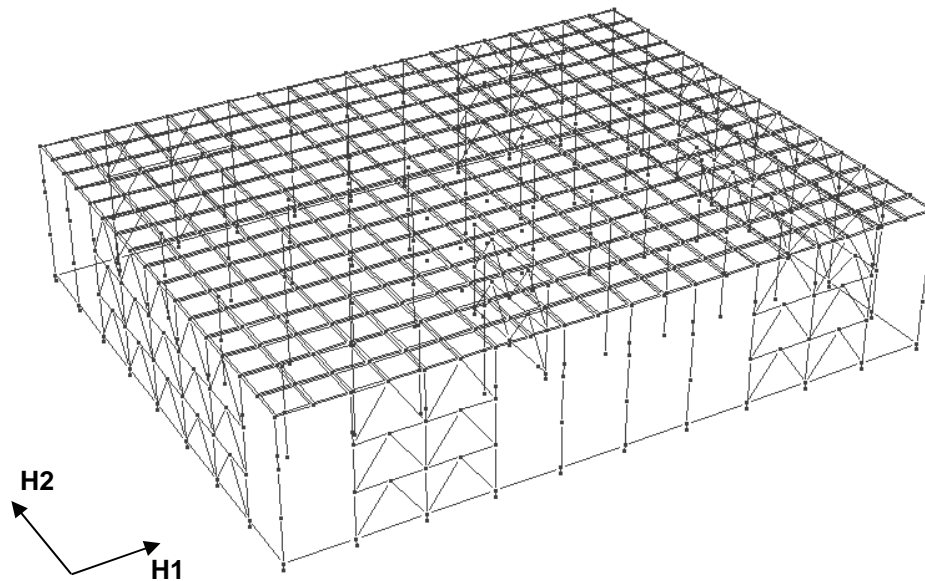
Table 6-2 Modal Properties for Building B

	Transverse (H2)		Longitudinal (H1)	
	Mode 2	Mode 5	Mode 1	Mode 3
T_1 (s)	0.539	0.295	0.549	0.318
M_n^*/Weight	31.6%	58.9%	62.3%	28.8%

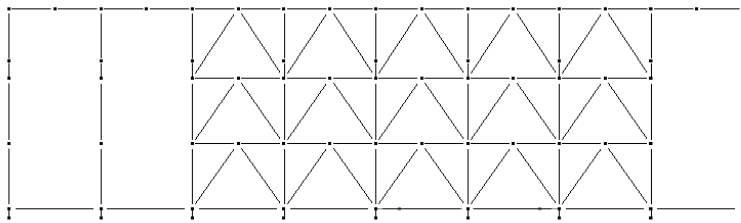
6.3.2 Results from Nonlinear Response History Analysis

The results from nonlinear response history analyses are taken as the baseline for comparison with other analysis methods. The drift profiles for Building A exhibit a variety of deformation patterns, with peak story drifts occurring in upper levels in some records and in lower levels for other records (Figure 6-5). This variability illustrates the challenge of developing a single pushover analysis protocol that captures both types of behavior.

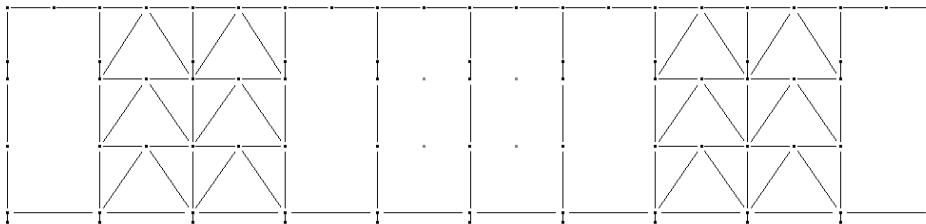
Results for Building B are shown in Figure 6-6 and Figure 6-7. Nonlinear response occurs in the braced frames in both directions, indicating the formation of a soft story mechanism in the first story. The effect of rocking in the longitudinal (H1) direction is visible in both the drift and displacement plots, indicated by lower demands below the roof level as compared to the transverse (H2) direction.



(a) Three-Dimensional View



(b) Transverse Frame Elevation (H2)



(c) Longitudinal Frame Elevation (H1)

Figure 6-4 PERFORM-3D model for Building B.

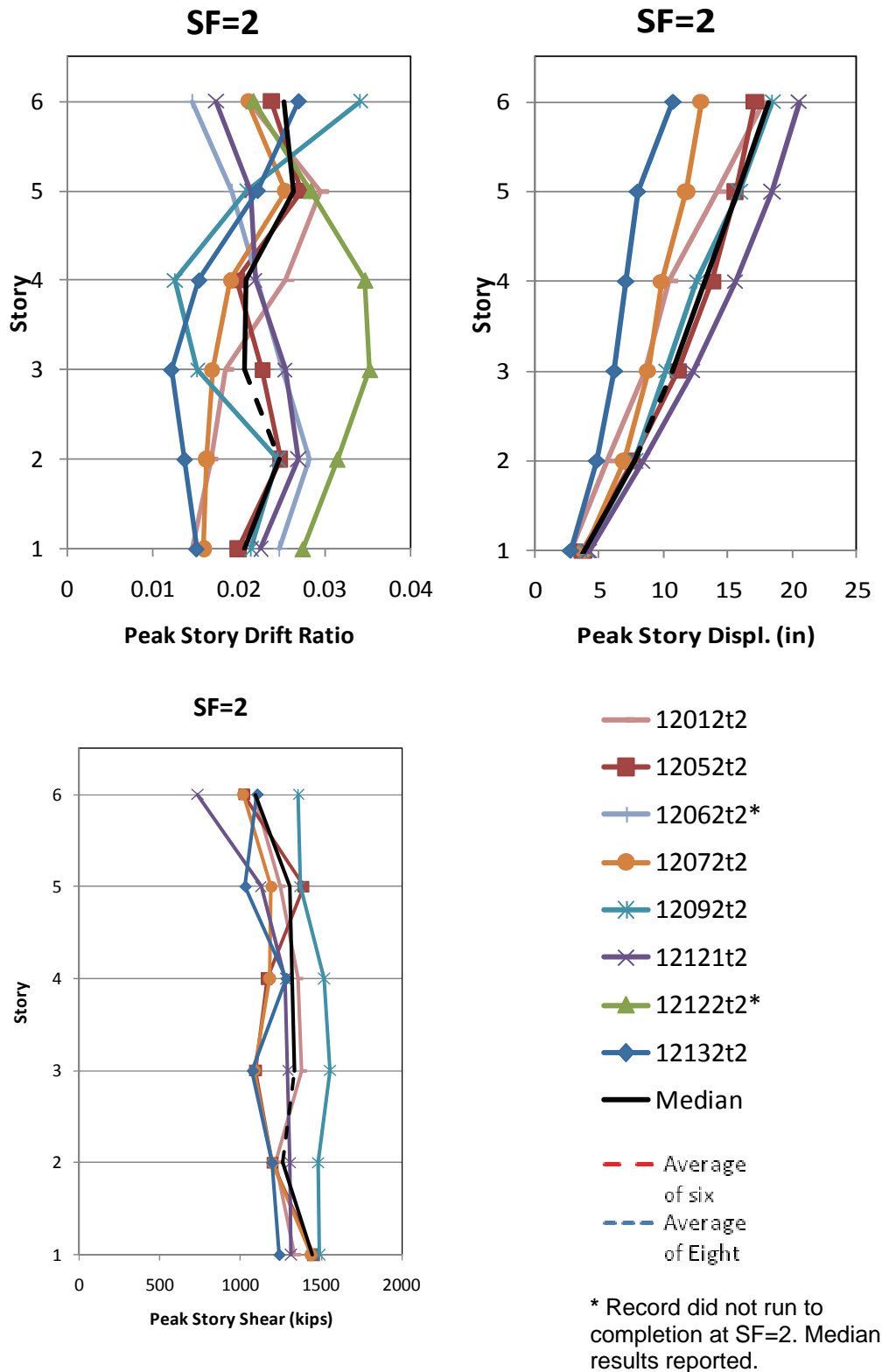


Figure 6-5 Dispersion in story-level demand parameters over height in Building A subjected to different ground motions.

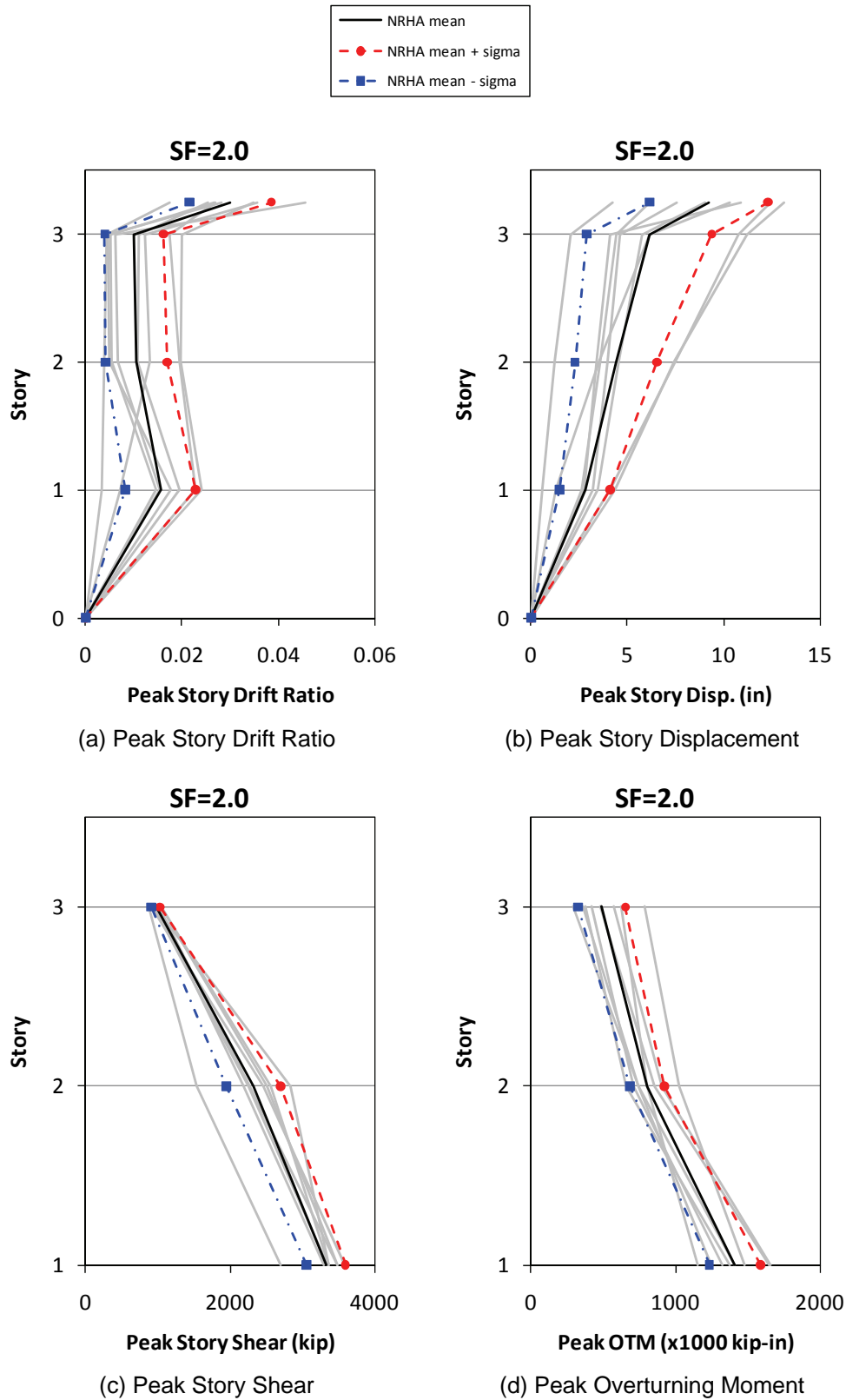


Figure 6-6 Dispersion in story-level demand parameters over height in Building B for longitudinal direction (H1).

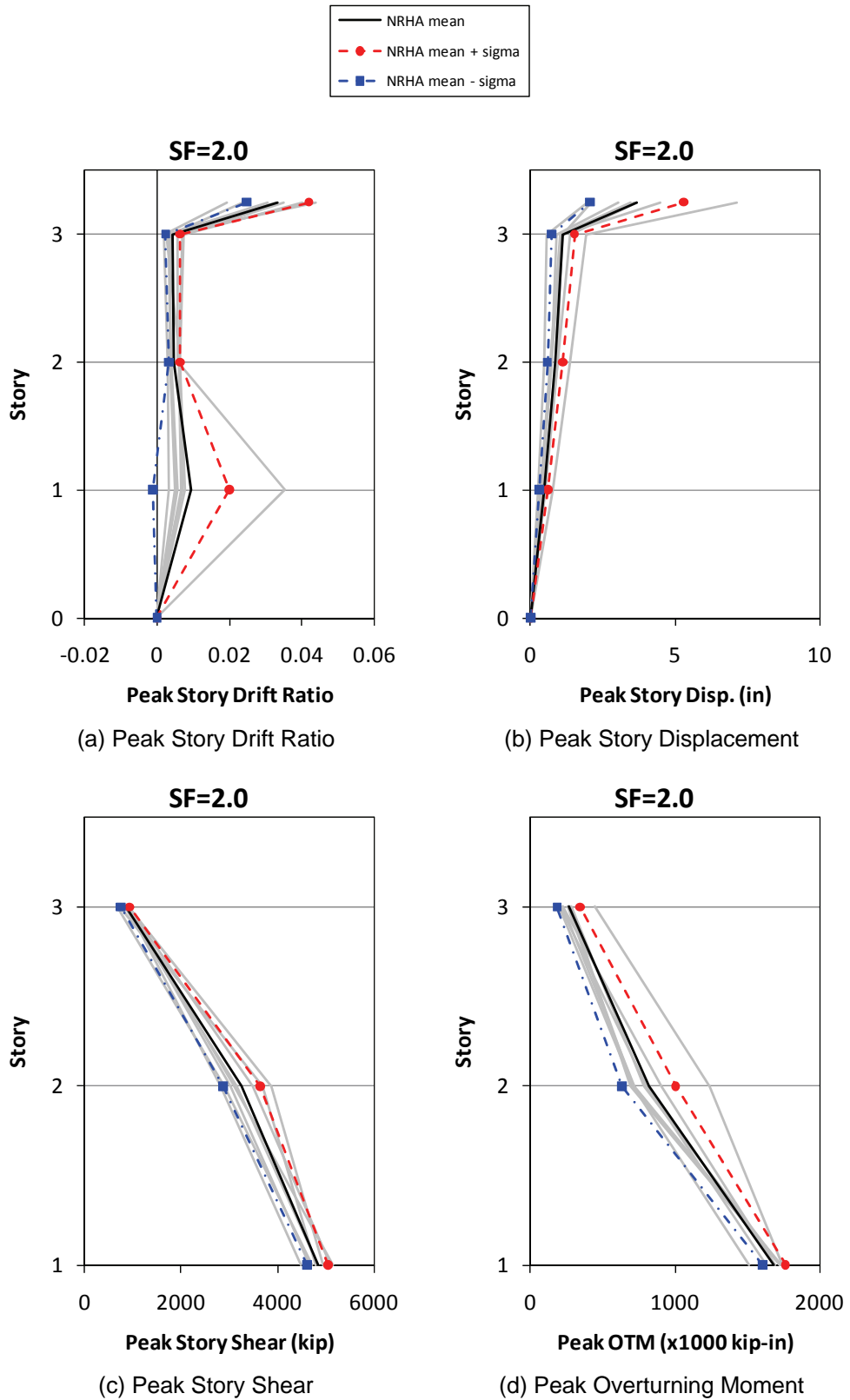


Figure 6-7 Dispersion in story-level demand parameters over height in Building B for transverse direction (H2).

6.3.3 Results from Nonlinear Static Pushover and Response Spectrum Analysis

Nonlinear static analysis and elastic response spectrum analysis procedures were performed on both buildings. Results for a single-mode response spectrum analysis are presented to illustrate the effects of including the higher modes in the analysis. Modal pushover curves for Building A and Building B are presented in Figure 6-8 and Figure 6-9, respectively.

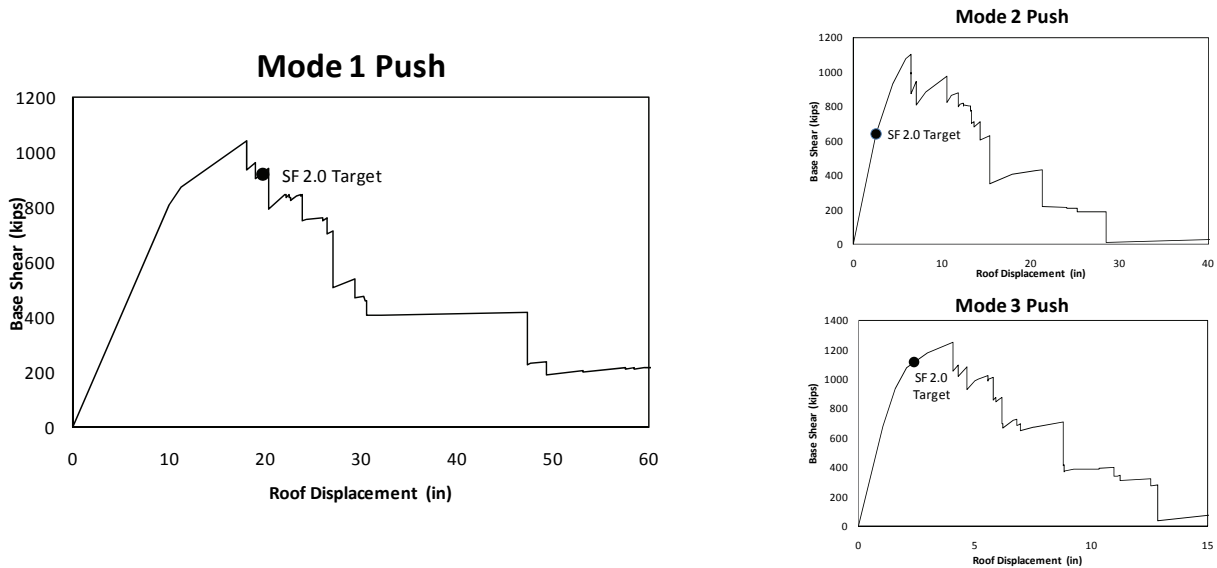


Figure 6-8 Modal pushover curves for Building A.

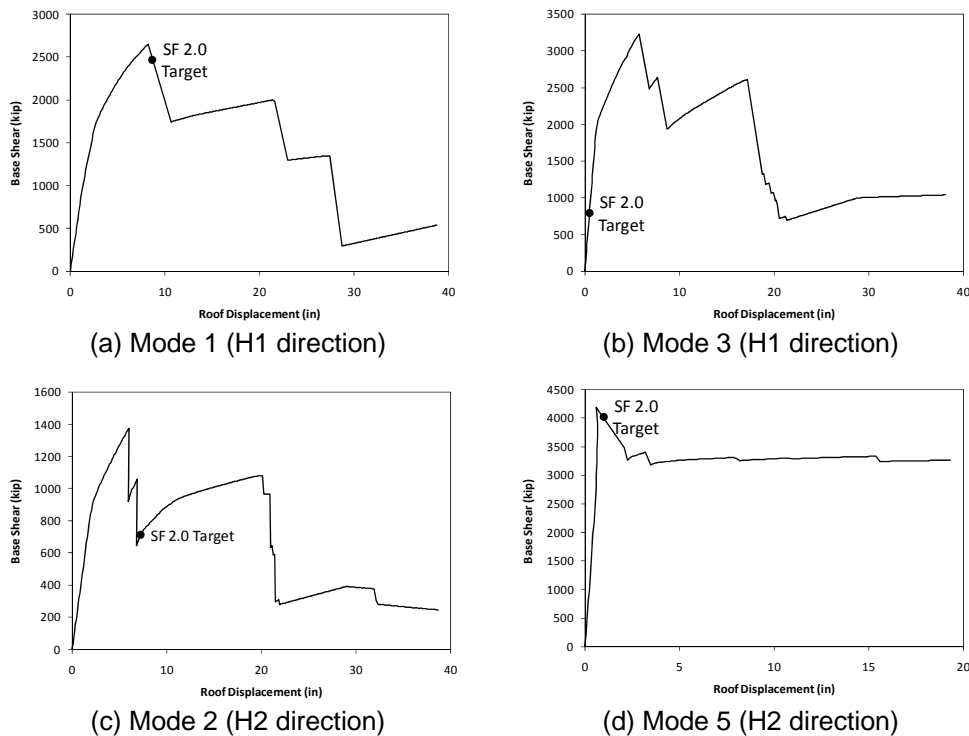


Figure 6-9 Modal pushover curves for Building B.

Results from nonlinear static and elastic response spectrum analysis are compared with the nonlinear response history analysis results in Figure 6-10 and Figure 6-11. Traces labeled ASCE 41 show nonlinear static analysis results and the traces labeled RSA-all modes and RSA-1 mode show the elastic modal response spectrum analysis results. The figures also show results for the SRSS of pushover analysis results combined with higher mode response from the elastic response spectrum analysis (labeled ASCE 41 plus RSA). This is technically a special case of the Modal Pushover Analysis, which is described in Section 6.3.4.

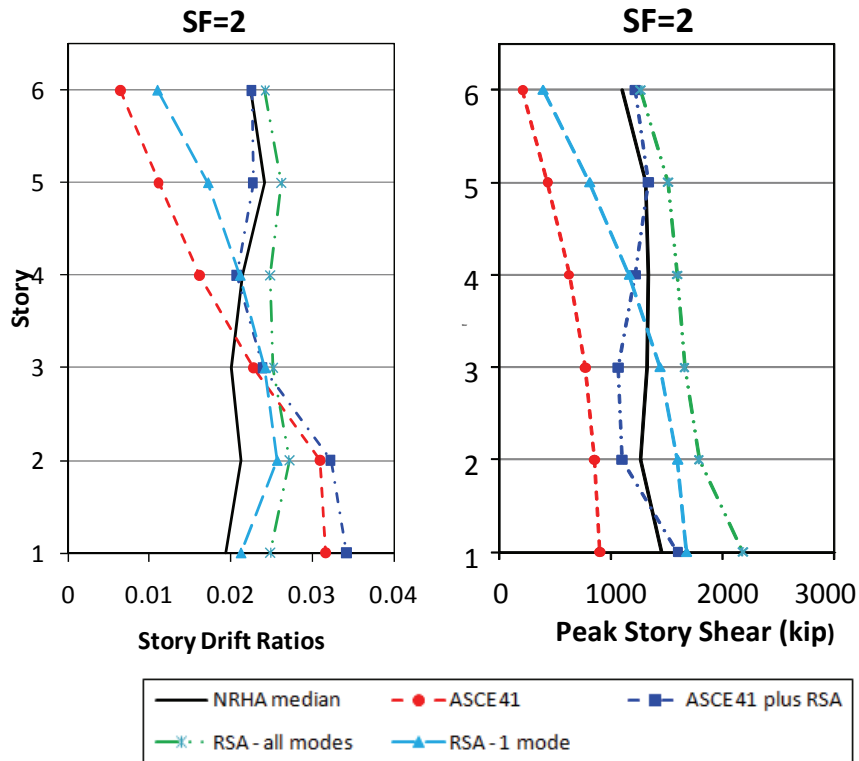


Figure 6-10 Comparison of nonlinear response history analysis median results to nonlinear static and elastic response spectrum analysis results for Building A.

Observations for Building A

The threefold increase observed in the sixth story shear between the pushover and the response spectrum analysis results is well in excess of the 1.3 factor that would trigger consideration of higher mode response. The computed value of R for this assessment is below the ASCE/SEI 41-06 limit of R_{max} , indicating that nonlinear response history analysis would not be required, even at a scale factor of 2.0.

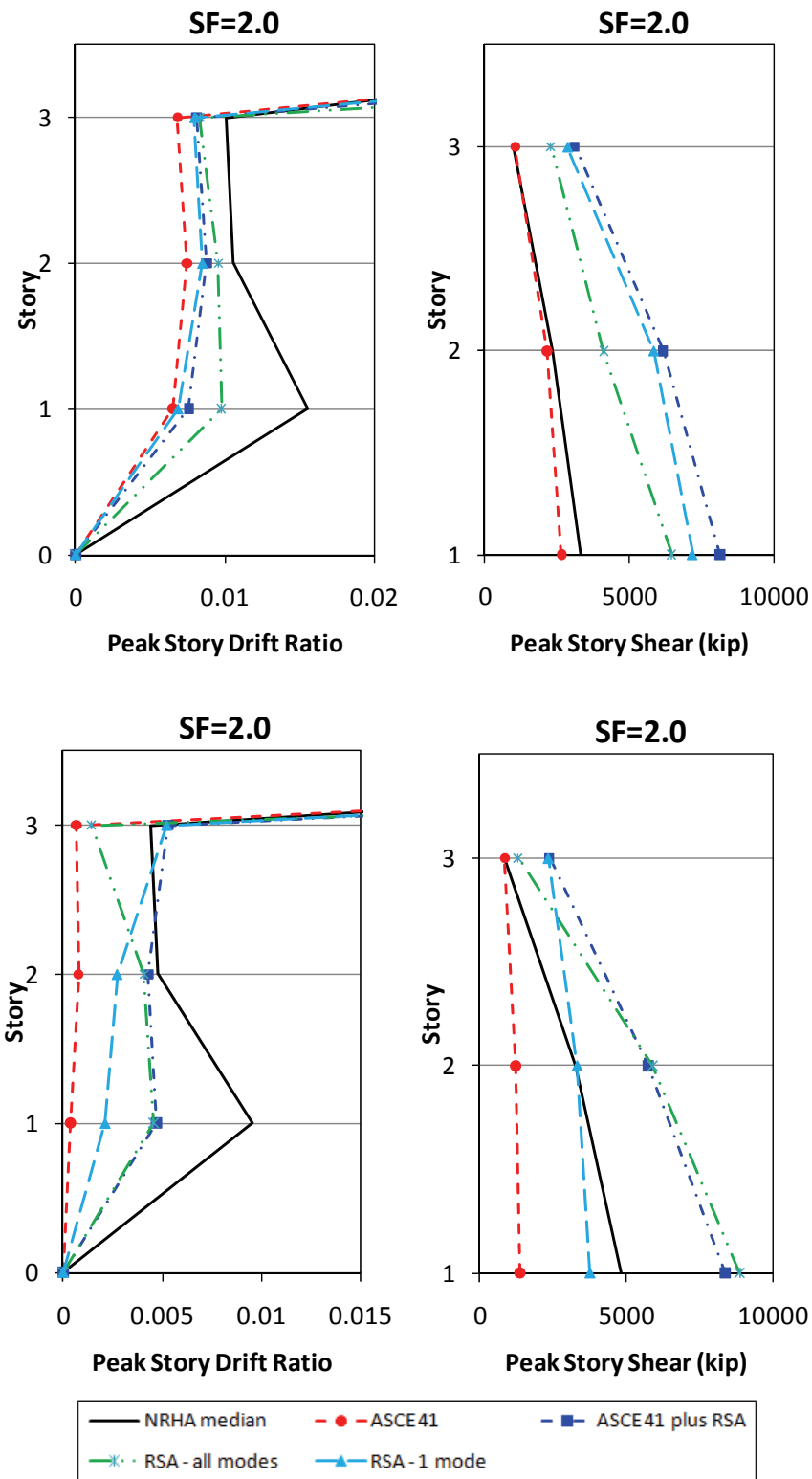


Figure 6-11 Comparison of nonlinear response history analysis median results to nonlinear static and elastic response spectrum analysis results for Building B in the longitudinal direction (top) and transverse direction (bottom).

At a scale factor of 2.0, the linear and nonlinear methods started to diverge for reported story shears and overturning moments. Even though the basic response spectrum analysis does a relatively good job of matching drifts, it overestimated shears. In ASCE/SEI 41-06, response spectrum analysis is currently used as a supplement to pushover analysis to account for higher mode response. This process is feasible because results from the two analysis techniques are never combined.

If the results are combined across linear and nonlinear techniques, as displayed in the ASCE 41+RSA trace, then there is an inherent conflict because forces in deformation-controlled components can exceed member capacities. This implies that yielding should have occurred earlier in the analysis than was indicated in the pushover analysis alone. The presence of the higher mode forces can also change the mechanism that develops in the frame.

Observations for Building B

In the longitudinal (H1) direction, the drift and displacement results for Building B are a relatively good match to the nonlinear response history analysis results for scale factors of 0.5 and 1.0. At a scale factor of 2.0, all simplified methods fail to pick up the soft first story and underestimate the response. In the transverse (H2) direction, the single-mode methods (ASCE 41 and RSA-1 mode) markedly underestimate the nonlinear response history analysis results, even at low scale factors, due to lower mass participation in the fundamental mode.

The story shear results in the longitudinal (H1) direction show the effect of rocking on response (Figure 6-11). Nonlinear static analysis results are a relatively close match to nonlinear response history analysis results, and elastic response spectrum analysis results overestimate forces, as expected.

The multimode methods (RSA and ASCE 41+RSA Higher Modes) show good agreement in the elastic range at low scale factors, but again overestimate the results at a scale factor of 2.0. Nonlinearity initially occurs in the roof diaphragm, and then occurs in the first-story braced frames. There is no rocking mode to control the response, which becomes increasingly more multi-mode at larger scale factors. The story shears from pushover analysis consequently underestimate the nonlinear response history analysis results at all scale factors.

6.3.4 Results from Modal Pushover Analysis

Modal Pushover Analysis (MPA) results are compared with the nonlinear response history analysis results in Figure 6-12 and Figure 6-13. These figures also show results other multi-mode techniques, including Consecutive Modal Pushover and Extended Consecutive Modal Pushover.

Observations – Modal Pushover Analysis

Modal Pushover Analysis improved the match to the nonlinear response history analysis results for all demand parameters at all scale factors. In Building B the improvement is most noticeable in the transverse (H2) direction (Figure 6-13).

Modal Pushover Analysis, however, failed to pick up the story mechanism that developed in the first story, as the target displacement for the second mode in each direction was lower than that required to cause buckling of the braces. This highlights the primary observed limitation of the Modal Pushover Analysis technique, which is that forces and deformations combined at the end of the analysis may subsequently imply that yield or failure should have occurred earlier in the analysis than was actually the case.

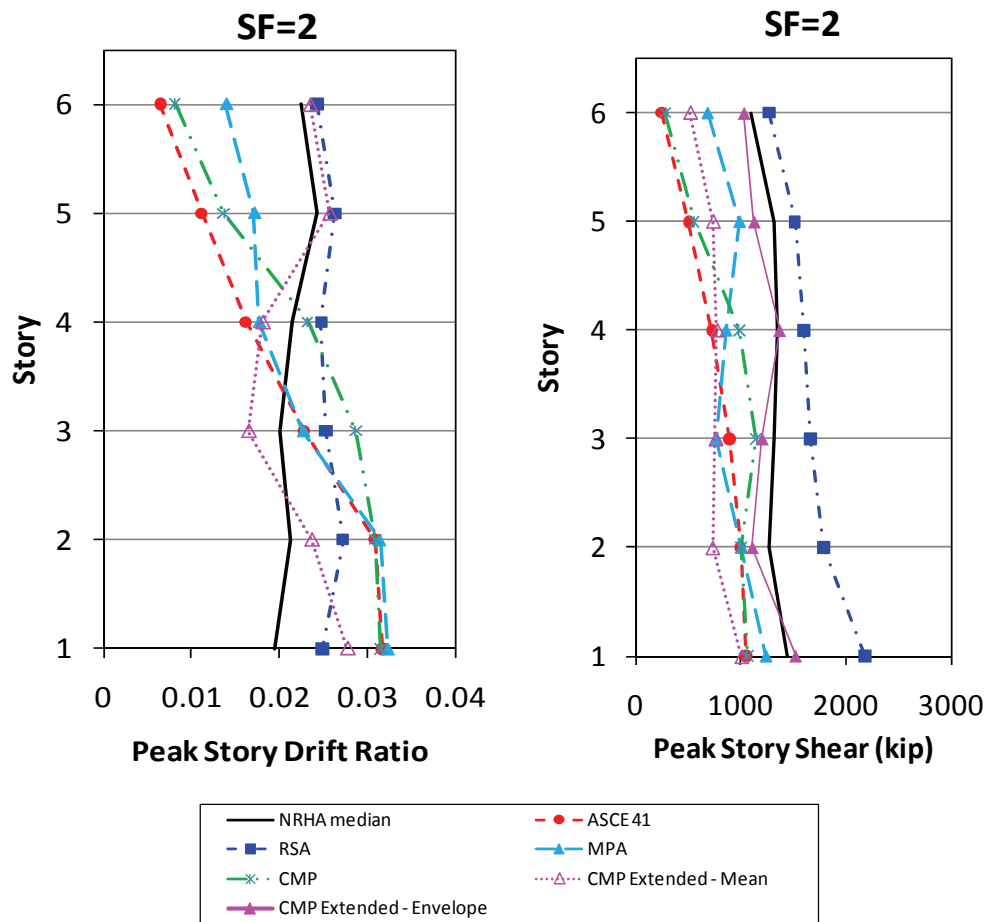


Figure 6-12 Comparison of nonlinear response history analysis results to various multi-mode pushover analysis results for Building A.

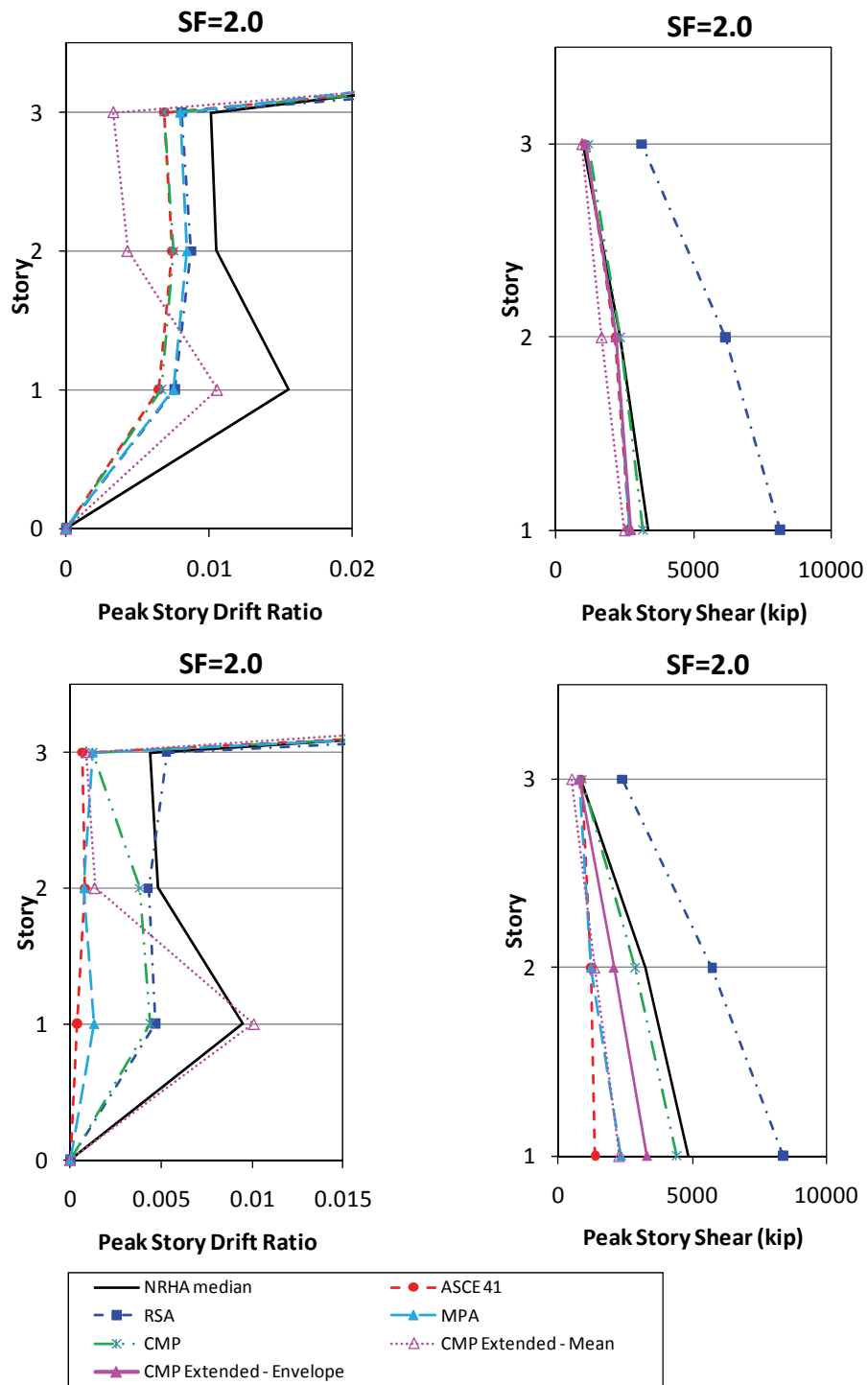


Figure 6-13 Comparison of nonlinear response history analysis results to various multi-mode pushover analysis results for Building B in the longitudinal direction (top) and transverse direction (bottom).

While MPA does introduce alternate deformation patterns into the analysis, it does not consider interaction between the modal demands. For example, performing the second-mode pushover before or after the first-mode pushover has no effect on the results of a Modal Pushover Analysis.

6.3.5 Results from Consecutive Modal Pushover Analysis

Consecutive Modal Pushover (CMP) results are compared with the nonlinear response history analysis results in Figure 6-12 and Figure 6-13. Results from other multi-mode techniques are also shown in the figures.

Observations – Consecutive Modal Pushover

In the Consecutive Modal Pushover analysis of Building A, only a positive second mode push after an initial fundamental mode push was considered; therefore, the observed improvement was relatively modest. In the longitudinal (H1) direction, results from Consecutive Modal Pushover analysis of Building B are nearly identical to the pushover and MPA results. In the transverse (H2) direction, the Consecutive Modal Pushover analysis results in inconsistent changes relative to the nonlinear response history analysis results, depending on scale factor.

The difference was attributed to considering only a positive direction second mode pushover, which is applied after the first mode pushover. Since it was scaled to produce an increase in roof displacement, the second mode in each direction tended to reduce the story shears in the lower levels. This accounted for the comparatively low response predictions at the first and second floor levels at a scale factor of 2.0, and illustrated the importance of considering both positive and negative signs for modal responses when using this procedure.

6.3.6 Results from Extended Consecutive Modal Pushover Analysis

In this study, the Consecutive Modal Pushover procedure was modified (extended) based on observed limitations in the previous analyses. The effect of sign (direction) of the modal responses was considered by altering the sign of the applied higher modes. This has the effect of generating different deformation patterns in the structure, similar to what occurs in nonlinear response history analysis for different earthquake records.

Additionally, it was observed that the deformation pattern in the building was more significantly altered if higher modes were applied before the fundamental mode pushover. This suggested that a series of consecutive pushover analyses, with modes of different signs and different sequencing, could be used to develop the range of deformation patterns that might occur in the nonlinear dynamic response of a building.

For Building A, implementation of the Consecutive Modal Pushover required three modes, creating a large number of possible modal pushover combinations and permutations. To limit the number of analyses, this study focused on consecutive combinations in which the higher mode pushovers were performed first, since these tended to produce more variation in the observed mechanism that formed in the frame.

In the Extended CMP, seven model pushover sequences were examined, as follows:

- +M1
- +M2+M1 and +M2-M1 (Figure 6-14)
- +M3+M2+M1, +M3+M2-M1, -M3+M2+M1, and -M3+M2-M1 (Figure 6-15)

where M1 represents the first-mode response, M2 represents the second-mode response, and M3 represents the third-mode response.

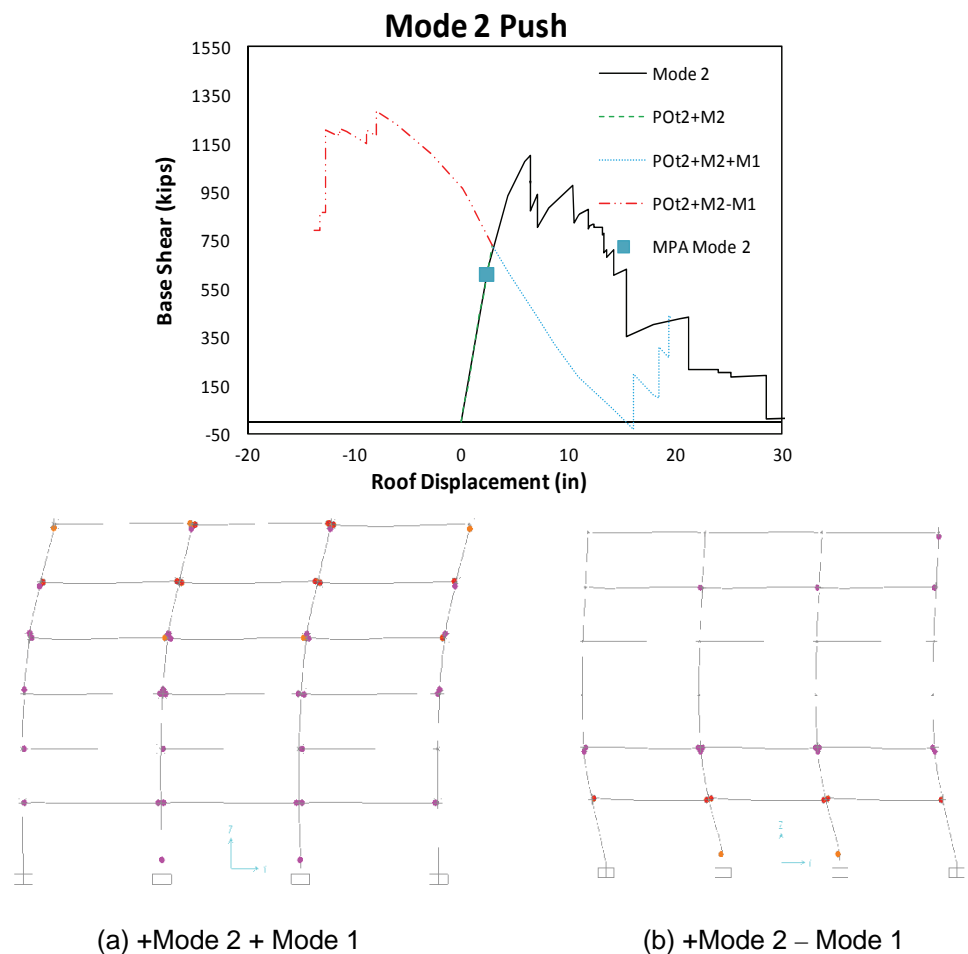
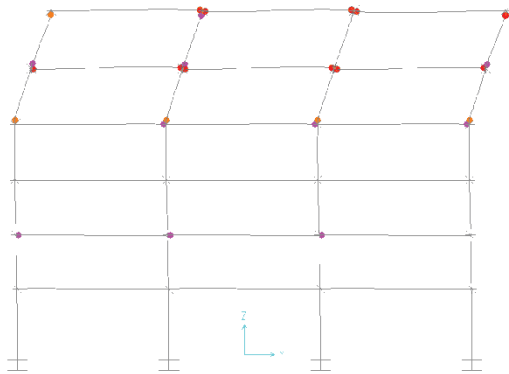
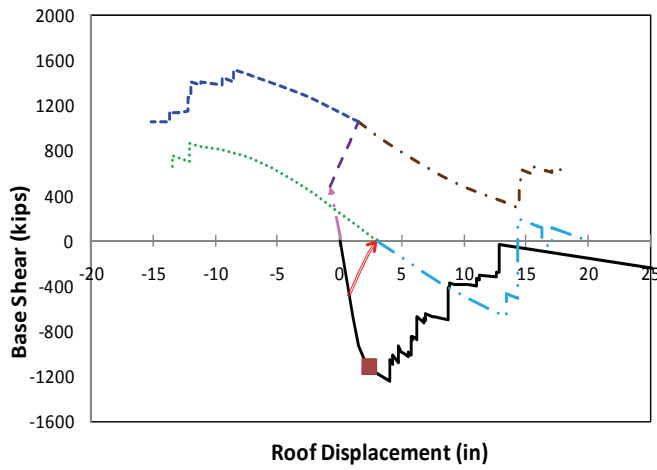
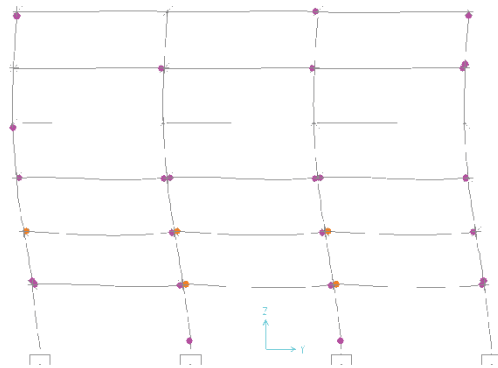


Figure 6-14 Two-stage Extended Consecutive Modal Pushover and deflected shapes for Building A.

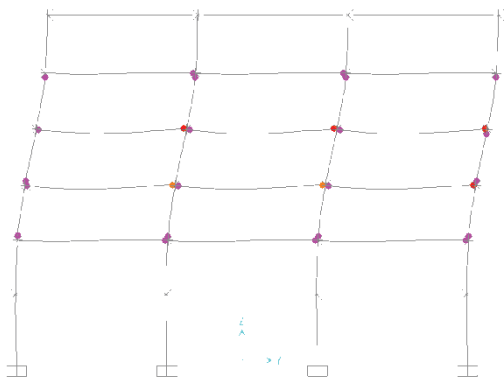
Mode 3 Push



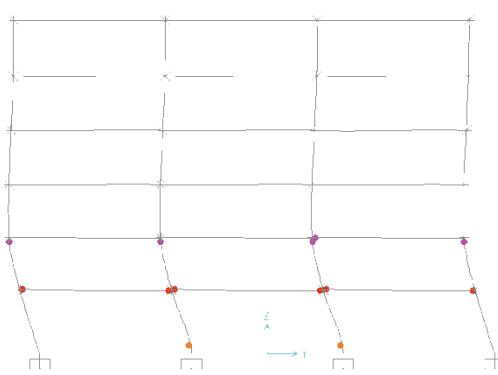
(a) +Mode 3 + Mode 2 + Mode 1



(b) +Mode 3 + Mode 2 - Mode 1



(c) - Mode 3 + Mode 2 + Mode 1



(d) - Mode 3 + Mode 2 - Mode 1

Figure 6-15 Three-stage Extended Consecutive Modal Pushover and deflected shapes for Building A.

The Consecutive Modal Pushover procedure sets the target displacement of the second mode in a two-mode pushover based on the first mode target displacement multiplied by one minus the first mode participating mass ratio. The result is that the first mode is pushed only to the target times the mass participation factor. When the second mode was applied first, it became apparent that the higher mode response was being overestimated, as the computed target displacements were substantially exceeding the target displacements calculated by Modal Pushover Analysis for the higher modes alone.

For the three-stage analyses, a slight variation was adopted to better balance the contribution of the higher modes. The total higher mode contribution for M2 and M3 was based on one minus the first mode mass participation factor. This was converted to a spectral displacement, which was then apportioned to each mode in proportion to its own computed spectral displacement and then converted back to roof displacement. In Figure 6-14 and Figure 6-15, which illustrate the two- and three-stage Extended Consecutive Modal Pushover procedure, the base shear versus roof displacement trace can be followed for the various modal combinations considered. The pushover curve intersections indicate the transition points from one mode to another.

Observations – Extended Consecutive Modal Pushover

Figure 6-16 shows a comparison of Extended CMP analysis results for Building A based on different modal pushover sequences. Figure 6-17 shows a comparison between Extended CMP results and all other analysis results for peak story drift ratio and peak story shear in Building A. Based on these results, the following observations on the Extended CMP procedure were made:

- Very different deformation patterns were developed as a result of the different modal pushover sequences, and each analysis produced peak response in a different part of the structure.
- Results from the Extended CMP were generally more variable than nonlinear response history analysis results.
- The mean of the seven Extended CMP analyses was a good match to the median nonlinear response history analysis result. If each Extended CMP analysis considered as a separate earthquake event, this is consistent with the concept of taking the average response of a suite of records.
- As an alternative, an envelope of the Extended CMP analyses was considered. This is consistent with the concept of taking the maximum response from a suite of records.

- Relative to other methods, the Extended CMP appeared to provide an improved match to maximum demands for Building A.

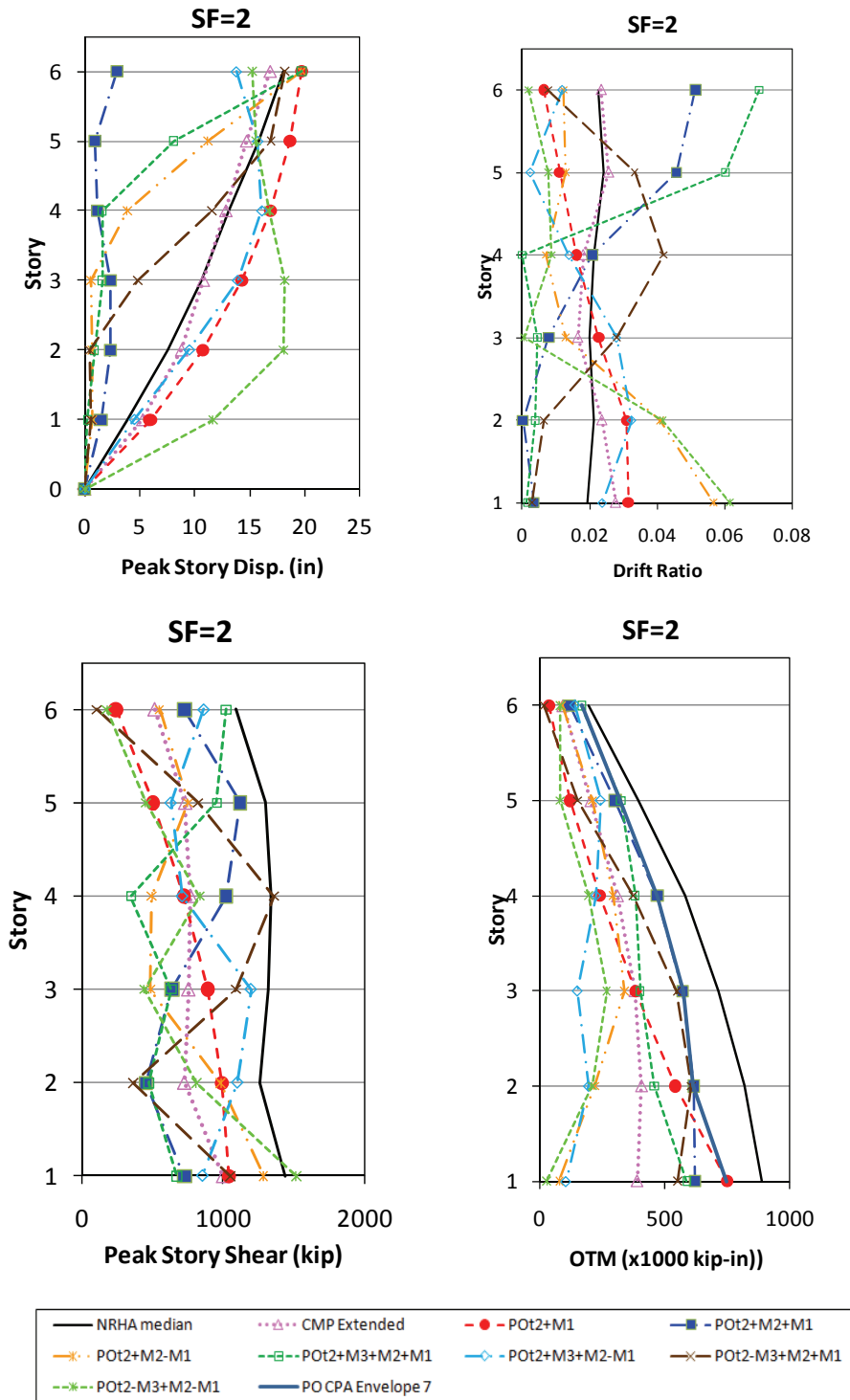


Figure 6-16 Comparison of Extended Consecutive Modal Pushover analysis results for Building A using different modal pushover sequences.

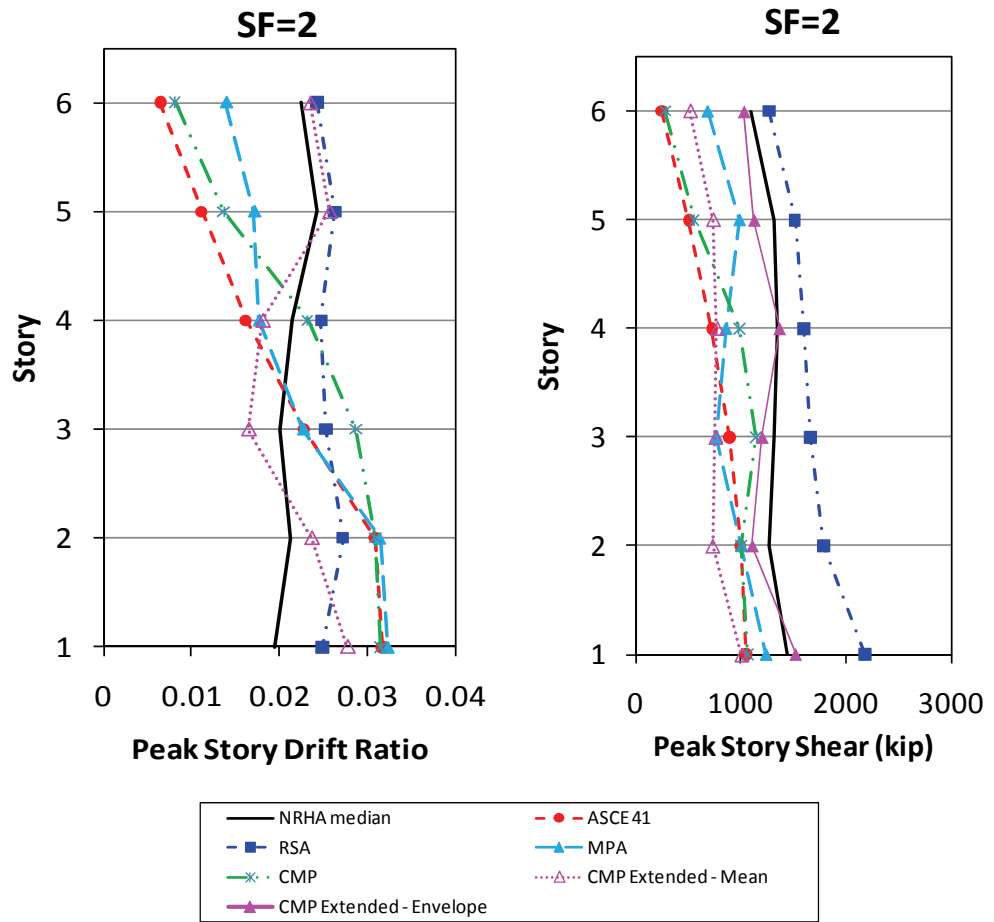


Figure 6-17 Comparison of peak story drift ratio (left) and peak story shear (right) for Building A from different analysis options.

- When applied to Building B, the method of computing the target displacements resulted in an unrealistically high value for the second mode target displacement which distorted the results. This distortion became more apparent in the Building B Extended CMP analyses, which failed to converge. Further investigation is needed to better understand potential limitations in the application of the Extended CMP concept.

6.3.7 Summary Response Quantity Ratios for Building A and Building B

Evaluation of the accuracy of nonlinear static analysis methods relative to nonlinear response history analysis must be conditioned on the details of the structural system and the assumptions made in the analytical model. This section summarizes ratios of response quantity estimates to median results from nonlinear response history analysis for Building A and Building B, in a format that is suitable for comparison

with the primary focused analytical study results presented in Chapter 5. Results are shown for single-mode nonlinear static analysis (ASCE 41), elastic modal response spectrum analysis (RSA), Modal Pushover Analysis (MPA), Consecutive Modal Pushover (CMP), and Extended Consecutive Modal Pushover (Extended CMP).

Summary ratios are presented in Table 6-3 for Building A, Table 6-4 for Building B in the longitudinal (H1) direction, and Table 6-5 for Building B in the transverse (H2) direction. Values in the tables represent maximum deviations over the height of the structures investigated. A ratio less than 1.0 indicates an underprediction of median demands, and a ratio more than 1.0 indicates an overprediction of median demands relative to nonlinear response history analysis.

Variations in the ratio of peak response quantity estimates over height for Building A are shown in Figures 6-18 and 6-19. Variations in the ratio of peak response quantity estimates over height for Building B are shown in Figures 6-20 and 6-21.

Table 6-3 Ratios of Response Quantity Estimates for Building A

Scale Factor	Analysis Procedure	Response Quantity Ratios ¹			
		Disp.	Story Drift	Story Shear	OTM
1.0	ASCE 41	0.9 to 1.0	0.4 to 0.9	0.4 to 0.8	0.3 to 1.0
	ASCE 41 plus RSA higher modes	1.0 to 1.0	0.9 to 1.3	1.0 to 1.0	1.0 to 1.0
	RSA - all modes	1.0 to 1.0	0.8 to 1.1	1.0 to 1.1	1.0 to 1.0
	RSA-1 mode	0.9 to 1.0	0.4 to 0.9	0.3 to 0.8	0.3 to 1.0
	MPA	0.9 to 1.0	0.6 to 1.0	1.0 to 1.0	0.4 to 1.0
	CMP	1.0 to 1.1	0.9 to 1.1	1.0 to 1.0	1.0 to 1.0
2.0	ASCE 41	1.1 to 1.5	0.3 to 1.6	0.2 to 0.7	0.2 to 0.8
	ASCE 41 plus RSA higher modes	1.1 to 1.6	1.0 to 1.8	1.1 to 1.2	0.9 to 1.0
	RSA - all modes	1.1 to 1.2	1.1 to 1.3	1.2 to 1.5	1.0 to 1.3
	RSA - 1 mode	1.0 to 1.1	0.5 to 1.1	0.4 to 1.2	0.3 to 1.3
	MPA	1.1 to 1.6	0.6 to 1.7	0.6 to 0.9	0.4 to 0.8
	CMP	1.1 to 1.5	0.4 to 1.6	0.3 to 0.7	0.2 to 0.8
	Extended CMP	0.9 to 1.3	1.0 to 1.4	0.5 to 0.7	0.4 to 0.6
	Extended CMP- Maximum	N/A	N/A	0.9 to 1.1	0.8 to 0.9

¹ Values are the minimum and maximum ratios of estimated response quantity to mean value from nonlinear response history analysis.

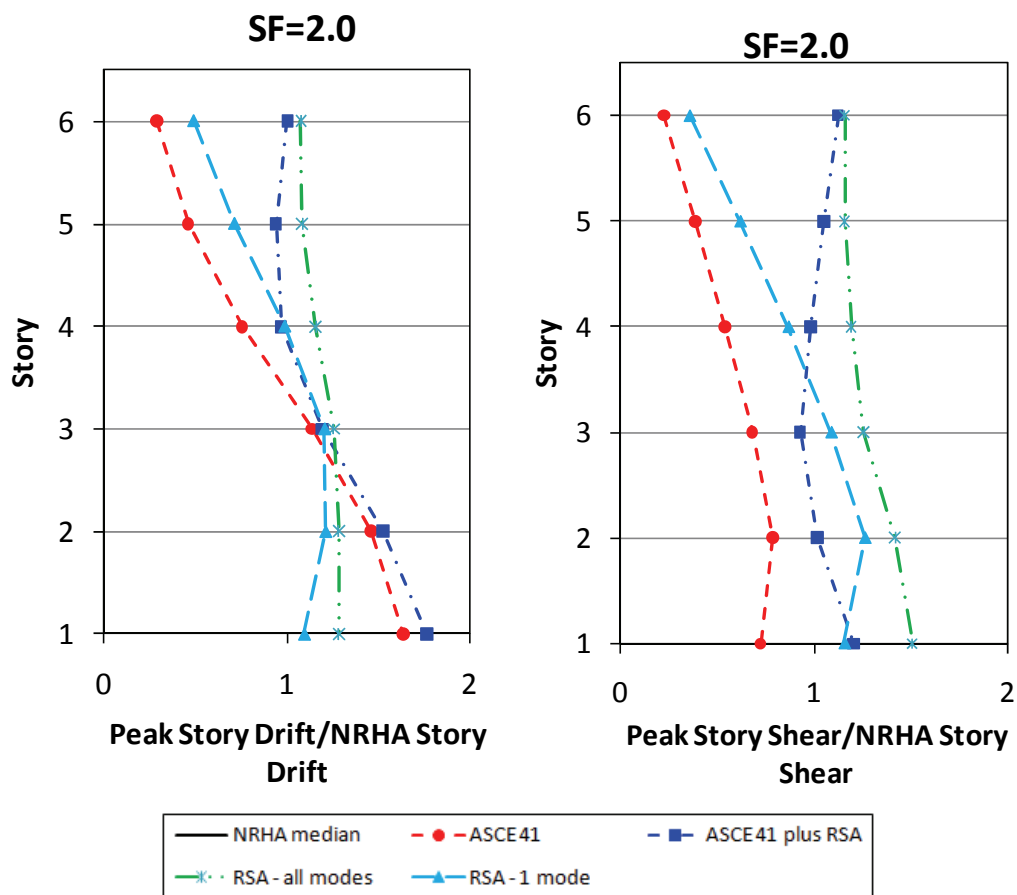


Figure 6-18 Variation in ratios of peak story drift and story shear over height from nonlinear static pushover and elastic response spectrum analysis of Building A.

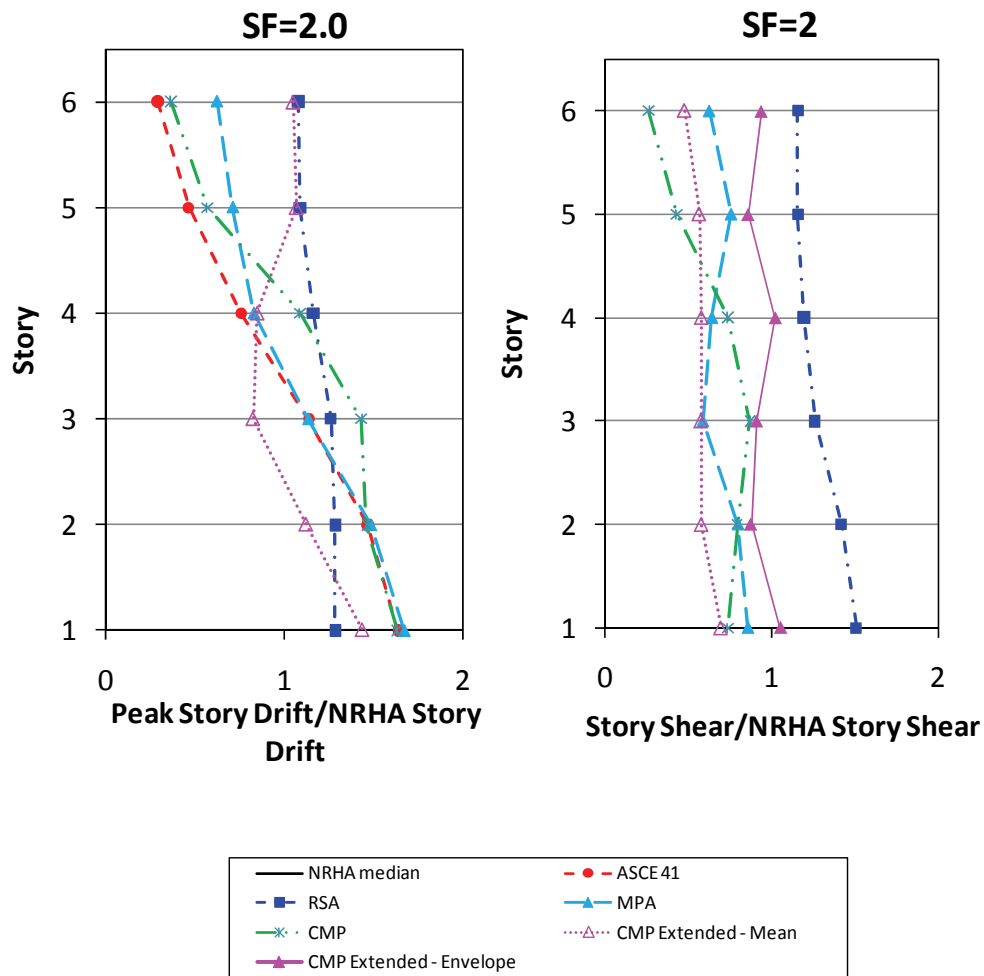


Figure 6-19 Variation in ratios of peak story drift and story shear over height for all analyses of Building A.

Table 6-4 Ratios of Response Quantity Estimates for Building B – Longitudinal (H1) Direction

Scale Factor	Analysis Procedure	Response Quantity Ratios ¹			
		Disp.	Story Drift	Story Shear	OTM
0.5	ASCE 41	0.9 to 0.9	0.8 to 0.9	0.8 to 0.8	0.7 to 1.0
	RSA - all modes	1.0 to 1.0	0.9 to 1.0	1.2 to 1.2	0.9 to 1.0
	ASCE 41 plus RSA higher modes	1.0 to 1.3	1.0 to 1.3	1.3 to 1.4	1.1 to 1.2
	RSA - 1 mode	0.9 to 0.9	0.9 to 0.9	1.1 to 1.1	0.9 to 0.9
	MPA	0.9 to 0.9	0.9 to 0.9	0.8 to 0.8	0.7 to 1.0
	CMP	0.9 to 0.9	0.8 to 0.9	0.8 to 1.0	0.9 to 1.0
1.0	ASCE 41	0.9 to 1.0	0.9 to 0.9	0.7 to 0.9	0.5 to 0.9
	RSA - all modes	0.9 to 0.9	0.9 to 0.9	1.4 to 1.7	1.0 to 1.3
	ASCE 41 plus RSA higher modes	1.1 to 1.3	1.1 to 1.3	1.4 to 1.6	0.9 to 1.2
	RSA - 1 mode	0.8 to 0.9	0.8 to 0.9	1.3 to 1.6	0.9 to 1.2
	MPA	0.9 to 1.0	0.9 to 0.9	0.8 to 1.0	0.5 to 0.9
	CMP	0.9 to 1.0	0.9 to 0.9	0.7 to 1.0	0.6 to 0.9
2.0	ASCE 41	0.4 to 0.6	0.4 to 0.7	0.8 to 1.1	0.4 to 1.0
	RSA - all modes	0.5 to 0.7	0.5 to 0.8	2.4 to 3.2	1.2 to 2.2
	ASCE 41 plus RSA higher modes	0.6 to 0.7	0.6 to 0.8	1.9 to 2.3	0.9 to 1.5
	RSA - 1 mode	0.4 to 0.7	0.4 to 0.8	2.2 to 2.9	1.1 to 2.1
	MPA	0.4 to 0.6	0.4 to 0.7	0.9 to 1.2	0.5 to 1.0
	CMP	0.5 to 0.8	0.5 to 0.7	0.8 to 1.1	0.4 to 1.0

¹Values are the minimum and maximum ratios of estimated response quantity to mean value from nonlinear response history analysis.

Table 6-5 Ratios of Response Quantity Estimates for Building B – Transverse (H2) Direction

Scale Factor	Analysis Procedure	Response Quantity Ratios ¹			
		Disp.	Story Drift	Story Shear	OTM
0.5	ASCE 41	0.3 to 0.6	0.3 to 0.8	0.3 to 0.9	0.2 to 0.7
	RSA - all modes	0.9 to 1.0	0.9 to 1.0	1.0 to 1.3	0.9 to 1.1
	ASCE 41 plus RSA higher modes	1.1 to 1.2	1.0 to 1.1	1.2 to 1.2	0.9 to 0.9
	RSA - 1 mode	0.4 to 0.7	0.4 to 1.0	0.4 to 1.3	0.5 to 1.1
	MPA	0.6 to 0.7	0.6 to 0.8	0.6 to 0.9	0.7 to 0.9
	CMP	1.0 to 1.7	0.8 to 1.7	0.9 to 1.7	0.7 to 1.5
1.0	ASCE 41	0.3 to 0.5	0.3 to 0.9	0.3 to 1.0	0.3 to 0.7
	RSA - all modes	0.9 to 1.1	0.9 to 1.3	1.1 to 1.9	1.1 to 1.4
	ASCE 41 plus RSA higher modes	1.1 to 1.2	1.0 to 1.1	1.3 to 1.4	1.0 to 1.1
	RSA - 1 mode	0.4 to 0.8	0.4 to 1.3	0.5 to 1.8	0.7 to 1.3
	MPA	0.6 to 0.7	0.6 to 0.9	0.7 to 1.1	0.7 to 0.8
	CMP	0.5 to 0.7	0.7 to 0.9	0.9 to 1.0	0.7 to 0.8
2.0	ASCE 41	0.0 to 1.0	0.0 to 0.2	0.3 to 1.0	0.3 to 0.6
	RSA - all modes	0.2 to 1.2	0.5 to 1.2	1.7 to 2.8	1.6 to 1.7
	ASCE 41 plus RSA higher modes	0.5 to 1.3	0.3 to 0.5	1.6 to 1.8	0.9 to 1.6
	RSA - 1 mode	0.5 to 1.2	0.2 to 1.2	0.8 to 2.8	1.0 to 1.7
	MPA	0.5 to 1.0	0.3 to 0.5	0.9 to 1.1	0.7 to 0.9
	CMP	0.1 to 1.0	0.1 to 0.3	0.5 to 0.9	0.4 to 0.6

¹Values are the minimum and maximum ratios of estimated response quantity to mean value from nonlinear response history analysis.

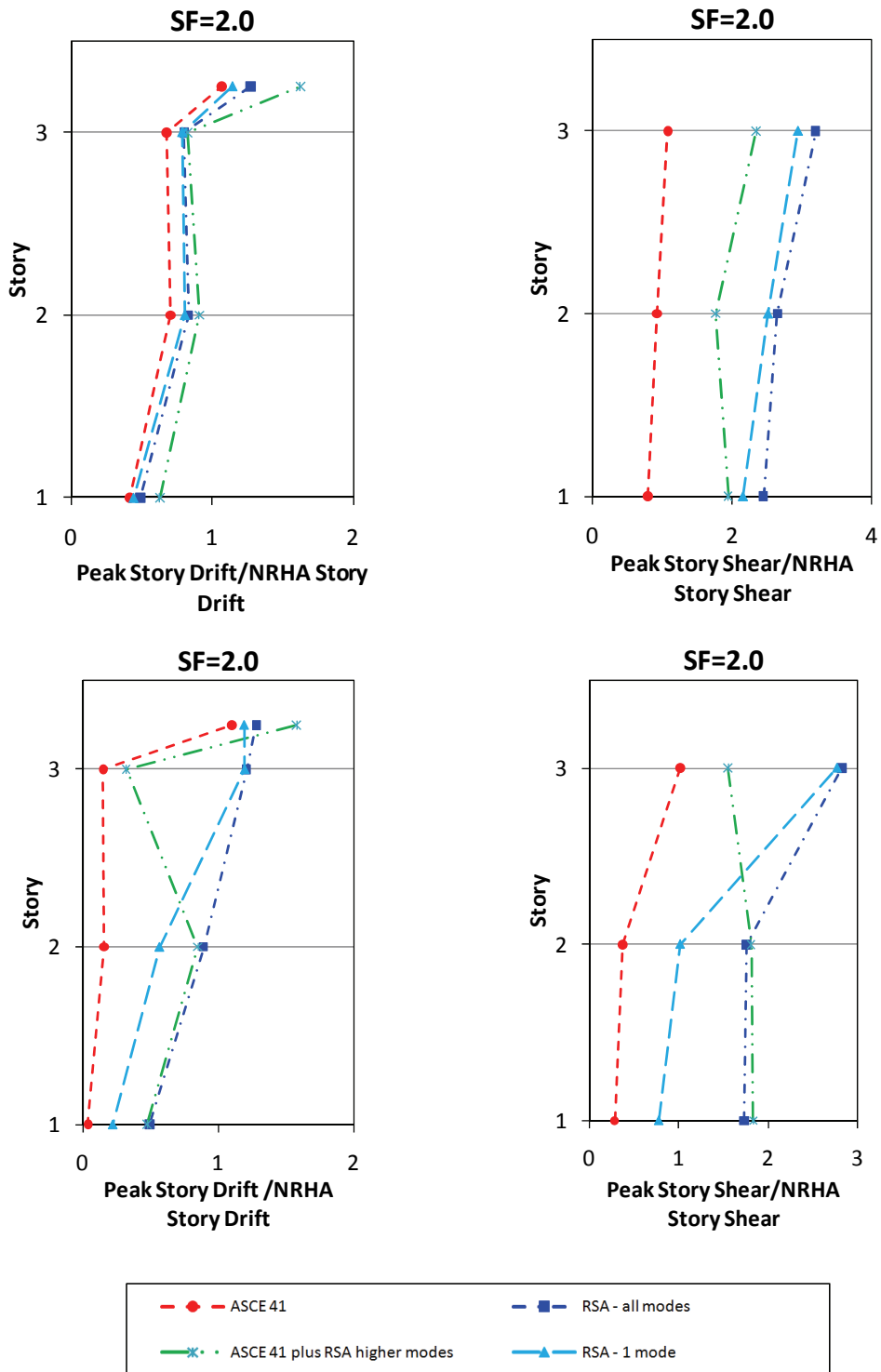


Figure 6-20 Variation in ratios of peak story drift and story shear over height from nonlinear static pushover and elastic response spectrum analysis of Building B in the longitudinal direction (top) and transverse direction (bottom).

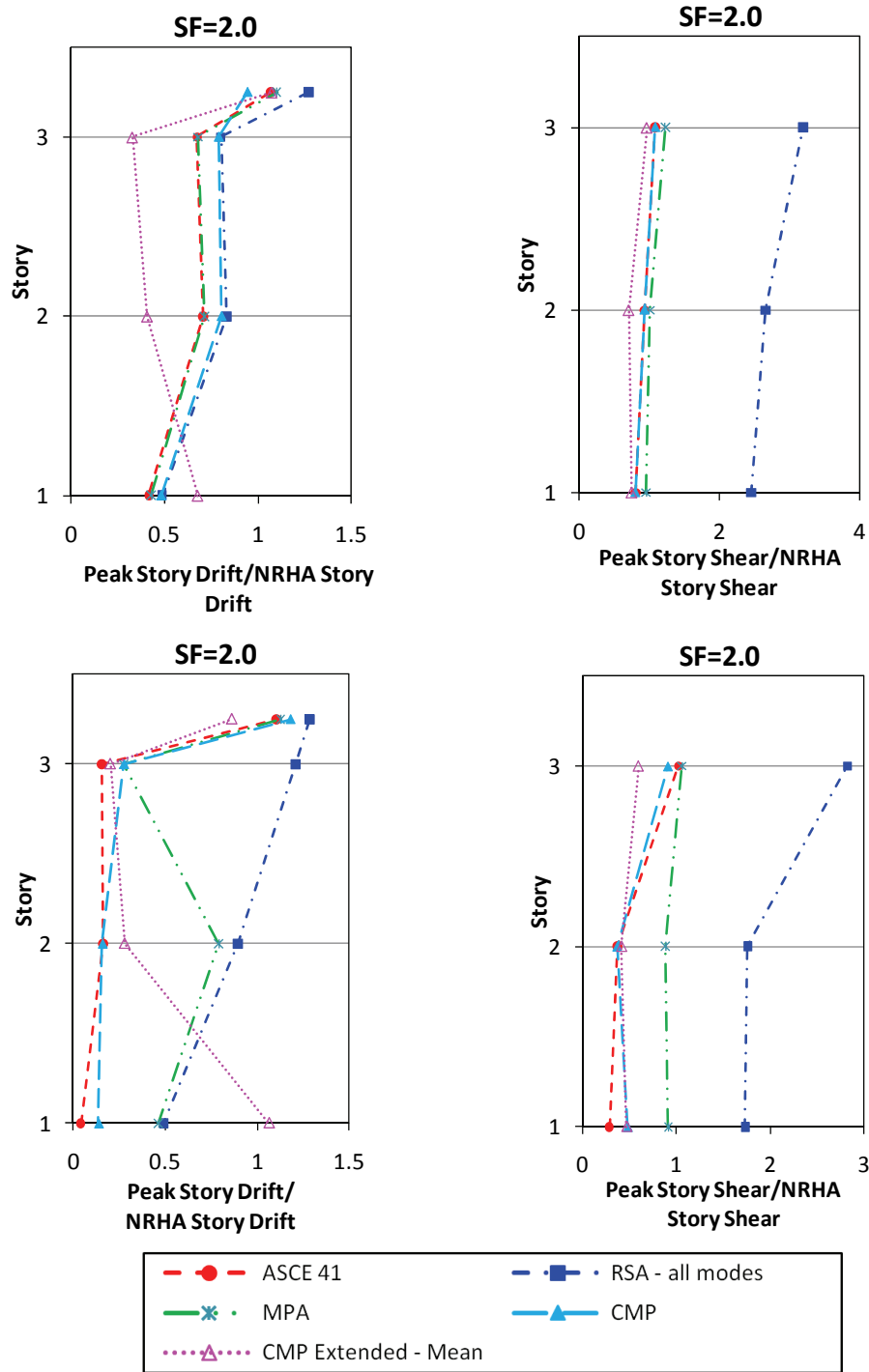


Figure 6-21 Variation in ratios of peak story drift and story shear over height from all analyses of Building B in the longitudinal direction (top) and transverse direction (bottom).

6.3.8 Performance Prediction

When judging the adequacy of analytical methods for design decisions, the relevance of the response quantity to design and performance should be considered.

Differences in analytically predicted response quantities can indicate changes in performance, but the significance of these differences should be considered in light of the acceptance criteria that define performance.

Explicit evaluation of ASCE/SEI 41-06 acceptance criteria was not performed, but predicted story drifts were used as a reasonable approximation for plastic hinge rotations in critical elements of Building A. Table 6-6 shows how story drifts predicted using each of the analysis methods might translate to performance predictions for Building A.

Performance is not always equally sensitive to variations in response quantity estimation. In the case of Building A, however, variations in predicted deformation levels led to corresponding changes in predicted building performance, and overestimated response quantities translated directly to underestimated seismic performance.

Table 6-6 Predicted Levels of Performance for Building A

Scale Factor	Analysis Procedure	Performance Level
2.0	Nonlinear Response History	Life Safety
	ASCE 41	Collapse Prevention
	ASCE 41 plus RSA higher modes	Collapse Prevention
	RSA - all modes	Life Safety
	RSA - 1 mode	Life Safety
	MPA	Collapse Prevention
	CMP	Collapse Prevention
	Extended CMP	Life Safety
	ASCE 41 Section 2.4.2.1 Combined ¹	Collapse Prevention

¹Worst prediction based on ASCE/SEI 41-06 nonlinear static procedure or RSA-all modes.

6.3.9 Summary Observations

Practical Implementation

The following challenges to practical implementation of nonlinear response history analysis were identified:

- More information is needed to perform nonlinear response history analysis than is needed for pushover analysis. Appropriate models must be selected and implemented for structural damping and cyclic hysteretic behavior, and proper calibration requires interpretation of experimental test data. At present, there is no set of uniformly applicable guidelines that provide this type of information to practitioners.
- Ground motion records must be appropriately selected and scaled and applied to the model. At present there is no standard set of procedures for selecting and scaling ground motions. The process of obtaining such records, in practice, can take weeks to months.
- A wide variety of structural analysis software is currently in use by practicing engineers, yet only a limited subset of these is capable of nonlinear response history analysis. Software that is capable of advanced analysis is often less capable of more basic structural analysis and/or design tasks. This can necessitate the development of multiple structural models.
- For a given analysis, nonlinear response history analysis requires more solution steps than pushover analysis. A nonlinear response history analysis model might, therefore, need to be smaller and less complex than that of a pushover analysis model in order to reach a stable solution.
- Some jurisdictions require peer review of nonlinear response history analysis.
- For the foreseeable future, it appears that practicing structural engineers will still need viable methods of simplified nonlinear analysis that are less complex than nonlinear response history analysis.

Findings Related to Primary Focused Analytical Studies

The following observations relevant to the scope of the primary focused analytical studies were made:

- Higher modes effects were found to be significant in the response of both structures. Pushover analysis was found to significantly underestimate story shear and overturning moment demands relative to nonlinear response history analysis.

- A single-mode pushover analysis could not capture the range of building deformation mechanisms that developed in different nonlinear response history analyses.
- The linear response spectrum analysis method produced displacement and drift quantities that were reasonably consistent with those from nonlinear response history analysis as long as soft-story mechanisms did not form.
- Both buildings failed the check for significant higher mode effects contained in ASCE/SEI 41-06. As a result, the dual-analysis requirement to perform both nonlinear static and elastic response spectrum analysis would be in effect. The dual-analysis requirement improves the enveloping of force demands, combining results from separate nonlinear static and elastic response spectrum analyses has limitations with regard to capturing yielding actions in components.
- The Modal Pushover Analysis technique improved the prediction of some response quantities, but did not capture the formation of secondary yielding mechanisms at higher scale factors.
- The Consecutive Modal Pushover technique has the advantage that final component forces and deformations are always consistent with the inelastic backbone curve, but limitations related to the sequence and sign of the modal response combinations were identified.

Extended Consecutive Modal Pushover

Preliminary work on an extension of the Consecutive Modal Pushover method concluded the following:

- By varying the sequence and sign of the modal responses, different response and deformation mechanisms are excited in the structure. For both buildings studied, the observed deformation mechanisms included all those found to occur in the corresponding suite of nonlinear response history analyses.
- It is proposed that each CMP analysis be treated as a separate “earthquake” and the results of the analyses be averaged for deformations and enveloped for forces.
- The Extended CMP approach produced good agreement for one building, but for the second building the results were inconsistent due to overestimation of the higher mode component of the target displacement. Further investigation is needed to better understand potential limitations in the application of the Extended CMP concept.

Modeling Recommendations for Nonlinear Analysis of Multiple-Degree-of-Freedom Effects

The NIST GCR 10-917-5 Report, *Nonlinear Structural Analysis for Seismic Design: A Guide for Practicing Engineers* (NIST, 2010a) provides an overview of the issues that should be considered when nonlinear analysis is contemplated. It covers topics including backbone curves and deterioration, effective stiffness, damping, and soil-structure interaction. It also makes reference to other documents that provide the latest technical recommendations for nonlinear modeling.

Since all modeling decisions can be shown to have some effect on predicted response, much of the guidance in that report can be considered relevant to modeling of nonlinear multiple-degree-of-freedom (MDOF) effects. This chapter provides additional modeling recommendations that were developed on the basis of literature reviewed and analyses performed over the course of this project.

7.1 Representation and Effect of Geometric Nonlinearity on Story Strength Demands

Geometric nonlinearity in seismic-force-resisting systems can include both P-Delta (capital Delta) and P-delta (lower case delta) effects, caused by loads acting on the deformed configuration of the structure. System P-Delta is related to the effects of gravity loads acting on the displaced location of the joints. Member P-delta is concerned with the effects of loads acting on the deflected shape of a member between joints. Currently available engineering codes and standards typically require that all second-order effects be included in the analysis used to proportion the members. In particular, guidelines are provided to determine when P-delta effects may typically be neglected in the analysis (e.g., AISC, 2005a). Modeling strategies for including both P-Delta and P-delta effects within the analysis are available in the literature, and strategies for analysis, including use of moment amplification or direct second-order elastic analysis, may be used. This section focuses on modeling of system P-Delta effects.

From a static perspective, the P-Delta effect is equivalent to an additional lateral loading that increases member forces and lateral deflections, reduces the lateral load resistance of the structure, and causes softening (i.e., a negative slope) in the lateral load-displacement relationship at large displacements. From a dynamic perspective,

the structure P-Delta effect can lead to significant amplification in displacement response if the displacement demands are large enough to enter the range of negative tangent stiffness. A more detailed discussion of the consequences of P-Delta effects in seismic response is presented in PEER/ATC-72-1 (2010), Adam et al. (2004), Bernal (1998), Gupta and Krawinkler (2000), Ibarra and Krawinkler (2005), and Lignos and Krawinkler (2009).

The importance of P-Delta varies with the structural configuration, stiffness of the seismic-force-resisting system, and the extent of inelastic behavior. With the current availability of second-order analysis tools, these effects may be included in any nonlinear analysis, whether static or dynamic.

The P-Delta issue is illustrated in Figure 7-1 for a two-dimensional model of a structure. Figure 7-1 shows that there are two contributions to P-Delta that should be treated separately. One is the effect of gravity loads that are applied directly to the seismic-force-resisting system (direct P-Delta effect), and the other is the effect of gravity loads that are transferred to the seismic-force-resisting system through the floor diaphragm (indirect P-Delta effect).

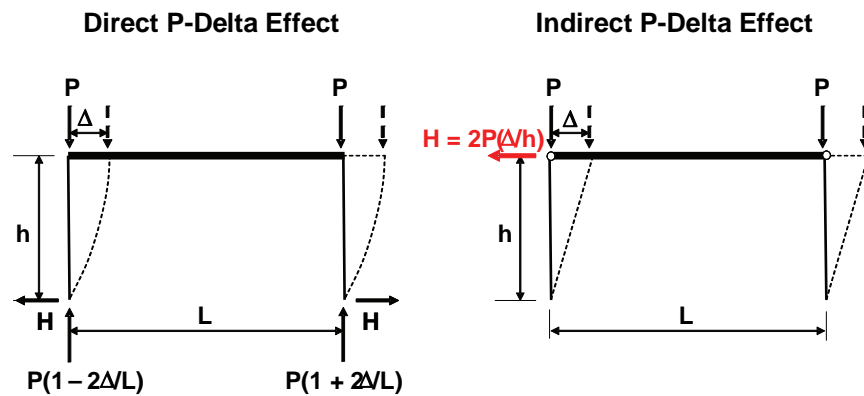


Figure 7-1 Illustration of direct and indirect P-Delta effects.

Direct P-Delta effects cause self-equilibrating shear forces and differences in axial forces in the columns, as illustrated in Figure 7-1. If the columns are sufficiently slender, member P-delta effects should also be modeled due to these directly applied loads. In this illustration, the lateral load that causes horizontal deflection and its first order reactions are not shown. Indirect P-Delta effects, which are transferred to the seismic-force-resisting system through the floor diaphragms, can be represented by a leaning column that transfers the horizontal loads needed to establish equilibrium through links to the seismic-force-resisting system. Direct and indirect P-Delta effects have similar but not identical consequences.

When incorporating P-Delta effects in an analytical model, it is recommended that these two effects are treated separately. Thus, gravity loads tributary to the seismic-force-resisting system should be applied directly to the seismic-force-resisting

elements. A set of one or more leaning columns should be incorporated in the analytical model if a significant portion of the gravity loads tributary to the seismic-force-resisting system must be transferred through the diaphragms. It is common to use a leaning column to represent the indirect P-Delta effect unless the entire gravity system is being modeled directly. The leaning column should have very large axial stiffness and negligible bending stiffness and should be connected to the seismic-force-resisting system at all floor levels by means of an axially-stiff link element. With a bit more effort, the leaning column could be assigned the appropriate bending stiffness and strength of all gravity columns, in order to simulate the beneficial effects of gravity columns to lateral resistance. It is quite feasible to incorporate all important components of the gravity system (gravity columns, beams, and connections and/or slab) explicitly in the analytical model. The major challenge in this effort will be appropriately modeling the contributions of connections and floor slabs to lateral stiffness and strength.

When assessing the effects of P-Delta on story shear forces, a clear distinction needs to be made between story shear forces that are in equilibrium with externally applied loads (in case of a pushover analysis) or inertia forces (in case of a nonlinear response history analysis), and story shear forces that are relevant for design of columns, walls or bracing systems. The former are referred to here as V_I , obtained by cutting a section through the full analytical model including the leaning column, and the latter, which include the effects of P-Delta on story shear forces, are referred to here as $V_{I+P-\Delta}$. These are obtained by cutting a section through the seismic-force-resisting system alone and omitting the P-Delta column. For a moment frame structure this process is illustrated in Figure 7-2. For design and evaluation of force demands for individual columns (or shear walls or bracing systems), the quantity $V_{I+P-\Delta}$ is much more relevant than V_I . For this reason, emphasis in this report is placed on $V_{I+P-\Delta}$ and the corresponding overturning moments, $OTM_{I+P-\Delta}$, which control the demands on axial forces in columns and bending strength of shear walls.

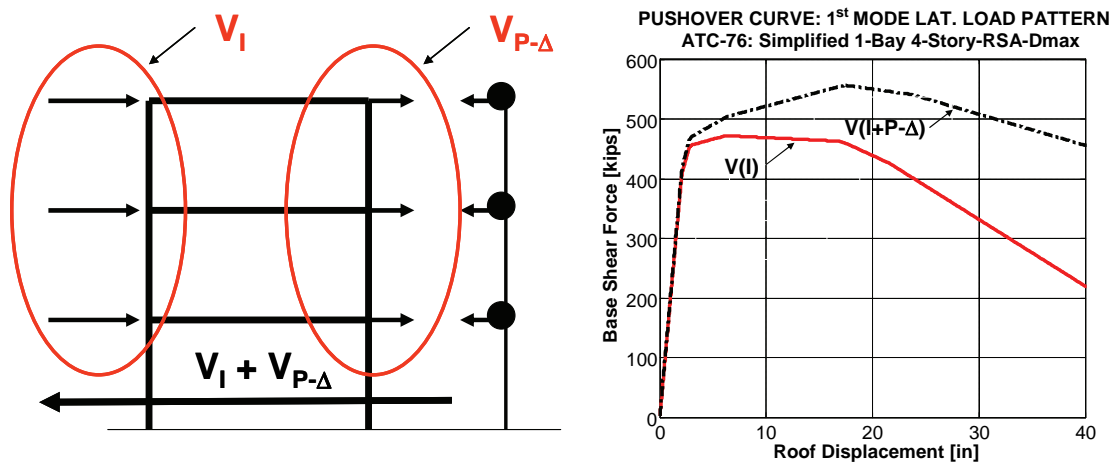


Figure 7-2 Differences between V_I and $V_{I+P-\Delta}$.

The importance of distinguishing between direct and indirect P-Delta effects is illustrated in Figures 7-3 and 7-4 for a 4-story steel moment frame. Figure 7-3 presents results for base shear versus roof drift relationships (V_I and $V_{I+P-\Delta}$), which shows the difference between assigning all gravity loads to the P-Delta column versus assigning the tributary gravity loads to the moment resisting frame. The results are identical for V_I (excluding P-Delta shear) but differ by about 5% for $V_{I+P-\Delta}$. These differences are minor, but proper assignment of gravity loads would eliminate them.

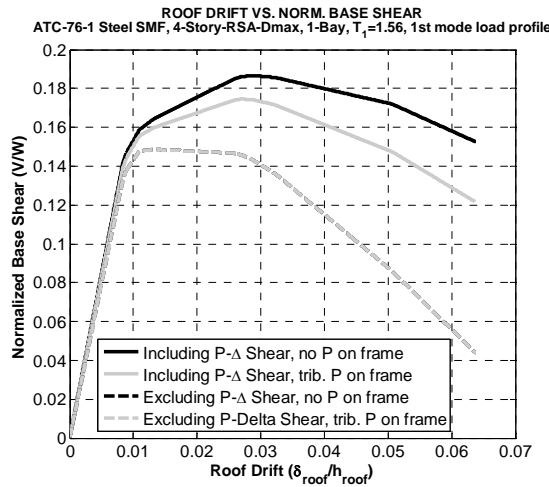


Figure 7-3 Influence of direct P-Delta effect on base shear of 4-story steel moment frame.

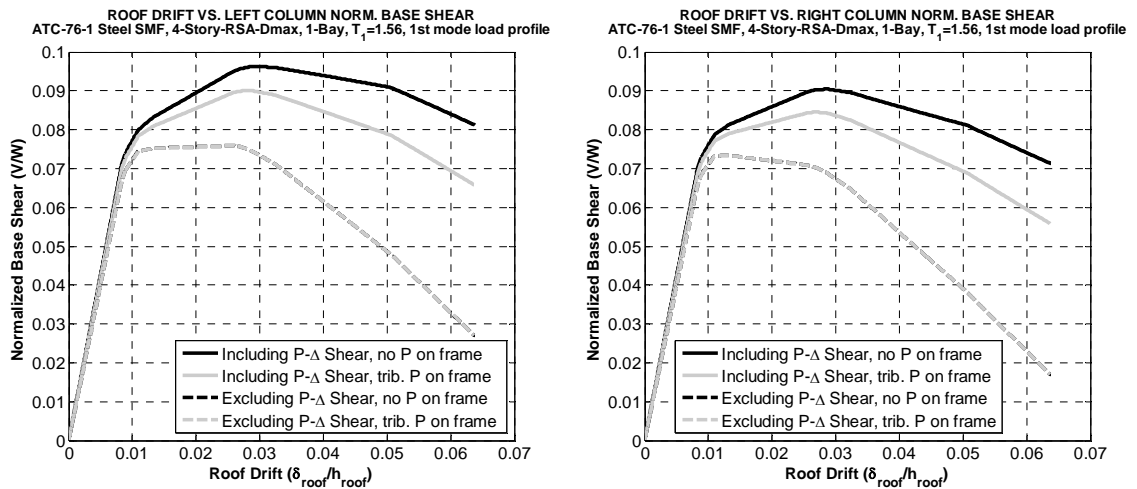


Figure 7-4 Influence of P-Delta effect on first story column shears (left and right columns) of 4-story 1-bay steel moment frame.

Figure 7-4 presents results for column shear versus roof drift relationships for the two columns of the 1-bay frame, obtained by applying lateral loads to the structure from left to right. It can be seen that the left column attracts a larger shear force than the

right column (again only by a few percentage points) because of direct P-Delta effects that increase the column shear force in the left column (see Figure 7-1).

In summary, P-Delta effects should always be represented in nonlinear analysis. They may not be important in the elastic region, but they become more important in the post-yield region and become very important in the post-capping region. A reasonable approach is to place tributary gravity loads directly on the seismic-force-resisting elements and to place the remaining gravity loads on a leaning column.

7.2 Torsion

Torsion is a degree of freedom in structural response that is difficult to address in nonlinear modeling. General observations on torsional effects, applicable to structures with very stiff (essentially rigid) diaphragms are provided below. These observations are based on Fajfar et al. (2008) and Perus and Fajfar (2005).

- Even if higher modes are not important and the structure responds elastically, several parameters are needed to define the elastic seismic response of asymmetric single-story buildings with a stiff diaphragm and mass concentrated at the level of diaphragm: (1) three uncoupled frequencies; (2) radius of inertia (related to torsional rotation); (3) and eccentricities in two horizontal directions, defined as the distances from the center of mass to the center of rigidity. Given that these quantities are known, torsional effects are usually well captured by linear response spectrum analysis (RSA).
- Displacements at the mass centers of an asymmetric structure are usually smaller than the displacements of a corresponding symmetric structure. The reason for this is smaller effective mass related to translation in the case of vibration of an asymmetric structure. However, due to the torsional rotation, the displacements at flexible edges are generally amplified, whereas the displacement at stiff edges may be either reduced or amplified, depending on the characteristics of the structure.
- The most important structural parameters are two frequency ratios, defined as the uncoupled torsional frequency divided by the uncoupled translational frequencies for the two horizontal directions. Structures can be classified as torsionally stiff (ratios greater than 1, first two modes are translational) or torsionally flexible (ratios smaller than 1, first mode is torsional). Typical dynamic behavior of a torsionally stiff structure is qualitatively similar to the static response, i.e., displacements are larger at the flexible edge and generally smaller at the stiff edge. The seismic response of torsionally flexible buildings is qualitatively different from that observed in torsionally stiff buildings and from that obtained in the case of static loading in mass center. Due to torsion, displacements at the flexible sides generally increase, as in the case of torsionally stiff structures. But

displacements at stiff sides are generally also larger than those of the symmetric structure. In many cases, torsional amplification at all sides may be very large.

- In the case of an inelastic system, the following additional parameters influence response: (1) relative strength of vertical components and strength irregularities and (2) ratio of strength in two orthogonal directions with the intensity of ground motion in the corresponding direction, which determines the extent of plastic deformation. The number of parameters increases considerably in the case of multi-story buildings. Consequently, it is practically impossible to study the influence of all possible combinations of parameters, except on a case-by-case basis.
- There is no general conclusion that can be drawn with confidence on response differences between elastic and inelastic systems. Moderate differences have been observed provided nonlinearities are minor or proportional in each vertical line of resistance. In many cases it was observed that the influence of torsion decreases with an increase in the intensity of ground motion and with correspondingly increased inelastic deformations.
- Strength irregularities are generally found to be more important than stiffness or mass irregularities.
- In the case of torsionally flexible structures, dynamic and static responses are qualitatively different, and pushover analysis may not provide adequate results. The difference between static and dynamic results may be unacceptably large. Thus, the value of pushover analysis for torsionally flexible structures is questionable.
- For torsionally stiff structures, pushover analysis yields qualitatively reasonable displacements, which are larger at the flexible sides than at the stiff sides. The displacements are usually underestimated at flexible sides, provided that the target displacement at the mass center for the pushover analysis is equal to peak displacement in dynamic analysis and the lateral loading is applied at mass center. At stiff edges, the results of a pushover analysis may be considerably different from those obtained by dynamic analysis. Known limitations in the ability of two-dimensional pushover analyses to capture higher mode effects are applicable in the case of torsion. Unless a suitable lateral load pattern is used, the seismic response in the upper parts of structures is often underestimated.

Bento and Pinho (2008) concluded that there is no one simplified three-dimensional analysis that works in all cases. A simplified approach is still needed, and the following approach proposed by Fajfar et al. (2008) is suggested for low-rise structures (without important vertical higher mode effects).

The approach is based on the following assumptions: (1) an upper limit for torsional effects can be estimated by a linear spectral analysis; and (2) favorable torsional effect on the stiff side, i.e., any reduction of displacements compared to the counterpart symmetric building, which could arise from elastic analysis, will diminish in the inelastic range. The approach requires execution of three-dimensional single mode pushovers and elastic three-dimensional modal analysis as follows:

1. Perform pushover analyses using a three-dimensional analytical model. Loading is applied at the mass centers, independently in two horizontal directions, in each direction, denoted with + and -. Determine the target displacement (displacement demand at mass center at roof level) for each of two horizontal directions (the larger value of two values) from an equivalent single-degree-of-freedom system. The ASCE/SEI 41-06 coefficient method can be used if the target displacement is not in the negative tangent stiffness region of the global pushover curve.
2. Perform an elastic modal analysis of the three-dimensional analytical model, independently for excitation in two horizontal directions, and combine the results using the square-root-sum-of-the-squares method.
3. Determine correction factors to be applied to the relevant results of pushover analyses. The correction factor is intended to take into account the torsional effects determined by the elastic modal analysis performed in step 2. It is defined as the ratio between the normalized roof displacements obtained by elastic modal analysis and pushover analysis. The normalized roof displacement is the roof displacement at an arbitrary location divided by the roof displacement at the mass center. If the normalized roof displacement obtained by elastic modal analysis is smaller than 1.0, the value 1.0 is used, i.e., no deamplification due to torsion is taken into account. Correction factors are defined for each horizontal direction separately.
4. Determine seismic demand by multiplying all relevant quantities obtained by pushover analyses with appropriate correction factors. For example, in a perimeter frame parallel to the X-axis, all quantities are multiplied with the correction factor determined with pushover results obtained for loading in the X-direction and for the location of this frame. The relevant quantities are, for example, story drifts, story shears, and overturning moments, or localized deformations in structural components.

The results obtained by this procedure are influenced both by nonlinear static pushover and elastic dynamic analysis. Displacement demand (amplitude and the distribution along the height) at the mass centers is determined by the usual nonlinear

static procedure. The amplification of demand due to torsion is determined by elastic dynamic analysis, while reduction of demand due to torsion is not taken into account.

There are many simplifications involved in this approach. Like most other pushover methods, it permits visualization of response and its progression from small loads to loads associated with the target displacement. There are alternatives to this approach, such as the extension of the modal pushover analysis to unsymmetric plan buildings and three-dimensional analysis (Chopra and Goel, 2004, Reyes and Chopra, 2010), and the extension of the Adaptive Capacity Spectrum Method developed by Casarotti and Pinho (2007) to asymmetric buildings (Bento et al., 2008).

7.3 Gravity Framing

It usually is left up to the engineer whether or not to explicitly include contributions of the gravity system to lateral stiffness and strength in the analytical model. The general recommendation is to incorporate the gravity system explicitly, which can have the effect of decreasing drift demands and increasing collapse capacity. This might be particularly attractive if the pushover curve exhibits an early negative tangent stiffness that may lead to large displacement amplification or even collapse. Incorporation of the gravity system will cause the negative stiffness to be reduced and could make it positive.

If strength and deformation capacity of individual components of the gravity system are not of concern, a simple way to incorporate the gravity framing is by means of a fishbone arrangement of the type shown in Figure 7-5. If the effects of axial load on the bending strength of the column are incorporated in the properties of the fishbone model, then the column of the fishbone can also be utilized as the leaning column for modeling of P-Delta effects.

An arrangement with two half-beams is preferred because it prevents accumulation of a large axial force in the spine (column) of the fishbone. In this arrangement all beams are lumped into a single beam (I/L of beam = $\sum EI_i/L_i$ of all beams), all columns are lumped into a single column (I of column = $\sum I_j$ of all columns), and all gravity connections are lumped into two connections represented by rotational springs. Modeling of connection strength and stiffness is often a major challenge and should be done conservatively.

Benefits derived from incorporating the gravity system in the analytical model may range from small to significant, depending on structural configuration. In general, the larger the ground motion intensity, the larger is the reduction in demand parameters due to the gravity system. Stiffness and strength of the gravity system will reduce the collapse potential of structures, provided that the gravity system has sufficient deformation capacity to support gravity loads at large inelastic deformations without deterioration in strength.

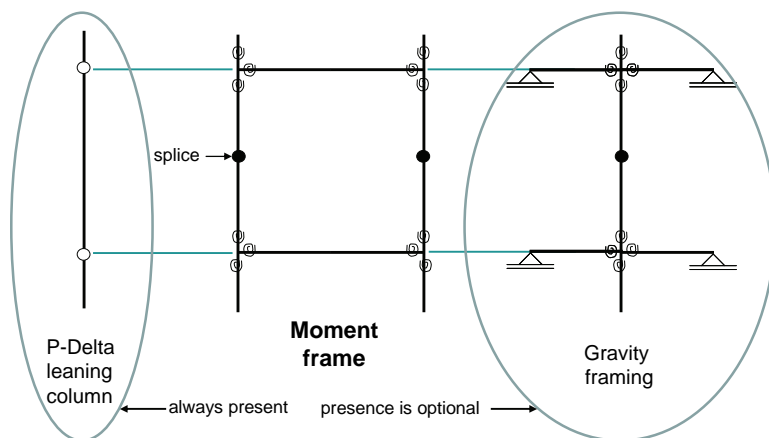


Figure 7-5 Model of a moment frame structure, illustrating simplified modeling of P-delta effects and fishbone modeling of gravity framing.

7.4 Other Simplifications

This section discusses additional simplifying approximations that can be used in nonlinear modeling of MDOF systems.

7.4.1 Two-Dimensional Models

Perhaps the most common simplification is the use of two-dimensional representations of three-dimensional structures. Nearly all of the focused analytical studies on this project used this simplification. While two-dimensional analysis improves computational efficiency and eases interpretation of results, simplified approaches can limit the applicability of the results. It also means that conclusions cannot be extended directly to systems with pronounced torsional response or complex bi-dimensional interactions.

7.4.2 Beam-Column Joint Modeling

A common assumption for steel moment frame modeling is that beams and columns can be represented by centerline dimensions and panel zone shear deformations can be ignored. For frames in which panel zones are sufficiently strong to develop the bending strength of the beams framing into the joint, the errors involved in making these two approximations often compensate each other. On the other hand, frames with weak panel zones should not be approximated in this manner. Instead, explicit panel zone modeling, such as that described in the PEER/ATC-72-1 report (PEER/ATC, 2010), is necessary.

For concrete moment frames, the behavior of the beam-column joint region can be even more complex due to bond slip of longitudinal beam and column bars in the joint region. Supplement 1 to ASCE/SEI 41-06 provides useful guidance on both explicit and implicit modeling of concrete joint behavior.

7.4.3 Single-Bay Frame Models

Single-bay frame models involve the representation of multiple bays of a moment-resisting frame using a single bay. This simplification was employed in the steel moment frame analytical studies. It is applicable when all bays of the moment frame are of about equal width and stiffness and joint panel zones are not dominating strength characteristics of the system. A simplified single-bay system is shown in Figure 7-5.

Multi-bay frames can be approximated by a single-bay frame by setting $\Sigma EI_i/L_i$ and ΣM_p of all beams equal to EI/L and M_p of the 1-bay beam, respectively, and setting ΣEI_i and ΣM_{pc} of all columns equal to $2EI$ and $2M_{pc}$ of the 1-bay columns, respectively. For taller frames in which overturning moment and axial deformations in columns are important, these effects can be approximated by setting L of the 1-bay frame equal to the distance between end columns of the multi-bay frame and by setting the area of the 1-bay column equal to the area of the end column of the multi-bay frame. This simplification is based on the assumption that overturning effects are resisted mostly by the two end columns. The accuracy of these approximations decreases when the spans of the moment frame vary considerably.

Figure 7-6 compares pushover curves for 3-bay and 1-bay models of the 4-story and 8-story steel moment frame structures described in Appendix A. The graphs show that the simplified 1-bay model captures accurately the global characteristics of the 3-bay structure.

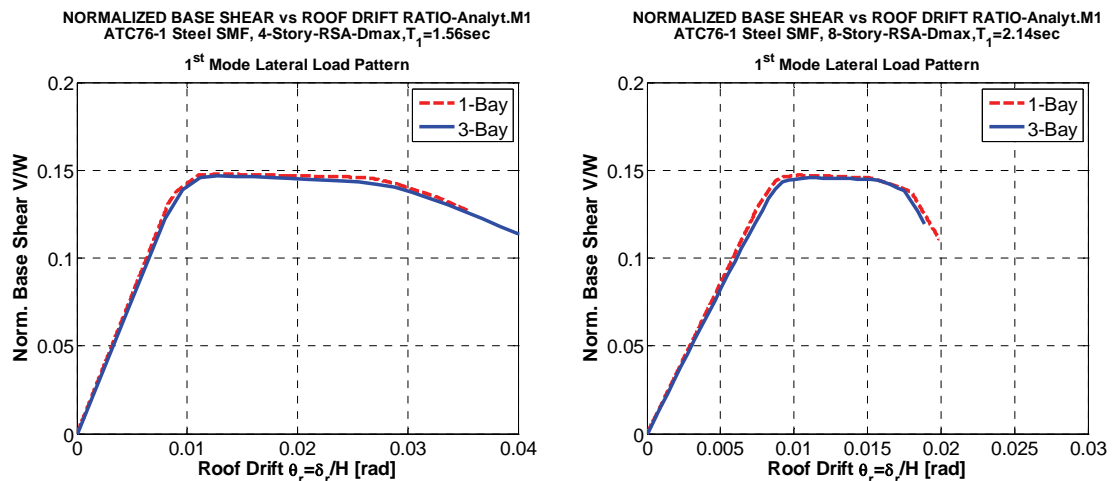


Figure 7-6 Comparison of global pushover curves ($V_i/W - \theta_r$) of 4-story and 8-story 1-bay and 3-bay steel moment frame analytical models.

Figure 7-7 compares nonlinear response history analysis predictions for a specific ground motion, obtained from a Drain-2DX 3-bay model and an OpenSees 1-bay model of the 4-story steel moment frame. The roof displacement history responses

and the base shear history responses are almost identical, providing confidence in the ability to represent the response of the 3-bay frame in a simplified 1-bay model.

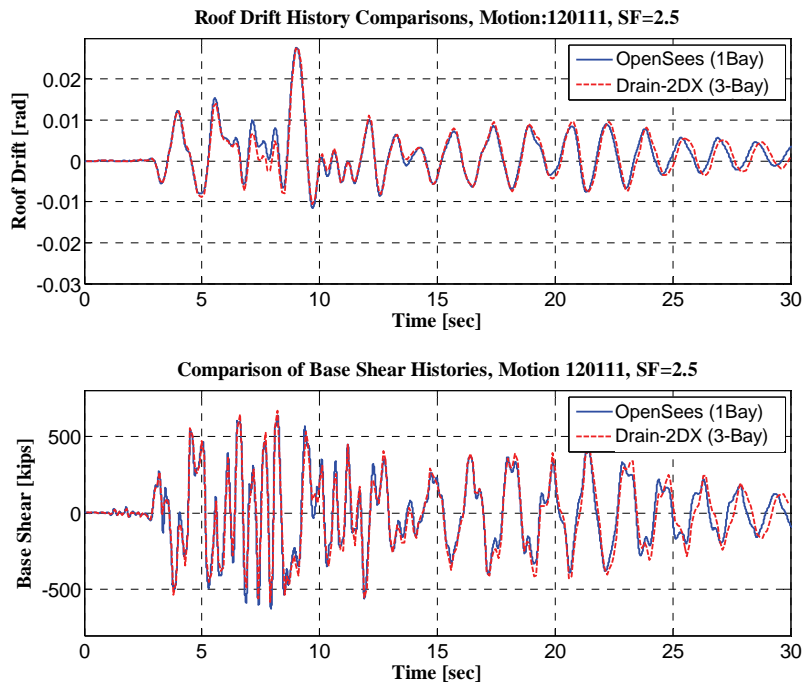


Figure 7-7 Comparison of nonlinear response history analysis results for 1-bay and 3-bay models of a 4-story steel moment frame.

Use of a 1-bay simplified model reduces the computational effort and often facilitates interpretation of global results. A simplified 1-bay model, however, will likely be inadequate for extracting local force and deformation demands, such as plastic hinge rotations and axial and shear forces in individual elements. Thus, the primary usefulness of simplified 1-bay models is to assess global and story-level demand parameters and to detect weaknesses that should become the subject of more detailed evaluation.

7.5 Suggestions for Further Study

There are a number of areas where focused study could result in improved modeling guidance for nonlinear analysis, such as development of a resource that provides detailed, specific, up-to-date technical recommendations for the full range of structures commonly encountered in practice. The following areas of further study related to nonlinear modeling are suggested:

- **System modeling:** Guidance on the required amount of below-grade stiffness and mass to be modeled is needed. The relation to the location and characterization of ground motion input should be clarified.

- **Component modeling:** It is important to determine the appropriate stiffness for cracked concrete in component modeling. Also, appropriate practical hysteretic models for various behaviors and the time to model demand parameter interactions (such as shear-flexure, shear-tension, and axial force-biaxial flexure) should be determined. Additional guidance on treating cyclic degradation is needed, including guidance on establishing the initial capacity boundary.
- **Damping modeling:** It is important to determine the magnitude of damping appropriate at element and system levels for various levels of response. The studies should consider what characterization of damping most appropriately reflects real response, and what damping models best approximate that response.
- **Soil-structure interaction:** Current recommendations concerning base slab averaging and other soil-structure interaction techniques to reduce higher mode effects need to be evaluated. In addition, foundation conditions that are required to justify base slab averaging and the most appropriate way to include these effects in response history analysis should be determined.
- **Ground motion:** Further study is needed to determine appropriate and practical methods for selection and scaling of ground motion records. These methods should provide guidance on how ground motion should be treated where multiple source types control first mode and higher mode response, and how the ground motions should be oriented with respect to the structural system.
- **Conditional mean spectra:** The use of conditional mean spectra in lieu of uniform hazard spectra to separate or reduce higher mode effects should be studied. Further studies should determine whether it could be appropriate to consider several different condition mean spectra, each peaking at a different mode and taking the maximum response, and whether this could be combined with base slab averaging.
- **Modal combinations:** In practice, the complete quadratic combination (CQC) and SRSS methods are most often used. More research is needed to determine whether other combinations would be more appropriate for nonlinear response.
- **Validation:** More assessments of the ability of analysis techniques to match recorded response of instrumented buildings are needed. In addition to studying the results of nonlinear response history analysis, examination of how well multi-mode pushover techniques perform is needed. A database of buildings damaged as a result of higher mode effects should be assembled and studied using various analysis methods to determine what methods can identify real performance problems.

Assessment of Nonlinear Analysis Methods

Engineering design decisions are concerned with component forces and deformation demands. Methods for prediction of response, whether local (e.g., component forces and deformations), or intermediate (e.g., story drifts and shears), or global (e.g., roof displacement or collapse), depend on the design objective. No attempt is made here to express preferences on analysis methods based on different design objectives. Instead, it is emphasized that good seismic design includes more than code-based quantitative demand assessment, usually represented by demand/capacity ratios. It includes good judgment and qualitative evaluation, in which exact numbers are less relevant than understanding of behavior. Simple nonlinear static analysis methods can provide valuable qualitative insight in the evaluation and design process, but care is necessary when they are used alone to establish design quantities.

Linear analysis techniques are widely used in practice but were not the focus of this project. In general, nonlinear analysis methods provide more useful and arguably more accurate results than linear methods. The amount of deviation from median values obtained from nonlinear response history analysis (NRHA) has been the subject of the exploratory studies summarized in Chapter 5 and presented in detail the appendices to this report.

This chapter is a compendium of summary observations drawn from these studies. These observations are case specific and are intended to aid in the decision process when and to what degree an engineer can rely on quantitative demand parameter predictions obtained from a nonlinear static analysis. These studies are not, and cannot be, considered comprehensive because of the large variability in system configurations and design decisions, all of which will affect the deviation between analytical methods to varying degrees.

There are many additional options for executing a nonlinear static analysis, some of which are summarized in Chapter 3. Many of these methods have clear advantages in specific applications, but all involve additional computational effort, and none have been found to be generally applicable for all conditions.

8.1 Assessment of Nonlinear Static Procedures

By definition, nonlinear static procedures cannot, except in very simple cases, provide accurate demand predictions for dynamic loading. Many attempts have been

reported in the literature to account for these effects, but in most cases the effort needed for implementation defeats the purpose of simplified analysis. Justification for the use of a nonlinear static procedure depends on the procedure employed, the type and configuration of the structural system, the degree of inelastic response expected, and the objectives of the analysis effort.

In general, nonlinear static analysis is always useful and should be part of every evaluation process concerned with inelastic behavior, whether or not the objective is rigorous quantification of demand parameters. The following is a list of situations where nonlinear static analysis is valuable:

- Checking and debugging a nonlinear analysis model
- Evaluating the fairness of modeling assumptions
- Augmenting understanding of the yielding mechanisms and deformation demands
- Understanding of behavior and of existence of an adequate load path
- Investigating alternative design parameters and the effect of variations in the component properties on inelastic response
- Augmenting understanding of behavior close to collapse
- Providing an estimate of the lateral strength of the structure
- Estimating structure overstrength in relation to seismic design loads
- Providing information to help establish a force-displacement capacity boundary of a structure in order to estimate global response characteristics such as roof displacement using an equivalent single-degree-of-freedom system
- Providing a global understanding of base shear versus roof displacement response, including estimates of post-yield stiffness and displacement at which the tangent stiffness becomes clearly negative
- Discovering locations of excessive deformation demands that should become the focus of more detailed study
- Discovering potential problems caused by overloading components with inadequate ductility
- Discovering potential problems caused by story-based strength and stiffness discontinuities
- Discovering potential problems caused by P-Delta effects and strength deterioration

The use of nonlinear static analysis methods for quantification of demand parameters and demand-capacity ratios is appealing in practice because this type of analysis produces a single, clear result without the need for selection and scaling of ground

motion records. However, this result will deviate from the median result obtained from nonlinear response history analysis with a set of representative ground motions. The amount of deviation depends on the importance of higher mode contributions and inelastic redistribution. Also, a single, deterministic answer will not provide information on all mechanisms that could potentially develop when the structure is subjected to a set of ground motions with different frequency characteristics. Finally, lateral dynamic instability cannot be addressed directly with nonlinear static analysis methods.

8.1.1 Single-Mode Nonlinear Static Analysis with Invariant Load Pattern

The differences in demand predictions between first mode nonlinear static and response history analyses can be judged from the demand ratios presented in Chapter 5 for each of the systems studied. The following general observations are made:

- Accuracy of demand predictions from single-mode nonlinear static analysis depends on system type and configuration, on the first mode mass participation (related to system type and number of stories), on the extent of inelastic deformations, on the relative spectral accelerations at the modal periods, on the sensitivity of story yield strength to the applied load pattern, on the variation of story strength and stiffness over the height, and on the mechanism controlling inelastic response.
- For the regular 2-story systems considered, response quantities from first mode nonlinear static analysis generally agree well with median results from nonlinear response history analysis.
- The deviation from the median of nonlinear response history analysis results increases with increasing height. Underestimation of response occurs mostly in the upper stories and varies significantly between story drift, story shear, and floor overturning moment responses.
- In general, estimates of demand parameters for reinforced concrete shear wall structures are better than those for moment frame structures of the same height, provided that the estimate of the target displacement is reasonable.
- The ASCE/SEI 41-06 coefficient method, which is based on an effective elastic stiffness, often will not result in a good estimate of the target displacement if the pre- and post-cracking stiffnesses of the global pushover curve are clearly different. In such cases it is recommended to compute the target displacement from nonlinear response history analysis of equivalent SDOF systems.
- If the target displacement is in the negative tangent stiffness region of the pushover curve, it is recommended to compute it from nonlinear response history analysis of an equivalent SDOF system. Prediction of target displacements from the ASCE/SEI 41-06 coefficient method is likely to be inadequate in such cases.

- The invariant load pattern might create a perceived weak story if the relative strength of stories is sensitive to the load pattern. If the pushover indicates an irregularity that concentrates drift in a single story, mechanics principles should be employed to estimate story strength in order to assess whether a weak story indeed exists or whether the perception of a weak story is created by the application of an invariant load pattern.
- If a strength irregularity exists and it is concentrated in a single story, then the first mode nonlinear static procedure is expected to provide good estimates of drift and force demand parameters.
- If strength irregularities exist in more than one story, it is unlikely that an invariant load pattern will be able to detect more than one irregularity. In such cases, nonlinear response history analysis is recommended.
- Pushovers with an invariant load pattern can be misleading regarding the dominant yielding mode. Studies have shown that the relative magnitude of bending moment to shear force (M/V ratio) can vary considerably during nonlinear dynamic response, and the base shear force can increase significantly due to dynamic amplification. Shear walls that would be expected to yield in flexure might actually yield in shear. In such cases, application of a shear amplification factor or nonlinear response history analysis is recommended.
- Collapse safety was not addressed specifically in the focused studies, but collapse is an important issue if the target displacement is in the negative tangent stiffness region of the pushover curve. FEMA P-440A (FEMA, 2009a) recommends the use of parameter R_{di} to safeguard against collapse if the target displacement becomes very large. This parameter is useful for the intended purpose, except that the third term in the equation can rarely be quantified with confidence for a global structure pushover curve. If all segments of the pushover curve are not well defined and if there is considerable uncertainty near collapse, it is conservative to eliminate the third term in the equation for R_{di} in evaluating lateral dynamic instability of the system.

8.1.2 Multi-Mode Nonlinear Analysis

The following multi-mode analysis procedures were studied in this project: elastic modal response spectrum analysis (RSA), Modal Pushover Analysis (MPA), Consecutive Modal Pushover (CMP), and Extended Consecutive Modal Pushover (Extended CMP). General observations are as follows:

- Elastic modal response spectrum analysis is useful for predicting deformation and force demand parameters, as long as the structure is in the elastic or near-elastic range. For inelastic systems, this method provided estimates of floor displacement and story drift that were as good as or better than those from nonlinear static analysis methods, but only for structures without strength irregularities. The method consistently overestimates story shears and

overturning moments for yielding systems. Elastic modal response spectrum analysis is valuable as a backup method for predicting story drifts for regular structures, but cannot be considered a generally applicable method for inelastic response prediction because of its inability to account for limited component strength that might control inelastic response.

- Use of Modal Pushover Analysis improved the prediction of demand parameters for regular structures.
 1. In many cases, Modal Pushover Analysis overestimated demands in the lower stories of the structure, but was better than the single-mode nonlinear static analysis in estimating demands in the upper stories.
 2. Modal Pushover Analysis, which uses invariant modal load patterns, does not protect against inaccuracies in drift that may be caused by load-pattern-sensitive story mechanisms in more than one story.
 3. In most of the regular structures investigated, second mode contributions were elastic, which simplifies the modal combination and avoids ambiguities that might be caused by displacement reversals sometimes observed in inelastic higher mode pushover analyses.
 4. When Modal Pushover Analysis is employed, it is possible to compute component forces from component deformations, as discussed in Goel and Chopra (2005). This may lead to an improved estimate of component forces but necessitates the execution of a second analysis phase.
- The Consecutive Modal Pushover analysis method as proposed by Poursha et al. (2009), which involves sequential loading with first and second mode load patterns, offered no consistent improvement over single-mode pushover analysis.
- The Extended Consecutive Pushover analysis method, developed in ancillary studies, in which sequence and sign (direction) of modal load patterns are varied, caused the formation of different inelastic mechanisms and replicated the mechanisms that develop in nonlinear response history analyses for various ground motion records.
 1. For the 6-story steel moment frame building studied, reasonable agreement between median results of nonlinear response history analyses was achieved using the mean Extended CMP result for displacement and drift quantities, and the maximum (envelope) Extended CMP result for story shear and overturning moment quantities.
 2. For the 3-story steel concentrically braced frame building studied, where multiple-degree-of-freedom effects included diaphragm flexibility, the Extended CMP did not produce acceptable agreement with the results of nonlinear response history analysis.

8.2 Assessment of Nonlinear Response History Analysis

In practice, appealing features of nonlinear response history analysis include the following:

- More realistic and explicit modeling of response is possible, producing more reliable assessment of seismic performance.
- Possible responses, and their corresponding probabilities, can be determined.
- Generation and application of artificial static load patterns is not required.
- The significance of cyclic loading and associated deterioration can be assessed directly.
- Lateral dynamic instability can be addressed directly.

However, these improvements come at a cost of additional effort and complexity of the procedure. In concept, the major differences associated with modeling for nonlinear response history analysis include the need for a representative set of ground motions and the need to model hysteretic behavior of structural components.

It is suggested that demand prediction from nonlinear response history analysis based on the simplest hysteresis diagram is more realistic than demand prediction from a nonlinear static analysis. Modeling of the “backbone curve” (or force-displacement capacity boundary) of individual components is needed one way or another, and the use of a simple component model becomes the core of facilitating the use of nonlinear response history analysis.

The PEER/ATC-72-1 report (PEER/ATC, 2010) provides recommendations for simple component modeling, such as the use of a bi-linear hysteresis model for steel structural components or a peak-oriented hysteresis model for reinforced concrete components. The associated backbone curve (or force-displacement capacity boundary) could include a post-capping range of negative tangent stiffness if this range is deemed to be important in response predictions. If the modified force-displacement model of ASCE/SEI 41-06 is used as the backbone curve (analysis Option 3 in PEER/ATC-72-1, illustrated in Figure 8.1), there would be no need to incorporate cyclic deterioration in the analysis since the tabulated properties of this model account for cyclic deterioration. Thus, the modeling effort for nonlinear response history analysis would not be much greater than that for nonlinear static analysis.

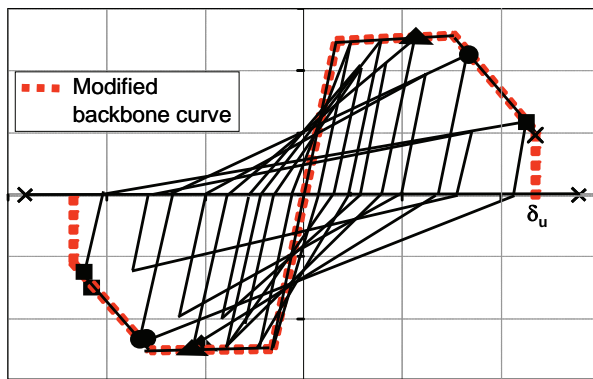


Figure 8-1 Component model for analysis Option 3 in PEER/ATC-72-1 (PEER/ATC, 2010), which accounts for cyclic deterioration.

8.3 Utility of Various Demand Parameters by System Type

As discussed in Chapters 5 and 6, the accuracy of approximate analysis methods is not uniform for all demand parameters and system types. Therefore, selection of analysis methods for use in either initial or final design must be based on consideration of the objectives of the analysis, the demand parameters of interest, and the characteristics of the structural systems being assessed.

Most of the available codes, standards, and guidelines prescribe component-level force and deformation limits that must be satisfied for every action of every component for final design. Because these demand parameters generally correlate well with intermediate-level parameters such as story drift, story shear, and floor overturning moment, earlier phases of design may often focus on intermediate-level demand parameters. Although not yet addressed by codes, standards, and guidelines used in practice, identification of other intermediate- and system-level behaviors may reflect even more important aspects of seismic performance for all system types. These include assessment of lateral dynamic instability, identification of controlling mechanisms or modes of failure, and quantification of residual drifts.

Many aspects of the behavior of diaphragms, chords, and collectors, as well as the design of many nonstructural systems and their anchorage, are controlled by floor accelerations, which may be obtained explicitly only from dynamic analysis procedures.

Intermediate- and component-level demand parameters that most clearly relate to overall seismic performance are summarized in Table 8-1. Early phases of design generally focus on intermediate-level response, later phases of design generally focus on component-level response, and final design calculations must address all of the code-prescribed intermediate- and component-level demand parameters for all stories and all actions on all components.

Table 8-1 Key Demand Parameters for Schematic and Preliminary Design

System type	Schematic Design (system-level comparisons)	Preliminary Design (member proportioning)
Steel moment frame	Story drift	Beam moment
	Floor overturning moment	Column (and splice) axial force
Steel concentrically braced frame	Story drift	Brace axial force
	Story shear	Column (and splice) axial force
	Floor overturning moment	
Reinforced concrete moment frame	Story drift	Column shear
	Story shear (especially for nonductile columns in existing buildings)	Beam moment
	Floor overturning moment	Joint shear force
Reinforced concrete shear wall	Story shear	Column moment and axial force
	Floor overturning moment	Wall shear
	Story drift	Wall overturning moment
		Coupling beam rotation

8.4 Suggestions for Further Study

The following areas of further study related to assessment of analytical methods are suggested:

- Additional studies to address the effects of higher mode responses on performance prediction, assessment of collapse, or required strengthening. The studies should aim to find simple and practicable ways to overcome negative impacts of higher modes on performance.
- Additional studies are needed to identify how other higher mode effects, such as diaphragm flexibility, torsional response, and multiple towers on a common podium, should be treated for various structure types.
- Further studies are needed to validate the applicability of the Extended Consecutive Modal Pushover. Possibilities include sequence of application of modes, treatment of torsion, bidirectional loading, processing of results for design (average, maximum, other), fraction of target displacement to assign to each mode, and use of base slab averaging and conditional mean spectra to reduce higher mode response.

Treatment of Variability in Design Decisions

This chapter discusses the treatment of uncertainty and variability in current practice and research, and describes methods for direct determination of target displacement, evaluation of lateral dynamic instability, intensity measure selection, and selection of ground motion subsets. It also recommends an approach for incorporating uncertainty into dynamic analysis and possible extensions of this approach to introduce consideration of variability into nonlinear static analyses.

9.1 Treatment of Variability in Current Practice

Treatment of uncertainty and variability in current code provisions is completely obscured in nonlinear static procedures and extremely simplistic in nonlinear response history analyses (NRHA). This neglect of uncertainty may compromise the ability to design for reliable performance.

Current code provisions for nonlinear dynamic analysis select ground motion records on the basis of their elastic pseudo-acceleration response spectra, in relation to a prescribed design spectrum. Although it is desirable to select motions having magnitudes, fault distances, source mechanisms, and soil conditions that are characteristic of the site, the difficulty of doing so in practice is often addressed by scaling the amplitudes of selected ground motions, possibly in conjunction with alteration of the ground motion frequency content or, alternatively, by using entirely synthetic records in order to more closely match the design spectrum.

Many codes allow peak response values obtained from just three records to be used for design, although mean values can be used if seven or more records are used. Because mean demand parameter values generally exceed medians, some measure of dispersion is inherently introduced in the selection of mean values. However, even when using recorded ground motions, it is recognized that different analysts may apply different scale factors to the records of a given suite to satisfy code requirements and can obtain vastly different results from nonlinear dynamic analysis (FEMA, 2009d). Spectrally matched records may be even less capable of representing realistic dispersions in response quantities. These approaches to selection and scaling of ground motions do not address variability well.

Even where design decisions are based on central tendencies without explicitly including variability, awareness of the sources of variability (and their corresponding magnitudes) can inform qualitative judgments made in selecting and applying design and analysis methods. For instance, recognizing the likely range of displacement response (Section 9.3.1) or the degree to which a small subset of ground motion records reflects the inherent variability of a larger population of representative records (Section 9.3.5) may prevent unrealistic confidence in marginal analysis results or may mitigate concern about inaccuracies in simple analysis methods.

Codes, standards, and guidelines that address nonlinear analysis often target median or mean response quantities but require consideration of variability in some manner. For instance, Section 3.3.3.2.1 of ASCE/SEI 41-06 requires that the base shear force versus lateral displacement relationship from nonlinear static analysis be established for displacements up to 150% of the target displacement. According to the corresponding commentary, the reason for this requirement is to encourage investigation of likely building performance and behavior of the model under extreme load conditions that exceed design values since the target displacement represents a mean displacement value for the design earthquake loading, and there is considerable scatter about the mean (ASCE, 2007).

Uncertainties in performance arise due to random variations as well as epistemic uncertainties. Epistemic uncertainty refers to lack of knowledge (e.g., about material properties or modeling of nonlinear response), while aleatory uncertainty is associated with randomness in nature, which in the context of this chapter is dominated by variability in response from one ground motion record to the next. Probabilistic methods allow one to establish that desired performance is achieved with a desired level of confidence. While the characterization of uncertainty is somewhat crude at present, explicit incorporation of rough measures of uncertainty is preferable to ignorance and neglect.

Analysis methods exist on a continuum from the extremely simple and coarse to the very complex and presumably more accurate. Simple methods may use safety factors to avoid explicit treatment of uncertainty or even rely on ample ductility capacity to “absorb” uncertainties in the analysis, while the more complex methods can address uncertainty explicitly. Numerical results may reflect or ignore contributions of higher modes, whether considered statically or dynamically.

Using nonlinear dynamic analysis, it is theoretically possible to evaluate mean annual frequency of exceedance of a demand parameter value and to determine the capacity required to ensure failure occurs with a desired frequency at a desired confidence level. This can be done numerically or using computationally simpler approaches that make use of assumed functional forms to describe relationships among various parameters. Of course, even with nonlinear dynamic analyses, inaccuracy in the assumed relationships or

in parameter values reduces the accuracy of a computed result and may necessitate modifications to impart some conservatism.

Rigorous assessments using large numbers of dynamic analyses as conducted in current research studies are rarely feasible for design practice. This does not mean, however, that consideration of uncertainty should not be attempted. Simpler approaches for estimating demand parameter statistics have been suggested in recent years. These include the use of spectrally matched records and the selection of record subsets to estimate, for example, median demand parameters.

Use of spectrally matched records is controversial, in part because matching of elastic spectra is not sufficient to ensure representative inelastic response. Appendix D explores the possibility of using subsets of recorded ground motions to obtain index values to characterize demand parameter distributions. The use of record subsets to estimate demand parameter distributions, augmented by other parameters to represent aleatory randomness and epistemic uncertainty, is developed in this chapter. Where such approximate approaches are insufficient (e.g., near collapse, for demand parameters having unusual distributions, or where greater precision is needed), more rigorous approaches can always be used.

One might argue that as long as a low enough probability of failure is achieved, whether a brittle or ductile failure occurs is immaterial. However, the models used in analysis are imperfect representations of reality, and there will always be an advantage to providing ductility capacity that can “absorb” inaccuracies in analysis while also ensuring a low probability of failure of force-protected members. Thus, seismic design could aim to:

- ensure adequate performance of ductile components of the seismic-force-resisting system with moderate confidence at a design level mean annual frequency of exceedance;
- determine required strengths of force-protected components of the seismic-force-resisting system at a higher confidence for the design level mean annual frequency of exceedance; and,
- ensure adequacy of the gravity load-resisting components at a higher confidence for a lower mean annual frequency of exceedance.

This approach recognizes that dispersion varies spatially over the structure and in general differs for every demand parameter and thus should result in better proportioned structures than are obtained with current approaches (for instance, using overstrength factors in current code approaches). In addition, it may also be desirable to check for essentially elastic performance of some components under serviceability earthquakes.

An additional concern with code approaches to nonlinear response history analysis is the use of uniform hazard response spectra. Since uniform hazard spectra represent the

response of linear elastic SDOF systems at a stated mean recurrence interval, they are not representative of modal amplitudes to be expected at multiple periods of vibration in any realization of a design event. The use of so-called epsilon compatible records and conditional mean spectra addresses this issue (Baker and Cornell, 2005).

9.2 Probabilistic Approaches

Results of dynamic analyses are sensitive to variability of ground motions and system properties such as initial stiffness, strength, strain hardening, hysteretic behavior, component capacities, and magnitude of gravity load. The need to assure adequate performance given uncertainties in loading and capacities and the sensitivity of response to modeling brings in probabilistic considerations.

9.2.1 Sources of Variability

One convenient approach to describe sources of variability considers whether variation in results is considered to be random or a consequence of lack of knowledge. The terms aleatory and epistemic are used to distinguish between randomness and lack of knowledge, respectively. For seismic design the main source of randomness (aleatory uncertainty) is the variability in response associated with different ground motion records, each representing a common intensity level. If the intensity measure selected coincides with the demand parameter of interest, aleatory uncertainty is zero. This may occur if S_a or S_d is the intensity measure used in conjunction with a linear SDOF oscillator.

Sources of epistemic uncertainty are various and include uncertainties about material properties, dimensions, member response under prescribed and highly variable (earthquake-induced) loading histories, hysteretic models, and values of model parameters, as well as component capacities associated with performance limit states of interest. Outright errors in engineering and construction are another source of uncertainty and are particularly difficult to consider.

While much work is needed to characterize more precisely these uncertainties, progress has been made in recent studies. Haselton and Deierlein (2007) report dispersions in member strengths to be on the order of 0.1 and dispersions in the capping or post-capping deformation parameters of 0.5 to 0.6 for tests of reinforced concrete members. Intermediate values of dispersion may be appropriate for stiffness and deformation at yield.

Dispersion in computed demand parameters or response quantities depends, in part, on the interaction of multiple modes, the potential to form different inelastic mechanisms in the structure, the location of interest within the structure, and the response quantity of interest. This has bearing on the nature of the distribution of the response quantity. For example, distributions of some parameters, such as story drift, may be approximated well

by a log normal distribution while other quantities, such as the forces in yielding members, may saturate at intensities sufficient to develop yielding in a significant portion of the responses. Still other quantities, such as plastic hinge rotations, may vary significantly if different inelastic mechanisms are activated and, furthermore, may have peculiar distributions owing to the presence of many near-zero values where peak responses are essentially elastic. In such cases it may be fruitful to examine the distributions of total rotation relative to the chord and infer plastic rotation at a later step.

9.2.2 *Quantification of Uncertainty*

The quantification of epistemic uncertainty is a core issue in performance-based seismic design. It involves a large number of known or unknown factors that may affect response, thus making accurate estimation of response quantities a difficult undertaking due to the large number of alternate models and parameter assumptions that need to be evaluated. Further complicating the problem is that epistemic uncertainty may affect not only the variability in parameter values but also the central value (mean or median). In essence, this means that the median-parameter model may not yield the median response.

Perhaps the most important simplification of the theoretical structure of the problem has been offered in Cornell et al. (2002) by adopting the first-order assumption. Therein, it is assumed that epistemic uncertainty does not influence the median response, but only inflates its variability. Under the typical lognormal assumption of demand parameter distribution, the total dispersion (standard deviation of the log of the response) is estimated as the square-root-sum-of-the-squares (SRSS) combination of the aleatory and epistemic dispersions.

This greatly facilitates the estimation of epistemic uncertainty as it reduces the problem to the quantification of the variability in the median estimate due to the various uncertainty sources, essentially decoupling aleatory randomness and epistemic uncertainty calculations into two separate problems that can be recombined at will using the SRSS rule. This idea has been exploited in depth in the FEMA 350 (FEMA, 2000a) and FEMA 351 (FEMA, 2000b) reports, where the dispersion of various sources of uncertainty has been quantified for use in performance estimation (although simple placeholder values are used in these documents rather than structure-specific values).

This has spurred research efforts to properly compute estimates of epistemic uncertainty effects, at least for model parameters of various types of structures. For example, Ibarra et al. (2005) proposes a method to propagate the uncertainty from model parameters to structural behavior using first-order-second-moment principles verified through Monte Carlo simulations to evaluate collapse capacity uncertainty. Lee and Mosalam (2005) have also used first-order-second-moment principles to determine the response uncertainty of a reinforced concrete shear wall structure to several modeling parameters. Recently, Monte Carlo simulation has been introduced to incorporate parameter uncertainty into performance calculations using nonlinear dynamic analyses. Liel et al.

(2009) and Dolsek (2009) have applied this method to reinforced concrete frame structures, and Vamvatsikos and Fragiadakis (2010) have used it for steel moment-resisting frames. In all cases, the impact of epistemic uncertainties on the response quantities investigated was found to be important, yet lower than the effect of aleatory variability, which tends to dominate in the determination of the total variability. In addition, such studies have confirmed the existence of a consistent small to moderate underestimation bias in the median response quantities, suggesting that the first-order assumption may be slightly unconservative.

More recently, Fragiadakis and Vamvatsikos (2010) have offered a rapid approximation methodology that can estimate the effect of model parameter uncertainty. It is based on the capacity boundary (static pushover) curve using an equivalent single-degree-of-freedom system as a basis for extrapolating the results to nonlinear multiple-degree-of-freedom dynamic response via the SPO2IDA tool (Vamvatsikos and Cornell, 2005b), thus offering a practical alternative to approaches that rely on nonlinear response history analysis.

Despite these efforts, the issue of epistemic uncertainty will remain open for the foreseeable future, especially regarding modeling assumptions and their impact on response estimation. Detailed work is needed to provide proper uncertainty quantification. The only short-term solution may be to require the use of standardized, general consensus values.

9.3 Determination of Variability

In order to properly treat the influence of variability on the performance of structures, it is desirable to accurately quantify variability due to all sources of uncertainty on each demand parameter. The following sections describe approaches for incorporating uncertainty into various steps of static and dynamic analyses.

9.3.1 *Target Displacement*

The displacement coefficient and equivalent linearization approaches traditionally have been used to obtain point estimates of the central value (mean or median) of target displacement for use in nonlinear static analysis. Once a capacity curve has been determined by nonlinear static analysis, two other approaches can be used to better estimate the central value and to estimate the statistical distribution of target displacement: (1) nonlinear response history analysis of the Equivalent SDOF system using a suite of suitable ground motion records, and (2) application of the open source software tool SPO2IDA.

If the target displacement estimate is based on elastic spectral ordinates, significant variability is observed in the peak displacements obtained by nonlinear dynamic analysis. Both of these approaches (described in Appendix E) can be used to determine not only an

estimate of the central value but also the dispersion of the roof displacement obtained during dynamic response that is associated with record-to-record variability. In some sense, this dispersion is a product of the reliance on elastic spectral descriptions of hazard. Yet another alternative is to determine the S_{di} hazard curve and estimate the target displacement at a desired probability of exceedance, P_o , as Γ_I times the mean S_{di} value associated with $\lambda_{S_{di}}(P_o)$.

Computation of the response of Equivalent SDOF systems to ground motion records can make use of atypical hysteretic relationships and site-specific ground motions, if relevant, but requires some time to do. In contrast, the SPO2IDA tool makes use of sophisticated interpolation schemes to mine a database of previously computed results, and hence, at this time, is best suited for systems with moderate pinching, where ordinary ground motion records are suitable (i.e., records without any soft-soil or near-source directivity effects). The response statistics obtained by either of these approaches, at multiple levels of the intensity measure, are needed to generate an S_{di} hazard curve, for a given S_a hazard curve.

These approaches to displacement estimation are illustrated for the 4-story reinforced concrete moment frame structure that was studied in this project and summarized in Chapter 5. Figure 9-1 shows a multi-linear capacity boundary fitted to the results of a first mode pushover analysis of the moment frame.

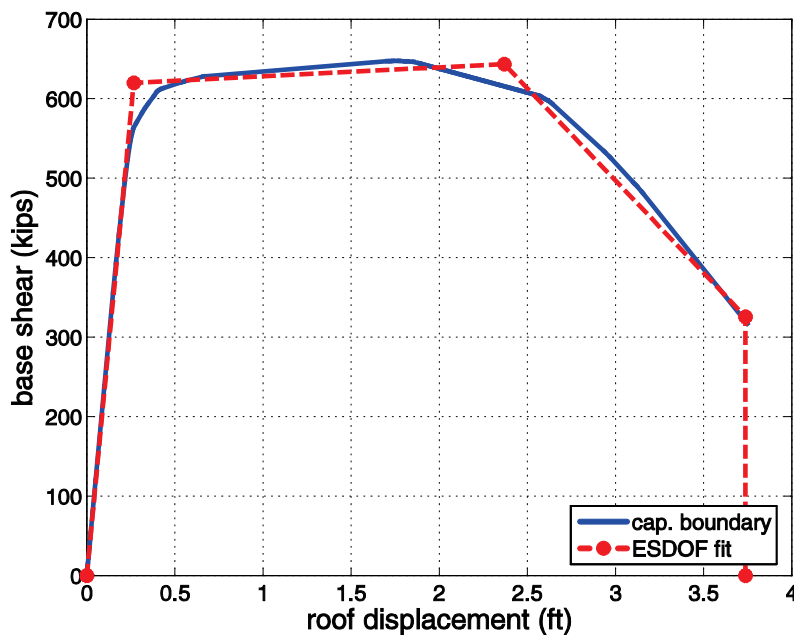


Figure 9-1 Capacity boundary (pushover curve) of the equivalent single-degree-of-freedom system fitted to the results of a first-mode pushover of the 4-story reinforced concrete moment frame building model.

Figure 9-2 shows the results of nonlinear dynamic analyses of the Equivalent SDOF oscillator subjected to the full set of 44 ground motion records, repeatedly scaled to

different intensity levels (effectively developing full Incremental Dynamic Analysis (IDA) curves for the Equivalent SDOF system). In this case, analyses were done using the IIDAP program (Lignos, 2009). The left panel of this figure shows results for each ground motion, while the right panel shows 16th, 50th, and 84th percentile curves (of S_a at a given S_{di} value).

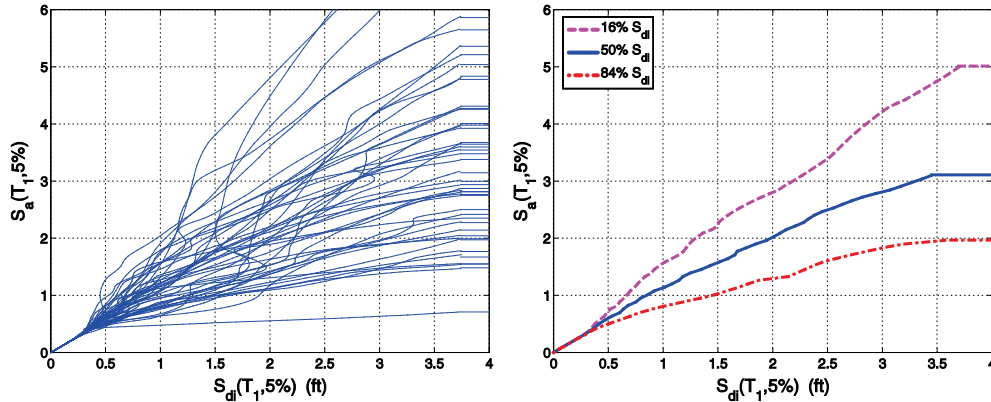


Figure 9-2 Complete set of 44 IDA curves and the summary 16th, 50th, 84th percentile IDA curves in S_a - S_{di} coordinates as produced by nonlinear response history analysis for the equivalent single-degree-of-freedom fitted to a 4-story reinforced concrete moment frame building model.

Figure 9-3 shows estimates of the 16th, 50th, and 84th percentile curves that were determined using the SPO2IDA tool (Vamvatsikos and Cornell, 2006). The SPO2IDA results are within 10-15% of the IIDAP results.

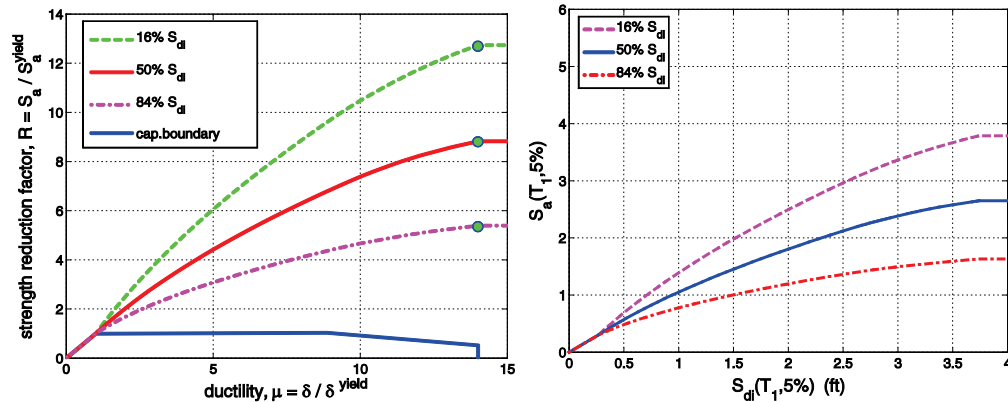


Figure 9-3 Percentile IDA curves in R - μ and S_a - S_{di} coordinates as produced by SPO2IDA for the equivalent single-degree-of-freedom fitted to a 4-story reinforced concrete moment frame building model.

Figure 9-2 and Figure 9-3 plot results for the peak displacement of the Equivalent SDOF oscillator, also known as S_{di} . The two approaches allow target displacements to be estimated (at the roof level) at any desired quantile as the product of I_T and the corresponding value of S_{di} . A comparison of results obtained at different scale factors is provided in Figure 9-4. This figure presents mean plus and minus one standard deviation

results for the Equivalent SDOF estimates of roof displacement; 16th, 50th, and 84th percentile estimates of roof displacement determined using SPO2IDA; mean plus and minus one standard deviation results obtained from nonlinear response history analysis of the MDOF system to the 44 ground motion records; and the point estimate obtained using the displacement coefficient approach of ASCE/SEI 41-06 (ASCE, 2007). The predicted central values are similar for this structure.

The explicit estimation of target displacement using Equivalent SDOF or SPO2IDA offers two advantages: (1) the explicit methods can address a broader range of structural response than do the coefficients tabulated in ASCE/SEI 41-06 and (2) the explicit methods provide information regarding the uncertainty of the estimates.

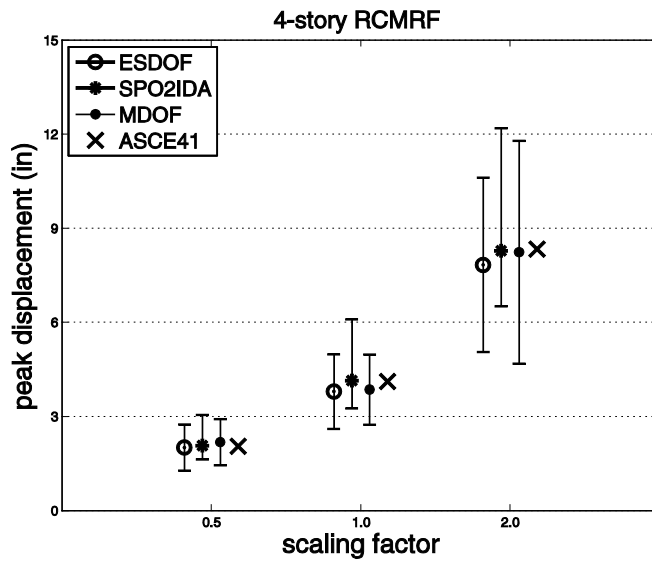


Figure 9-4 Comparison of Equivalent SDOF and SPO2IDA estimates of roof displacement with results from 44 nonlinear response history analyses and ASCE/SEI 41-06 coefficient method estimates for a 4-story reinforced concrete moment frame.

9.3.2 Lateral Dynamic Instability

Lateral dynamic instability occurs when lateral resistance degrades due to damage caused by response to ground motion. Nonlinear response history analysis of the MDOF system is considered the most accurate approach for assessing the likelihood of lateral dynamic instability. However, several simpler approaches based on pushover analysis may be used for this purpose; it seems that evaluation of lateral dynamic instability by pushover analysis may be more robust than evaluation of nonlinear response more generally, possibly due to the response at collapse being dominated more by a single mode than at lower levels of nonlinear response. Each of these pushover-based approaches makes use of the capacity boundary determined from nonlinear static analysis.

One approach uses the IDA curves obtained from either explicit nonlinear response history analysis of the Equivalent SDOF oscillator or SPO2IDA relationships (the right panels of Figures 9-2 and 9-3). Collapse due to lateral dynamic instability is considered to occur when the IDA curves have zero slope, which indicates the quantile value of pseudo-spectral acceleration at which lateral displacements increase without limit. This approach provides an estimate of both the median and the distribution of collapse capacity.

A second approach to assess the median collapse capacity does so by means of a maximum limit on the parameter R_{di} , developed through a large number of SDOF simulations in FEMA P-440A (FEMA, 2009), but without consideration of its distribution (variability).

9.3.3 Intensity Measure S_a

Scaling ground motions to $S_a(T_1, 5\%)$ is most appropriate where site-specific hazard data, including ground motion records, are available. Ground motions are individually scaled in amplitude to match the value of pseudo-spectral acceleration corresponding to the first mode period of the structure, T_1 . A set of at least 30 such records is recommended where median response quantities or their dispersions are sought. Subsets of these records will be used for the MDOF nonlinear response history analyses. Ground motion records for this approach ideally should be epsilon-consistent (Baker and Cornell, 2005) to obtain unbiased response quantity estimates. Nevertheless, ignoring this issue generally will lead to conservative estimates. A simple alternative is to select ground motions for which the scale factor applied to each natural (corrected) record is between 0.4 and 2.5. The target pseudo-spectral acceleration level $S_a(T_1)$, symbolized as S_a^{des} , is determined from the S_a hazard curve at the desired mean annual frequency of exceedance.

9.3.4 Intensity Measure S_{di}

As stated by Tothong and Cornell (2008), S_{di} offers advantages over $S_a(T_1, 5\%)$ in that S_{di} removes the so-called peak-valley effects associated with period elongation during nonlinear response, and can reduce the potential bias in scaling the amplitude of ground motions, thus simplifying record selection by avoiding strong emphasis on other ground-motion record properties such as ε (Baker and Cornell, 2005), M_w , and distance, and thus is less restrictive than $S_a(T_1, 5\%)$ scaling as far as the selection of acceptable ground motion records.

Another advantage over $S_a(T_1, 5\%)$ scaling for approximate performance assessment is that the hazard curve, when expressed in terms of S_{di} rather than $S_a(T_1, 5\%)$, may be more closely approximated by a line (in log-log space) in the region of interest. This region, according to the suggestions in Jalayer and Cornell (2009), Dolsek and Fajfar (2008), and Vamvatsikos and Dolsek (2010), consists of the interval $[0.25, 1.5] S_i^{\text{des}}$; the region of interest extends further into the lower intensities because these have higher probabilities

of exceedance. This results in S_{di} -based performance assessment being more accurate, avoiding a conservative bias caused by the introduction of a linear approximation in S_a -based hazard assessment. Note that for taller buildings (where higher modes are more significant), an S_{di} -based approach should be augmented by consideration of higher mode contributions.

The disadvantage of the use of S_{di} as an intensity measure is the need to compute the S_{di} hazard unless a probabilistic seismic hazard analysis is done using attenuation relationships for S_{di} (e.g., Tothong and Cornell, 2006). Computation of the S_{di} hazard is illustrated below, and may be automated in future versions of software tools such as IIIDAP and SPO2IDA.

The target S_{di} value is determined from the S_{di} hazard curve at the desired mean annual frequency of exceedance, termed S_{di}^{des} . Where the S_{di} hazard curves are determined by explicit nonlinear response history analyses of the Equivalent SDOF oscillator, the IDA curves obtained for the individual ground motion records also identify the value of spectral acceleration that the records should be scaled to so that each individually scaled record produces the target S_{di} .

An estimate of the S_{di} hazard curve, $\lambda_{S_{di}}$, can be obtained by performing an integration of the S_a versus S_{di} (as intensity versus response) curves with the S_a hazard as follows:

$$\lambda_{S_{di}}(x) = \int_0^{+\infty} F(S_{ac} | S_{di} = x) \left| \frac{d\lambda(S_a)}{dS_a} \right| dS_a \quad (9-1)$$

where the first term represents the cumulative distribution function of the S_a capacity values for a given S_{di} value and the term in absolute value is the local slope (derivative) of the S_a hazard curve for a given value of S_a . The integration is performed for multiple values of S_{di} (typically about 10), as each $(S_{di}, \lambda_{S_{di}})$ pair resulting from one evaluation of the above integral is a single point of the S_{di} hazard curve. This task is relatively easy to automate and is illustrated in Figure 9-5 for the 4-story reinforced concrete moment frame study for a Life Safety performance level corresponding to a 10% frequency of exceedance in 50 years for a site with mean hazard.

First, S_{di} values that are of interest for the assessment are determined within the range of $[0.25, 1.5] \cdot S_{di}^{\text{des}}$. Since S_{di}^{des} exceeds the target displacement of the Equivalent SDOF oscillator (from a typical nonlinear static procedure application) at this mean recurrence interval, the upper bound of the interval of S_{di} values to use can be taken as three times the estimated target displacement S_{di}^{Po} corresponding to the design spectral acceleration S_a^{des} . Thus, numerical evaluation of the integral of Equation 9-1 can be done at 10 or more equally spaced values of S_{di} from 0 to $3S_{di}^{\text{Po}}$; each of these values of S_{di} is referred to in the following by s_{di} .

For each s_{di} value, the statistics of the corresponding S_a values need to be evaluated. The simplest way is to consider the 16th, 50th, and 84th percentile curves of Figure 9-2 or Figure 9-3 as defining a lognormal distribution of S_a at each of the S_{di} values. The log-mean, $m(s_{di})$ and log standard deviation $s(s_{di})$ are given by

$$\begin{aligned}
 m(s_{di}) &= \ln[S_{a,50\%}(s_{di})] \\
 s(s_{di}) &= 0.5 \{ \ln[S_{a,84\%}(s_{di})] - \ln[S_{a,16\%}(s_{di})] \}
 \end{aligned}
 \tag{9-2}$$

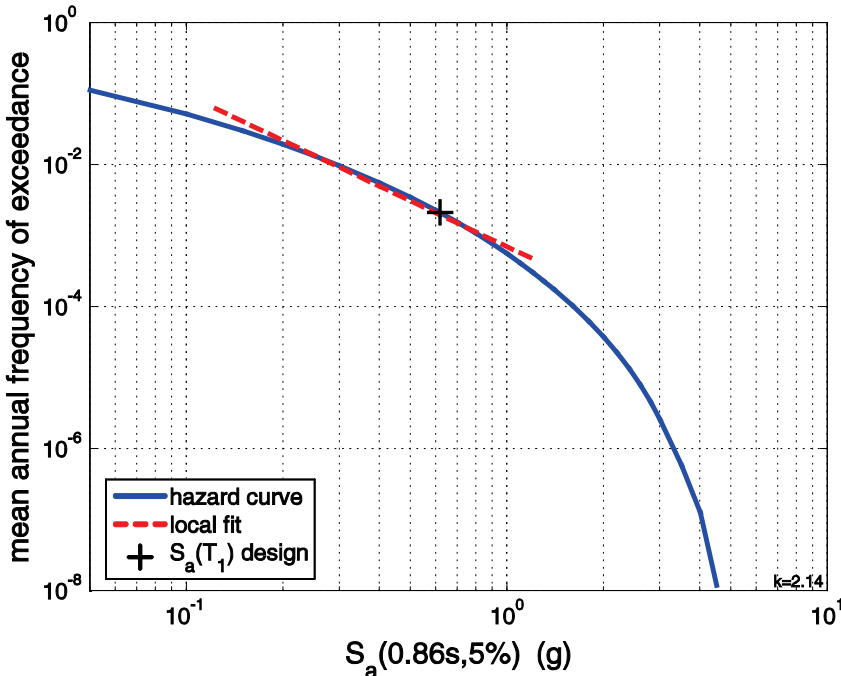


Figure 9-5 The mean S_a hazard curve used in evaluation of the 4-story reinforced moment frame structure.

This process is illustrated using the data of the right panel of Figure 9-3 (for SPO2IDA) in Table 9-1, and the relevant quantile data are plotted in Figure 9-6. Note that wherever the 16th, 50th, and 84th percentiles coincide (in this case, for s_{di} values less than the yield displacement of 0.267 ft), the oscillator remains elastic, the dispersion is zero, and the S_{di} hazard is identical to the S_a hazard. Thus, the corresponding mean annual frequency of exceedance values obtained for S_a from Figure 9-5 are used for the elastic s_{di} values.

It should be noted that a theoretically more accurate approach would use the actual response data, such as the right panel of Figure 9-2, obtained through explicit dynamic analysis of the Equivalent SDOF system rather than fitting an assumed distribution to the data to determine the empirical cumulative distribution function of the resulting 44 intersection points. However, this is much more complex to implement, while results obtained were nearly identical to those obtained using the above approach.

Table 9-1 Statistics of S_a Values Associated with Ten S_{di} Values Determined Using SPO2IDA

Slice	$x = S_{di}$	16% S_a	50% S_a	84% S_a	$s(x)$
1	0.140	0.224	0.224	0.224	0.0%
2	0.280	0.461	0.451	0.444	2.0%
3	0.421	0.804	0.688	0.602	14.5%
4	0.561	1.121	0.905	0.738	20.9%
5	0.701	1.417	1.106	0.860	25.0%
6	0.841	1.695	1.294	0.972	27.8%
7	0.981	1.959	1.472	1.075	30.0%
8	1.122	2.210	1.642	1.171	31.8%
9	1.262	2.450	1.803	1.261	33.2%
10	1.402	2.681	1.959	1.347	34.4%

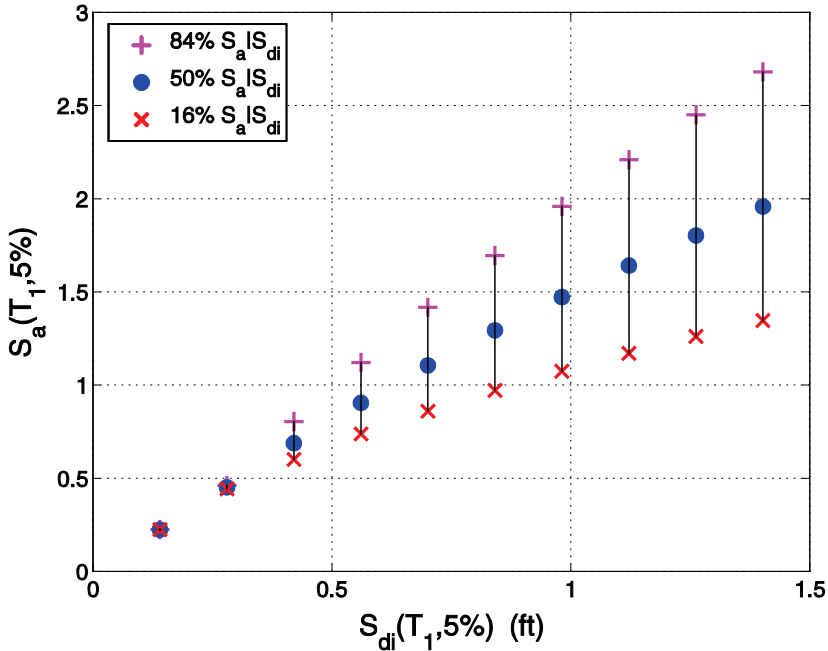


Figure 9-6 Vertical slices and the 16th, 50th, 84th percentile values of required S_a to achieve a given S_{di} .

Next, the elastic portion of the S_{di} hazard curve is estimated. Conceptually, this corresponds to graphically locating the appropriate mean annual frequency of exceedance values for the median S_a of each slice from Table 9-1. For greater accuracy, linear interpolation between the actual discrete values of the S_a hazard curve in log-log (or ln-ln) coordinates is advised. It may be implemented by first selecting the closest two

points that bracket the S_a value, designated as point 1 and 2, where $S_{a2} > S_{a1}$, and estimating the slope in logarithmic coordinates:

$$a_{\ln} = \frac{\ln(\lambda_{S_{a2}}) - \ln(\lambda_{S_{a1}})}{\ln S_{a2} - \ln S_{a1}} \quad (9-3)$$

Then, the corresponding value of λ_{S_a} for any value of S_a between S_{a1} and S_{a2} can be approximated as:

$$\lambda_{S_a} \cong \exp[\ln \lambda_{S_{a1}} + a_{\ln} (\ln S_a - \ln S_{a1})] \quad (9-4)$$

The resulting λ_{S_a} values are used directly as the mean annual frequency of exceedance values corresponding to slices that show zero or near-zero dispersion.

Lastly, S_{di} hazard values ($\lambda_{S_{di}}$) are estimated by numerically performing the integration in Equation 9-1. For the integration itself, the entire S_a range where the hazard curve is non-zero is used, thus restricting the integration within [0.05, 4.5]. For efficiency reasons, 25 integration points are selected for each of the 10 slices, uniformly spread in log-space. The probability of S_a capacity exceedance, described by the cumulative distribution function $F(S_a|S_{di})$ is determined by a normal distribution function for all 25 S_a integration points along each slice using the corresponding $s(s_{di})$ and $m(s_{di})$ parameters from Table 9-1. This can be easily automated (e.g., using the NORMDIST function in Excel). Then, linear interpolation in logarithmic coordinates is performed using Equations 9-3 and 9-4, for each S_a value. Following the properties of differentiation, the absolute value of the slope in linear coordinates is estimated from its value in log-space as:

$$|a| \cong -\frac{\lambda_{S_a}}{S_a} \cdot a_{\ln} \quad (9-5)$$

For the evaluation of the integrant $|a|F(S_a|S_{di})$, the trapezoidal rule can be followed by finding the average of each two adjacent integrant values, multiplying by the difference in the S_a values, and summing up the 24 individual contributions to obtain the $\lambda_{S_{di}}$ for a given value of s_{di} .

This process must be repeated for each of the slices that have non-zero S_a dispersion (slices 2 to 10 in this case). The final results appear in Table 9-2, which also includes the elastic $\lambda_{S_{di}}$ results. The inelastic S_{di} hazard estimation is not as accurate when the slice S_a dispersion is less than 10%; therefore, the maximum of the elastic and inelastic estimates of hazard for each slice are used as the final value for $\lambda_{S_{di}}$. As shown in Table 9-2, this updates only the value for slice 2, where the S_a dispersion was only 2%.

The final resulting values are very accurate, differing less than 1% from those obtained by the more rigorous approach using complex spline fitting with hundreds of integration points. Using only 10 or 15 integration points will produce increasing errors, on the order of 40% and 3%, respectively. Given the associated computational load, the use of no fewer than 20 integration points is recommended.

Table 9-2 Final S_{di} Hazard Results by Combining Inelastic and Elastic Hazard Estimates

Slice	$x = S_{di}$	λS_d	λS_{di}	$\max(\lambda S_d, \lambda S_{di})$
1	0.14	1.58E-02	0.000E+00	1.58E-02
2	0.28	4.26E-03	3.900E-03	4.26E-03
3	0.42	1.59E-03	1.686E-03	1.686E-03
4	0.56	7.51E-04	8.664E-04	8.664E-04
5	0.70	4.01E-04	5.164E-04	5.164E-04
6	0.84	2.33E-04	3.372E-04	3.372E-04
7	0.98	1.43E-04	2.345E-04	2.345E-04
8	1.12	9.17E-05	1.707E-04	1.707E-04
9	1.26	6.09E-05	1.287E-04	1.287E-04
10	1.40	4.11E-05	9.979E-05	9.979E-05

Figure 9-7 plots the hazard curve (as $\lambda_{S_{di}}$ versus S_{di} in a log-log format) calculated by this procedure as well as a straight line fit in the region of interest. For completeness, additional S_{di} slices were considered in order to plot the entire hazard curve. Global lateral dynamic instability occurs where S_{di} increases without limit for a constant mean annual frequency of exceedance value.

The design value, S_{di}^{des} , can now be determined, consisting of the S_{di} value that has a mean annual frequency of exceedance of 1/475 years (or 10% in 50 years). Note that in the inelastic range of response, S_{di}^{des} exceeds the median S_{di} value that corresponds to the S_a value with 1/475 years. S_{di}^{des} can be estimated as 0.57 ft using a linear interpolation in log-log coordinates. It can now be observed that the use of only the first seven slices would have been adequate to cover the range $[0.25, 1.50] \cdot S_{di}^{des}$. In general, additional slices can always be introduced wherever needed to improve the precision of the calculated hazard curve, should such precision be needed.

The straight line fit (in log-log space) over the interval $[0.25, 1.50] \cdot S_{di}^{des}$ establishes the local slope of the hazard curve, k , equal to 1.94 (as an absolute value). Due to the flatter shape of the S_{di} hazard curve, the resulting fit is superior to the one shown in Figure 9-5 for the S_a hazard. Thus, it avoids the consistent overestimation of the mean annual frequency of exceedance value for a given intensity measure value that is evident in Figure 9-5 for S_a , and, therefore, leads to a less biased assessment of the mean annual frequency of exceedance of demand (i.e., one that exhibits more controlled conservatism).

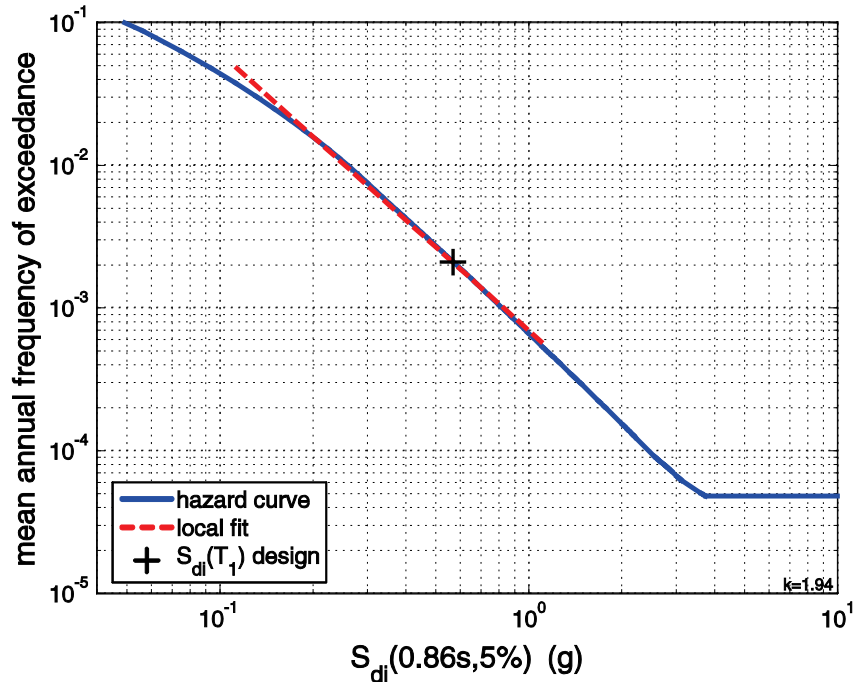


Figure 9-7 The S_{di} hazard curve, as estimated via SPO2IDA, and the power-law approximation in the area of the design point. Only minor differences appear if nonlinear response history results are used instead.

9.3.5 Selection of Ground Motion Subsets

Nonlinear response history analyses of the MDOF model of the structure can be implemented at a given hazard level either by using a moderately large pool (greater than 30) ground motion records or, more economically, by selecting subset(s) of ground motion records to estimate index values of the demand parameter distributions. The use of record subsets here follows the initial ideas presented by Azarbakht and Dolsek (2007, 2010). The ground motion records used in the nonlinear response history analyses are individually scaled to the target S_a or target S_{di} level.

Median values of demand parameters are estimated using a subset of records whose spectra individually and collectively best approximate the median (50th percentile) spectrum. These records comprise record subset A. Where estimates of dispersion are sought, records are selected whose spectra individually and collectively best approximate a desired higher level of response spectrum (taken as the 84th percentile in this discussion). These records comprise record subset B.

Subsets are selected using the elastic response spectra determined for the entire suite of ground motion records, scaled either to the target S_a value or to the target S_{di} value. The scaled elastic response spectra determined for the entire pool are used to generate 50th percentile (median) and 84th percentile spectra of S_a as a function of T . Subsets of ground motion records are selected based on how well the corresponding elastic spectra match the median or 84th percentile spectral amplitudes over a relevant period range.

For records scaled to the target S_a , spectral matching should be done over a period range defined by the two intervals $0.8T_i$ to $1.2T_2$ and from $0.8T_1$ to $1.5T_1$, where $i = \text{ceil}(\sqrt{N_{st}})$ with ceil being the ceiling function that rounds up to the nearest higher integer and N_{st} is the number of stories in the building. For records scaled to the target S_{di} , a period range approximately from $0.8T_i$ to $1.2T_2$ was identified. In the case of S_{di} scaling, the period range need not extend up to and past the first mode period as the use of S_{di} is expected to directly take into account the lengthening of the first-mode period of the structure that occurs as inelastic response develops.

Estimates of median values of each demand parameter, designated here as EDP, are given as the median of the demand parameter values obtained using record subset A as follows:

$$EDP_{50} \cong [EDP_i]_{A,50\%} \quad (9-6)$$

Dispersion is estimated in a two-step process. First an estimate of the 84th percentile demand parameter is obtained using either the median of the subset B records or a combined value of responses from subset A or B:

$$EDP_{84} \cong \begin{cases} \max([EDP_i]_{B,50\%}, [EDP_i]_{\max(A,B),50\%}), & \text{if } N_B < 0.2N_T \\ \max([EDP_i]_{B,50\%}, [EDP_i]_{\max(A,B),50\%}, [EDP_i]_{A+B,84\%}), & \text{if } N_B \geq 0.2N_T \end{cases} \quad (9-7)$$

Next, the dispersion in the demand parameter is estimated as:

$$\beta_{EDP|IM} \cong \ln(EDP_{84}) - \ln(EDP_{50}) \quad (9-8)$$

The use of 7 to 11 records in each subset appears to be adequate for many cases. There is no restriction to the maximum size of subset A or B, apart from a logical limitation of employing about one third of the total number of records. Thus, for a minimum pool size of 30 records, the maximum suggested corresponding subset size is 10 for each subset. Due to the approximate nature of the results obtained with record subsets, wherever greater accuracy is required, larger subsets may be selected from an even larger pool size, or the entire pool of records may be used.

If the entire set of ground motion records is used to determine the MDOF response, then, allowing for modeling limitations, the evaluation procedure would be expected to introduce negligible additional uncertainty (error). On the other hand, when subsets are used, there is an additional (non-negligible) error that must be considered depending on the number of records employed in each subset and the parameter type. To represent this error, it is suggested that the dispersion estimated via this methodology include additional epistemic uncertainty dispersion given by

$$\beta_{u,sub} = \begin{cases} 1.2 / \min(N_A, N_B), & \text{for global EDPs} \\ 1.5 / \min(N_A, N_B), & \text{for story - level EDPs} \\ 2.1 / \min(N_A, N_B), & \text{for component - level EDPs} \end{cases} \quad (9-9)$$

Examples of global EDPs are maxima over the entire building such as the maximum story drift or the peak roof drift over all stories, examples of story-level EDPs are story drifts and floor accelerations, and examples of component-level EDPs are member forces and section rotations.

9.3.6 Demand and Capacity

Design decisions ultimately require ascertaining that a design satisfies performance expectations, and this often requires comparison of seismically induced demands with the capacities associated with the materials and components that form the structure.

Although structural engineers are accustomed to treating uncertainty in demands and capacities in ultimate strength design for non-seismic loads, seismic design practice generally has neglected uncertainty, particularly on the demand side.

The determination of the mean annual frequency of exceedance of a demand parameter, or the required strength to ensure the capacity meets or exceeds the demand within limits implied by the mean annual frequency of exceedance at a given confidence level, necessitates the estimation of response for at least one additional seismic intensity level. When multiple intensity levels are being evaluated simultaneously (e.g., at the 50%, 10% and 2% in 50 year hazard levels), then the results at the next highest (if available) intensity level (lower probability, P_o) can be used for this purpose.

Alternatively, if the performance assessment is being made at only a single performance level (i.e., only one value of probability P_o), median response at a higher intensity level must also be determined, using a third subset of records comprising subset A scaled by a factor of 1.20 or larger. Specifically, let EDP'_{50} denote the median demand parameter at the higher IM' level, then the log-slope, b , of the median demand parameter curve is estimated as follows:

$$b_{EDP} = \frac{\ln(EDP'_{50}) - \ln(EDP_{50})}{\ln(IM' / IM)} \quad (9-10)$$

Estimates of the so-called factored demand and factored capacity can be made using the previous values at a mean annual frequency of exceedance of P_o , following Cornell et al., as

$$FD(MAF = P_o) = EDP_{50} \exp\left(\frac{1}{2} \frac{k}{b_{EDP}} \beta_{EDP|IM}^2\right) \quad (9-11)$$

$$FC = EDP_{C,50} \exp\left(-\frac{1}{2} \frac{k}{b_{EDP}} \beta_{C,aleatory}^2\right) \quad (9-12)$$

where FD and FC are the factored demand and factored capacity, respectively, stemming from the median demand, EDP_{50} , and the median capacity, $EDP_{C,50}$, modified by appropriate factors in a manner similar to load and resistance factors in strength design to

account for the influence of aleatory randomness. These influences appear in the form of the dispersions. The parameter k is the slope of the hazard curve when plotted in log-log coordinates and depends on local seismicity and site characteristics. The slope may be derived by taking a straight-line approximation to the hazard curve in log-log coordinates, as shown in Figure 9-5. The approximation should closely fit the hazard curve in the region $[0.25S_a^{\text{des}}, 1.5S_a^{\text{des}}]$.

Finally, to ensure the factored capacity, FC , exceeds the factored demand, FD , with the designated mean annual frequency of exceedance at a confidence level of α , the following must be satisfied:

$$FC > FD \cdot \exp(K_x \beta_{u,\text{total}}) \quad (9-13)$$

where K_x is the standard normal variate corresponding to the desired α (e.g., $K_x = 1.28$ for $\alpha = 90\%$, and $K_x = 0$ for $\alpha = 50\%$), and $\beta_{u,\text{total}}$ is the total uncertainty in demand and capacity. One component of $\beta_{u,\text{total}}$ is $\beta_{u,\text{sub}}$, which represents epistemic uncertainty associated with using record subsets to estimate demand, as given by Equation 9-9. Other components of $\beta_{u,\text{total}}$ address uncertainty in modeling parameters, $\beta_{EDP/IM}$ as determined according to Equation 9-8, and $\beta_{C,\text{aleatory}}$ and $\beta_{C,\text{epistemic}}$, the aleatory and epistemic uncertainty on the EDP capacity, respectively. The final value of $\beta_{u,\text{total}}$ is estimated by taking the SRSS combination of the individual components, thus generally being dominated by the influence of $\beta_{EDP/IM}$, $\beta_{C,\text{aleatory}}$, and $\beta_{C,\text{epistemic}}$.

The expressions for FD and FC can be used to test that the performance requirements are met for any demand parameter at any preset level of confidence, thus providing customizable checks that can be tailored to the desired level of safety. For example, recognizing the higher impact a brittle element failure may have for structural safety, Equation 9-13 can be used to determine the nominal strength required of brittle elements for which the capacity would be exceeded at a specified mean annual frequency of exceedance; typically this would be done for a higher level of confidence and possibly for a lower level P_o than used for evaluation of deformation demands of ductile elements.

9.4 Approximate Assessment of Variability in Nonlinear Response History Analysis

Given the presence of many sources of variability that can strongly influence structural response and the significant burden posed by rigorous probabilistic evaluations, approximate procedures to account for variability have value. This section presents an approximate approach based on the practical framework introduced in FEMA 350 and FEMA 351. This approach is simple and transparent and is intended to be calibrated to be conservative. More rigorous approaches, aiming at accurate and unbiased estimates of response, can always be employed where needed.

The simplifications employed affect the accuracy of the estimates and therefore limit the applicability of the approach. Limitations in the accuracy of the framework set forth in FEMA 350 and FEMA 351 have been described by Vamvatsikos and Cornell (2004) and Aslani and Miranda (2003), who note this approach is most accurate when away from the onset of global instability and for response quantities that are lognormally distributed and do not saturate. Saturation occurs where, for the given mean annual frequency of exceedance, the demand parameter reaches a maximum possible value around which results are tightly clustered.

Bearing such limitations in mind, the approximate approach described herein provides a relatively simple means to account for important sources of variability in nonlinear response history analysis, providing powerful analysis options to a knowledgeable user in a format that can be upgraded incrementally.

9.4.1 Define Seismic Hazard

As a first step, the seismic shaking hazard at the site of the structure is defined, allowing for the analysis results to be expressed probabilistically. Hazard may be determined from a site-specific probabilistic seismic hazard analysis. For sites within the United States, $S_a(T_1, 5\%)$ hazard data are available on the USGS website (www.usgs.gov).

Figure 9-5 presents an example of such data for the 4-story reinforced concrete frame structure at $T_1 = 0.86$ seconds, which corresponds to the first mode period. Also shown in the figure is a straight line fit to the log-log hazard curve over the interval of interest for performance estimates at the 10% in 50 year hazard level.

Mean hazard information is preferred over median data because the higher mean hazard curve also incorporates the important effect of epistemic uncertainty in the seismic hazard, as described in Cornell et al. (2002). Use of the mean hazard curve effectively allows the considerable uncertainty in seismic hazard to be addressed at this stage in a transparent way that simplifies the assessment process and avoids the need to explicitly account for seismic hazard uncertainty in subsequent steps.

9.4.2 Develop Multiple-Degree-of-Freedom Model

Next, a model representing the mechanical behavior of the structure is created. The model must be capable of representing important potential inelastic mechanisms and degradation of strength and stiffness as well as P-Delta effects.

9.4.3 Assess Potential for Lateral Dynamic Instability

Nonlinear static analyses must be performed to assess the potential for lateral dynamic instability. The methods set forth in Section 9.3 should be considered for investigating lateral dynamic instability.

9.4.4 *Select Intensity Measure*

The intensity measure for scaling ground motions for use in nonlinear response history analyses of the MDOF model must be selected. Two common options are $S_a(T_1, 5\%)$, the pseudo-spectral acceleration at first mode period for viscous damping equal to 5% of critical damping, and S_{di} , the peak inelastic displacement of the Equivalent SDOF system.

The two methods are expected to diverge at higher scaling factors, where the target S_a approach is expected to provide more conservative results while the S_{di} approach may be more accurate and allow for more economical designs. The better relative performance of the S_{di} approach is due to: (1) greater sufficiency of S_{di} as an intensity measure, making it able to better represent the implied seismic hazard; and (2) the ability to better approximate the relevant hazard curve with a power law function (Figure 9-7 compared with Figure 9-5). The S_{di} approach also has the advantage of requiring a nonlinear static (pushover) analysis, which provides the analyst with more insight into the behavior of the structure.

9.4.5 *Perform Nonlinear Response History Analysis*

Demand parameters are calculated by subjecting the MDOF model to nonlinear response history analysis. Subsets of motions may be used to estimate median and 84th percentile response quantities.

9.4.6 *Evaluate Demand Parameters*

Results from nonlinear response history analyses using record subsets A and B (where needed) are used to characterize demand parameter statistics, and ultimately, to determine factored demands and capacities that reflect a desired confidence level at a desired mean annual frequency of exceedance.

Particular care should be exercised regarding the characterization of the probabilistic distribution of each demand parameter, as the approach assumes that such distributions are approximately lognormal. While the studies undertaken in this project generally suggest the lognormal distribution is applicable, three cases exist that may need special consideration.

First, if several analyses do not converge numerically, this may indicate that global lateral dynamic instability has occurred. If lateral dynamic instability occurs for more than 16% of the total number of dynamic analyses performed using both subsets A and B, then the present methodology should be augmented by explicitly incorporating the probability of collapse [e.g., see Jalayer and Cornell (2009) and references therein], preferably using explicit numerical integration rather than parametric formulas to estimate the mean annual frequency of exceeding the capacity to compare with the desired frequency.

Second, the response may in some cases appear to be saturated, i.e., bounded by a maximum value at which many individual response quantities are clustered. For

example, this may occur for component-level force-related response quantities, such as moments, shear forces, or axial forces, that may repeatedly reach the member capacity in multiple response history analyses. Then, estimates obtained using the approximate approach, with suggested robust estimators of the median and dispersion, will tend to become biased on the safe side, as the lognormal distribution assumes unbounded values of response. If tighter control on the implied conservatism is needed, explicit numerical integration can be performed or an implicit assessment may be used (within the present methodology) where a physically related displacement parameter can be used in place of the saturated force quantity. For example, plastic hinge rotation, axial deformation, or shear deformation may be substituted for saturated moments, axial forces, or shear forces, as the former are unbounded and will better conform to a lognormal assumption.

Third, at low intensity levels, a strongly bimodal distribution may be encountered in the response, as often happens for plastic hinge rotations. Such response quantities may display several near-zero values for some records that do not induce plastic deformation. A conservative approximation may be achieved by trimming off the near-zero values—these are of no practical engineering significance, and the median and dispersion can be estimated on the basis of the remaining values. Using only the untrimmed values will bias the assessment on the conservative side; if this is undesirable, explicit numerical integration may be performed. Alternatively, distributions of total rotations (relative to the chord) may be used; by including elastic contributions, the distributions will be closer to lognormal.

9.5 Introducing Variability in Nonlinear Static Analysis

Regardless of the analysis method employed in seismic design, the real response is nonlinear, dynamic, and uncertain. Linear static and linear dynamic analysis procedures are used for nearly all design, evaluation, and rehabilitation projects, with dynamic procedures being indicated in cases where multiple-degree-of-freedom effects are expected to be important. As recommended in Chapter 8, nonlinear static analysis is always a useful tool, because it provides insights that cannot be obtained using linear procedures alone. However, the results of nonlinear analyses can be more sensitive to modeling assumptions and input parameters than those from linear analyses. Higher mode effects and irregularities are known to affect the accuracy of nonlinear static analysis relative to results obtained by nonlinear response history analysis. Also, nonlinear static analysis cannot be used to directly assess its own accuracy and to introduce conservatism in a targeted manner exactly where it is needed.

Even where the single, deterministic answer produced by nonlinear static analysis is judged to be sufficiently accurate, it is still uncertain. While that uncertainty can be considered in a crude or ad hoc manner, such as the ASCE/SEI 41-06 requirement to develop the force versus displacement curve for displacements up to 150% of the target

displacement, it is possible for the design process to more rationally and consistently reflect variability.

For instance, the approach described to approximately assess structural performance using nonlinear response history analysis could be adapted for application to nonlinear static analysis. Specifically, rather than performing nonlinear response history analysis, the results of nonlinear static analysis can be used, and the possible resulting error can be reflected as an additional source of uncertainty in determining the factored demand and factored capacity. This application would upgrade the standard nonlinear static procedure so that it could clearly and consistently reflect variability.

This need not become cumbersome, as it can be presented as a process that builds on the current procedure to give it robustness and reliability that it currently lacks. It would also cast the nonlinear static procedure into a framework that is compatible with current performance-based seismic design notions, allowing transition to more advanced and robust methods of analysis as needed, including the approximate nonlinear response history analysis approach described above.

This is not to suggest that this approach can transform nonlinear static analysis into a highly accurate and unbiased method. Rather, it may be that the required degree of conservatism may become prohibitively large in cases where the nonlinear static analysis is known to be inaccurate, assuming such cases are identified correctly. Greater accuracy should reduce the conservatism imposed by the method and can be obtained by using more accurate (and more complex) methods.

9.6 Suggestions for Further Study

The following areas of further study related to quantification of variability and uncertainty for design are suggested:

- Clarification of the range of applicability and potential limitations of the approximate nonlinear static and nonlinear response history analysis approaches, considering a broader range of building types, numbers of stories, and demand parameters.
- Elaboration of the relative merits of the S_a and S_{di} approaches, as well as the latitude provided by use of S_{di} for record selection.
- Improved quantification of uncertainty terms, and further calibration of the $\beta_{u,sub}$ term.
- Evaluation of the use of spectrum compatible motions in conjunction with S_a or S_{di} scaling to comprise or augment subsets A and B.
- Investigation of methods to identify inelastic mechanisms and the possible need to use larger subset sizes where multiple inelastic mechanisms are evident.

- Comparison of effects of approximate nonlinear static and nonlinear response history analysis approaches with results from code-based linear static and linear dynamic approaches on performance for a variety of building types.
- Development of benchmark problems (analogous to those in Appendix 7 of ANSI/AISC 360-05, *Specification for Structural Steel Buildings* (AISC, 2005b), which are used to assess the accuracy of second order analysis methods or computer programs) for use in both assessing the accuracy of and calibrating approximate nonlinear static and nonlinear response history analysis approaches.
- Establishment of appropriate statistical measures (median, mean, maximum, other) and/or mean annual frequencies of exceedance for ductile versus brittle elements, for lateral versus gravity elements, and for systems versus components.

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