

COMPUTATIONAL AND EXPERIMENTAL STUDY ON THE BEHAVIOR OF DIAPHRAGMS IN STEEL BUILDINGS

Gengrui Wei

Dissertation submitted to the faculty of the
Virginia Polytechnic Institute and State University
in partial fulfillment of the requirements for the degree of

Doctor of Philosophy
in
Civil Engineering

Mathew R. Eatherton, Chair
W. Samuel Easterling
Ioannis Koutromanos
Benjamin W. Schafer

December 13, 2021
Blacksburg, VA

Keywords: Bare Steel Deck Diaphragms, Concrete-filled Steel Deck Diaphragms, Standing
Seam Roof Diaphragms, Finite Element Analysis, Diaphragm Design Procedures

COMPUTATIONAL AND EXPERIMENTAL STUDY ON THE BEHAVIOR OF DIAPHRAGMS IN STEEL BUILDINGS

Gengrui Wei

ABSTRACT

The lateral force resisting system (LFRS) of a steel building consist of two parts, i.e., a vertical LFRS such as braced frames or shear walls, and a horizontal LFRS with diaphragms playing a crucial role. There are various types of floor and roof diaphragms in steel buildings, such as concrete-filled steel deck diaphragms for the floor system and bare steel deck diaphragms for the roof system of a typical steel braced frame building, and standing seam roof diaphragms for a typical metal building. Compared to vertical elements of a building's LFRS, our understanding of the horizontal elements, i.e., the diaphragms, is grossly lacking. The motivation for this work comes from the gaps identified in the research, including the lack of generally adopted acceptance criteria and modeling protocols for seismic performance-based design of bare steel deck and concrete-filled steel deck diaphragms through linear and nonlinear analysis, the need to better understand the complex behavior of concrete-filled steel deck diaphragms with irregular configurations such as reentrant corners and openings under lateral loading, the absence of appropriate R_s values for the alternative diaphragm seismic design approach in the current building code that considers diaphragm inelasticity, and the demand for understanding the in-plane behavior of a standing seam roof system and its use in lateral bracing of rafters in metal buildings.

A series of computational and experimental studies were conducted to investigate the behavior of diaphragms in buildings systems, including: 1) development of acceptance criteria and modeling protocol for performance-based seismic design of bare and concrete-filled steel deck diaphragms using a database of existing cantilever diaphragm tests; 2) a computational study on the nonlinear behavior of diaphragms with irregular configurations under lateral loading using high-fidelity finite element models validated against experiment test results; 3) investigation of the seismic behavior and performance of steel buildings with buckling restrained braced frames that considers different diaphragm design approaches and diaphragm inelasticity using nonlinear three-dimensional (3D) computational models; and 4) an experimental study that investigated the in-

plane behavior of full-scale standing seam roof assemblies and their use in lateral bracing of rafters in metal building systems.

The results of these studies contribute to a better understanding of the behavior of diaphragms in steel buildings and lead to several recommendations for diaphragm design. Firstly, a series of m -factors (ductility measures) and nonlinear modeling parameters (multi-linear cyclic backbone curves) were determined for bare steel deck diaphragms and concrete-filled steel deck diaphragms. These new provisions are recommended for adoption in ASCE 41 / AISC 342, which allows the use of ductility in steel deck diaphragms for their design and retrofits. Secondly, results of the finite element analysis on concrete-filled steel deck diaphragms revealed a concentrated distribution of shear transfer through the shear connections on the collectors of the diaphragm near braced frames and a stress concentration in the composite slab near reentrant corners and openings. Thirdly, results of eigenvalue analyses with nonlinear 3D building models showed that the consideration of diaphragm flexibility led to an increase in first mode period between 13% and 48%. A comparison of results from pushover analyses and response history analyses indicated that even though the pushover analyses (based on a first mode load pattern) identified the BRBF as being weaker than the diaphragms and therefore dominating the inelastic pushover behavior, response history analyses demonstrated that the diaphragms can experience substantial inelasticity during a dynamic response. The response history results also suggest that there would be a significant difference in seismic behavior of buildings modeled as two-dimensional (2D) planar frames as compared to the 3D structures modeled herein. Furthermore, the observed final collapse mode involves an interaction between large BRBF story drifts combined with diaphragm deformations that are additive and exacerbate second order effects leading to collapse. The computed adjusted collapse margin ratios for all buildings satisfied the FEMA P695 criteria for acceptance. Therefore, it is concluded that the alternative diaphragm design procedure with the proposed R_s values ($R_s = 2$ for concrete-filled steel deck diaphragm and $R_s = 2.5$ for bare steel deck diaphragm) are reasonable for use in design of these types of structures. Lastly, the effects of different standing seam roof configurations (panel type, clip type, thermal insulation, and purlin spacing) on the in-plane stiffness and strength of the standing seam roof system were investigated through an experimental testing program, and a method was described to use these experimental results in the calculations of required bracing for metal building rafters.

COMPUTATIONAL AND EXPERIMENTAL STUDY ON THE BEHAVIOR OF DIAPHRAGMS IN STEEL BUILDINGS

Gengrui Wei

GENERAL AUDIENCE ABSTRACT

A diaphragm is a horizontal structural component (e.g. floors and roof) that transfers lateral forces induced by wind or earthquakes to the vertical portions (e.g. frames and walls) of the lateral force resisting system (LFRS) of the building. There are various types of floor and roof diaphragms in steel buildings, such as concrete-filled steel deck diaphragms for the floor system and bare steel deck diaphragms for the roof system of a typical steel braced frame building, and standing seam roof diaphragms for a typical metal building. Compared to vertical elements of a building's LFRS, our understanding of the horizontal elements, i.e., the diaphragms, is grossly lacking. To address the research gaps in understanding the behavior of diaphragms and utilizing them in building design, this work presents a series of computational and experimental studies. In the first study, past experimental test data were analyzed to develop acceptance criteria and modeling protocol for performance-based seismic design of steel deck diaphragms. In the second study, finite element analyses were conducted to understand the nonlinear behavior of concrete-filled steel deck diaphragms subjected to in-plane lateral loading. In the third study, nonlinear three-dimensional computational building models were developed to investigate the seismic behavior and performance of steel buildings with different diaphragm design approaches and diaphragm inelasticity. In the fourth study, experimental testing on full-scale standing seam roof assemblies was conducted to investigate their in-plane behavior and their use in lateral bracing of rafters in metal building systems. The results of these studies contribute to a better understanding of the behavior of diaphragms in steel buildings and lead to several recommendations for diaphragm design.

To my father,

韦益松,

with love.

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to everyone who has helped me throughout my PhD journey at Virginia Tech. First and foremost, I am extremely grateful to my advisor, Dr. Matthew R. Eatherton, for his constant support and inspiration at every stage of my research projects. He has demonstrated extraordinary commitment and effectiveness as a mentor, with persistent efforts to provide mentoring for both my academic and professional development. His guidance on conducting research and solving problems will be invaluable to me throughout my professional life. I would not be where I am today if not for the training and mentoring I received from him. I would also like to extend my sincere gratitude to my committee members Dr. W. Samuel Easterling, Dr. Benjamin W. Schafer, and Dr. Ioannis Koutromanos for their continuous guidance and support on my research. Special thanks go to my colleagues and friends in the Department of Civil and Environmental Engineering at Virginia Tech and the Steel Diaphragm Innovation Initiative (SDII) team for their kind help and company during my PhD study.

This work was supported by the National Science Foundation under Grant No. 1562669 and the Steel Diaphragm Innovation Initiative which is funded by the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), the Steel Deck Institute (SDI), the Steel Joist Institute (SJI), and the Metal Building Manufacturers Association (MBMA). The Advanced Research Computing (ARC) at Virginia Tech provided high-performance computing resources which facilitated the computational study. The experimental study was supported by MABA and the materials were donated by Chief Buildings, Schulte Building Systems, and Bay Insulations Systems. Any opinions expressed in this dissertation are those of the author alone, and do not necessarily reflect the views of the National Science Foundation or the other sponsors.

Finally, and most importantly, I am deeply grateful to my parents, my wife and my son for their unconditional love and unwavering support during my adventure in the academic world. I owe them greatly and it is to them that I would love to dedicate this work.

TABLE OF CONTENTS

ABSTRACT.....	ii
GENERAL AUDIENCE ABSTRACT.....	iv
DEDICATION.....	v
ACKNOWLEDGEMENTS	vi
TABLE OF CONTENTS	vii
LIST OF FIGURES	xi
LIST OF TABLES	xx
CHAPTER 1. INTRODUCTION.....	1
1.1. Diaphragms in Steel Buildings.....	1
1.2. Motivation and Research Gaps	4
1.3. Objectives and Scope	7
1.4. Major Contribution.....	9
1.5. Organization of This Dissertation	10
CHAPTER 2. LITERATURE REVIEW.....	11
2.1. Introduction	11
2.2. Computational Studies	11
2.2.1. Finite Element Analysis of Composite Beams and Diaphragms.....	11
2.2.2. Seismic Performance Assessment of Steel Buildings Using FEMA P695 Methodology.....	19
2.3. Experimental Tests.....	22
2.3.1. Shear Connector Pushout Tests	22
2.3.2. Cantilever Diaphragm Tests	25
2.3.3. Standing Seam Roof Tests.....	29

CHAPTER 3. MODELING PARAMETERS AND ACCEPTANCE CRITERIA FOR SEISMIC EVALUATION OF STEEL DECK DIAPHRAGMS.....	32
3.1. Introduction and Motivation.....	32
3.2. Background	33
3.3. Development of Subassembly m-factors.....	36
3.4. Correction from Sub-assembly to Full Diaphragm	41
3.5. Development of Nonlinear Modeling Parameters and Acceptance Criteria	43
3.6. Conclusions	44
CHAPTER 4. FINITE ELEMENT MODELING OF CONCRETE-FILLED STEEL DECK DIAPHRAGMS UP TO FAILURE.....	46
4.1. Goals and Scope.....	46
4.2. Representation of Shear Stud Connections Using Nonlinear Springs	47
4.3. Finite Element Modeling of Concrete-filled Steel Deck Cantilever Diaphragm Tests ..	53
4.4. Computational Study of Diaphragm Behavior Using Finite Element Models.....	57
4.3.1. Prototype Structures and Modeling Scheme	57
4.3.2. Results and Discussion	61
4.5. Conclusions	70
CHAPTER 5. COMPUTATIONAL STUDY USING NONLINEAR 3D BUILDING MODELS	74
5.1. Introduction	74
5.2. Development of Archetype Buildings.....	75
5.3. Development of Computational Models	80
5.3.1. Modeling of Diaphragms.....	80
5.3.2. Modeling of Buckling-Restrained Braces	87
5.3.3. Other Modeling Details	89
5.3.4. Processing of Analysis Results.....	91

5.3.5. Type of Analyses and Related Issues	93
5.4. Results and Discussion.....	100
5.4.1. Eigenvalue Analysis	100
5.4.2. Nonlinear Static Pushover Analysis	102
5.4.3. Nonlinear Response History Analysis.....	104
5.5. Evaluation of 3D Effects on Seismic Collapse of Buildings	126
5.5.1. Details of Building Models.....	126
5.5.2. Analysis Results	127
5.6. Conclusions	133
CHAPTER 6. EXPERIMENTAL STUDY ON STANDING SEAM ROOF	137
6.1. Introduction	137
6.1.1. Overview	137
6.1.2. Motivation	137
6.1.3. Scope of Work.....	139
6.2. Testing Program	139
6.2.1. Test Matrix	139
6.2.2. Specimen Construction.....	143
6.2.3. Test Setup	144
6.2.4. Instrumentation.....	146
6.2.5. Loading Protocol	146
6.3. Results and Discussion.....	148
6.3.1. Test Results.....	148
6.3.2. Effect of Different Parameters.....	160
6.4. Application in Design and Practice.....	165
6.4.1. Concepts	165

6.4.2. Example	172
6.4.3. Discussion.....	176
6.5. Conclusions and Recommendations.....	177
6.5.1. Conclusions	177
6.5.2. Recommendations for Future Work	178
CHAPTER 7. SUMMARY.....	179
REFERENCES.....	181
APPENDIX A. ADDITIONAL INFORMATION FOR COMPUTATIONAL STUDY USING NONLINEAR 3D BUILDING MODELS	191
A.1. Member Sizes of Archetype Buildings	191
A.2. Modification of Pinching4 Backbone Parameters for Diaphragm Models	194
A.3. Additional Information about Nonlinear Response History Analysis Results.....	198
APPENDIX B. ADDITIONAL INFORMATION FOR EXPERIMENTAL STUDY ON STANDING SEAM ROOF	202

LIST OF FIGURES

Figure 1-1 Roles of diaphragm in a building system [from Sabelli et al. (2011)]	1
Figure 1-2 Components of a typical diaphragm system	2
Figure 1-3 Components of a typical bare steel deck roof system	3
Figure 1-4 Components of a typical concrete-filled steel deck floor system	3
Figure 1-5 Components of a typical standing seam roof system	4
Figure 2-1 Shear stud represented by spring element in the finite element model of a composite beam [from (Queiroz et al., 2007)]	13
Figure 2-2 Shear stud represent by beam element in the finite element model of a composite plate girder [from (Baskar et al., 2002)]	13
Figure 2-3 Predicted stud failure in a composite beam finite element model using solid elements for shear studs [from (Katwal et al., 2020)]	14
Figure 2-4 Assembly of finite element model for a composite slab using membrane elements for concrete slab [from Sebastian and McConnel (2000)]	15
Figure 2-5 Segment equilibrium analysis for a composite beam [from (Ayoub et al., 2000)).....	16
Figure 2-6 Typical pushout test setup [from (Rambo-Roddenberry, 2002)]	23
Figure 2-7 Schematic view of bare steel deck cantilever diaphragm test setup [from (O'Brien et al., 2017)]	27
Figure 2-8 Typical test setup for concrete-filled steel deck cantilever diaphragm tests [from (O'Brien et al., 2017)]	28
Figure 2-9 Typical test setup of standing seam roof base test [from (Trout, 2000)]	31
Figure 3-1 Definition of (a) shear angle (γ) and normalized load (S) in typical cantilever test and (b) stiffness (G'), strength (S_{max}), and subassembly ductility (μ_{sub}) evaluated from test results [from (O'Brien et al., 2017)]	33
Figure 3-2 ASCE41 Multi-linear Backbone Response Curve (ASCE 41-17)	36

Figure 3-3 Backbone definitions (red curves) initially considered in development for example data of a concrete-filled steel deck	37
Figure 3-4 Comparison of (a) initial bounding fit developed by the team and (b) EEEP fit recommended by TG for bare steel deck diaphragm with PAF structural connector and screwed sidelaps.....	37
Figure 3-5 Details of the EEEP model.....	38
Figure 3-6 Example EEEP fits to available data for bare steel deck diaphragms.....	39
Figure 3-7 Shear distribution in (a) sub-assembly test and (b) prototypical diaphragm span in a building	41
Figure 3-8. Normalized backbone curve for steel deck diaphragm	43
Figure 4-1 Backbone curve and pinching paths of Pinching4 material model [from (Ding, 2015)]	48
Figure 4-2 Verification of UEL with Pinching4 material in ABAQUS	49
Figure 4-3 NEU shear connector pushout test setup [from (Briggs et al., 2021)]	50
Figure 4-4 Calibration of Pinching4 material model against pushout tests	51
Figure 4-5 Finite element modeling scheme of a concrete-filled steel deck diaphragm assembly	55
Figure 4-6 Schematic stress-strain relationship for concrete damage plasticity model [from (Nguyen and Kim, 2009)]	55
Figure 4-7 Load-deformation plots of cantilever diaphragm tests.....	56
Figure 4-8 Contours of maximum principal stress in composite slab of cantilever diaphragm test specimens.....	57
Figure 4-9 Prototype diaphragm structures for finite element analysis.....	59
Figure 4-10 Finite element models for prototype floor diaphragms.....	60
Figure 4-11 Force-displacement plots for prototype diaphragms.....	61
Figure 4-12 Contour of shear stress magnitude in the composite slab of prototype diaphragms with concrete damage plasticity	64

Figure 4-13 Distribution of shear connection forces for prototype diaphragms with concrete damage plasticity	66
Figure 4-14 Free-body diagram of prototype diaphragm B.....	67
Figure 4-15 Example of effective shear transfer length calculation for prototype diaphragm A with concrete damage plasticity	68
Figure 4-16 Distribution of stud forces for prototype diaphragms with elastic composite slab...	68
Figure 4-17 Progression of shear connection failure along the left edge of prototype diaphragm B with elastic composite slab	70
Figure 5-1 Typical Floor Framing Plan	77
Figure 5-2 Typical Roof Framing Plan.....	78
Figure 5-3 Elevation view of 4-story building braced frames	78
Figure 5-4 3D OpenSees models of archetype buildings.....	80
Figure 5-5 Test setup and computational model of cantilever diaphragm test.....	81
Figure 5-6 Diaphragm meshing in computational models of archetype buildings.....	82
Figure 5-7 Pinching4 material model [from (Ding, 2015)]	83
Figure 5-8 Hysteretic response of diaphragm from experiment and simulation	84
Figure 5-9 Configuration of a typical BRB and computational model.....	88
Figure 5-10 Calibration of BRB Steel4 models for Specimens 1G and 3G	88
Figure 5-11 Schematic view of BRB modeling in 4-story archetype building.....	88
Figure 5-12 Flow chart of the algorithm for convergence tests.....	91
Figure 5-13 Lateral force distribution on 4-story archetype building for pushover analysis	93
Figure 5-14 Example ground motion scaling for DE and MCE (4-story building).....	96
Figure 5-15 Example time history of maximum story drift for analysis with convergence failure considered as building collapse (4-story Trad. / Alt.2, Ground Motion Set 21).....	99
Figure 5-16 Example time history of maximum story drift for analysis with convergence failure excluded from collapse ratio calculation (8-story Trad. / Alt.2, Ground Motion Set 26)	100

Figure 5-17 Mode shapes for the 1 st mode of four-story archetype models	101
Figure 5-18 Mode shapes of four-story archetype models	101
Figure 5-19 Pushover curves of archetype buildings with different diaphragm design procedures	103
Figure 5-20 Deformed shapes of archetype buildings with Trad. or Alt. 2 diaphragm design procedures (deformation amplification factor: 5)	104
Figure 5-21 Time history response of 4-story building with the Traditional / Alternative 2 diaphragm design under three levels of ground motions (from top to bottom: peak story drift, base story BRB hysteresis, floor diaphragm truss hysteresis, roof diaphragm truss hysteresis).....	106
Figure 5-22 Deformed shapes of 4-story archetype building with the Traditional / Alternative 2 diaphragm design under three levels of ground motions (deformation amplification factor: 10)	107
Figure 5-23 Time history of peak story drift in x and y directions of 4-story building with Traditional / Alternative 2 diaphragm design under MCE-level ground motion: total story drift vs. BRBF story drift	108
Figure 5-24 Example time history of base shear in x and y directions of 4-story building with Traditional / Alternative 2 design under DE and MCE-level ground motions: total base shear vs. BRBF base shear.....	109
Figure 5-25 Base shear vs. story drift hysteretic curves of 4-story building with Traditional / Alternative 2 diaphragm design under MCE-level ground motion	110
Figure 5-26 Contour of normalized diaphragm shear angle and normalized BRB strain of 4-story building with Traditional / Alternative 2 diaphragm designs under MCE-level ground motion	111
Figure 5-27 Contour of normalized diaphragm shear angle demand and normalized BRB strain demand of 4-story building with different diaphragm designs under DE and MCE-level ground motions.....	112
Figure 5-28 Distribution of median peak story drifts at each story along building height of 12-story archetype buildings with Trad. or Alt. 2 design under three levels of ground motions.....	114
Figure 5-29 Distribution of median peak resultant story drift along building height.....	115

Figure 5-30 Distribution of median peak story drift in x direction along building height	115
Figure 5-31 Distribution of median peak story drift in y direction along building height	116
Figure 5-32 Diaphragm shear demand of archetype buildings with Alternative 1 diaphragm design normalized by diaphragm design shear.....	117
Figure 5-33 Collapse ratio breakdown for buildings under three levels of ground motions	122
Figure 5-34 Example lognormal cumulative distribution plots for 4-story archetype with different diaphragm designs	124
Figure 5-35 Typical BRBF elevations of 8-story archetype building.....	127
Figure 5-36 Distribution of median peak story drift in y direction along building height with different diaphragm modeling assumption and bidirectional earthquake loading.....	129
Figure 5-37 Building response with rigid and inelastic diaphragms subjected to ACMR _{10%} hazard level and bidirectional ground motions for the Nishi-Akashi motion of 1995 Kobe, Japan earthquake	130
Figure 5-38 Distribution of median peak story drifts along building height for building models with rigid diaphragms considering unidirectional and bidirectional earthquake loading at MCE hazard level	132
Figure 6-1 Lateral bracing and load transfer of main frame	138
Figure 6-2 Lateral load from rafter lateral torsional buckling acting on purlins and SSR panels	139
Figure 6-3 Typical specimen layout	141
Figure 6-4 SSR panels used in the experimental program.....	142
Figure 6-5 Clips used in the experimental program.....	142
Figure 6-6 Specimen construction process	143
Figure 6-7 Pictures showing the seaming process in sequential order	144
Figure 6-8 Schematic view of test setup	145
Figure 6-9 Details of fixed support.....	145
Figure 6-10 Details of sliding support	146

Figure 6-11 Reaction Frame	146
Figure 6-12 Schematic view of instrumentation layout.....	147
Figure 6-13 Picture of test setup and specimen prior to testing.....	147
Figure 6-14 Typical deformed configuration with clip bending (from Specimen 7)	148
Figure 6-15 Typical deformed shape after panel buckling (from Specimen 7).....	149
Figure 6-16 Mechanical detachment of clips from top (left) and bottom (right) (from Specimen 8)	149
Figure 6-17 Tearing of clip material at the bottom (from Specimen 7).....	149
Figure 6-18 Load-displacement curve of Specimen 1	150
Figure 6-19 Load-displacement curve of Specimen 2	150
Figure 6-20 Load-displacement curve of Specimen 3	151
Figure 6-21 Load-displacement curve of Specimen 4	151
Figure 6-22 Load-displacement curve of Specimen 5	151
Figure 6-23 Load-displacement curve of Specimen 6	152
Figure 6-24 Load-displacement curve of Specimen 7	152
Figure 6-25 Load-displacement curve of Specimen 8	152
Figure 6-26 Load-displacement curve of Specimen 9	153
Figure 6-27 Load-displacement curve of Specimen 10	153
Figure 6-28 Load-displacement curve of Specimen 11	153
Figure 6-29 Example displacement curves for all purlins (from Specimen 1)	155
Figure 6-30 Free body diagram of clips and panels.....	155
Figure 6-31 Middle purlin clip and panel deformation curves	157
Figure 6-32 Panel slip measurement affected by panel buckling (from Specimen 2).....	159
Figure 6-33 Slip between adjacent panels (left: Specimen 4; right: Specimen 5).....	159
Figure 6-34 Load-displacement curves of specimens with different panel types.....	161

Figure 6-35 Stiffening and restraint of clips by standing seams with different profiles.....	162
Figure 6-36 Load-displacement curves of specimens with different clip types	163
Figure 6-37 Load-displacement curves of specimens with or without thermal insulation	164
Figure 6-38 Load-displacement curves of specimens with different purlin spacing.....	165
Figure 6-39 Plan view of typical metal building roof framing with assumed buckled shape of rafter	166
Figure 6-40 Section view of adjacent rafters braced by SSR and flange braces with assumed buckled shape.....	166
Figure 6-41 Structure assemblage for analyzing torsional bracing strength and stiffness of rafter provided by flange brace and purlin	170
Figure 6-42 Moment and axial force diagram of the assemblage subjected to concentrated moment applied at the top of rafter.....	171
Figure 6-43 Plan view of roof framing detail of prototype metal building	173
Figure A-1 Comparison of the diaphragm test specimen and archetype diaphragm mesh unit .	194
Figure A-2 Lognormal cumulative distribution plots for the archetype buildings	200
Figure B-1 Load-displacement curve for Specimen 1	202
Figure B-2 Purlin displacements of Specimen 1.....	202
Figure B-3 Deformed shape of Specimen 1.....	203
Figure B-4 Clip damage of Specimen 1.....	203
Figure B-5 Load-displacement curve for Specimen 2	203
Figure B-6 Purlin displacements of Specimen 2.....	204
Figure B-7 Deformed shape of Specimen 2.....	204
Figure B-8 Clip damage of Specimen 2.....	204
Figure B-9 Load-displacement curve for Specimen 3	205
Figure B-10 Purlin displacements of Specimen 3.....	205
Figure B-11 Deformed shape of Specimen 3.....	205

Figure B-12 Clip damage of Specimen 3.....	206
Figure B-13 Load-displacement curve for Specimen 4	206
Figure B-14 Purlin displacements of Specimen 4.....	206
Figure B-15 Deformed shape of Specimen 4.....	207
Figure B-16 Clip damage of Specimen 4.....	207
Figure B-17 Load-displacement curve for Specimen 5	207
Figure B-18 Purlin displacements of Specimen 5.....	208
Figure B-19 Deformed shape of Specimen 5.....	208
Figure B-20 Clip damage of Specimen 5.....	208
Figure B-21 Load-displacement curve for Specimen 6	209
Figure B-22 Purlin displacements of Specimen 2.....	209
Figure B-23 Deformed shape of Specimen 6.....	209
Figure B-24 Clip damage of Specimen 6.....	210
Figure B-25 Load-displacement curve for Specimen 7	210
Figure B-26 Purlin displacements of Specimen 7.....	210
Figure B-27 Deformed shape of Specimen 7.....	211
Figure B-28 Clip damage of Specimen 7.....	211
Figure B-29 Load-displacement curve for Specimen 8	211
Figure B-30 Purlin displacements of Specimen 8.....	212
Figure B-31 Deformed shape of Specimen 8.....	212
Figure B-32 Clip damage of Specimen 8.....	212
Figure B-33 Load-displacement curve for Specimen 9	213
Figure B-34 Purlin displacements of Specimen 9.....	213
Figure B-35 Deformed shape of Specimen 9.....	213
Figure B-36 Clip damage of Specimen 9.....	214

Figure B-37 Load-displacement curve for Specimen 10	214
Figure B-38 Purlin displacements of Specimen 10.....	214
Figure B-39 Deformed shape of Specimen 10.....	215
Figure B-40 Clip damage of Specimen 10.....	215
Figure B-41 Load-displacement curve for Specimen 11	215
Figure B-42 Purlin displacements of Specimen 11.....	216
Figure B-43 Deformed shape of Specimen 11.....	216
Figure B-44 Clip damage of Specimen 11.....	216

LIST OF TABLES

Table 2-1 Shear connector pushout tests in the literature	25
Table 2-2 Test Matrix for Cantilever Diaphragm Tests on Concrete-filled Steel Deck Diaphragms by Avelleneda-Ramirez et al. (2021)	29
Table 3-1 Cyclic cantilever diaphragm test results for bare deck from O'Brien et al. (2017)	34
Table 3-2 Cyclic cantilever diaphragm test results for filled deck from O'Brien et al. (2017) note all tests in this table from Porter and Easterling (1988).....	35
Table 3-3 Definition of m-factors per ASCE41-17 (reproduced from Ayhan et al. 2016)	39
Table 3-4 Developed subassembly m-factors from existing test data for steel deck diaphragms	40
Table 3-5 System level m-factors from existing test data for steel deck diaphragms assuming $L_p/L = 0.1$	42
Table 3-6. Developed nonlinear modeling parameters and acceptance criteria from existing test data for steel deck diaphragms.....	44
Table 4-1 NEU Pushout Test Matrix [from (Briggs et al., 2021)].....	51
Table 4-2 Calibrated Pinching4 Material Parameters	52
Table 4-3 Initial stiffness and peak forces of prototype diaphragms.....	63
Table 5-1 List of BRB Archetype Buildings for the Study	76
Table 5-2 Archetype Building Loading and Design Information.....	76
Table 5-3 Prototype Building Seismic Design Information	77
Table 5-4 Diaphragm Design Shear per Unit width at Diaphragm Edge along Short Dimension of Building.....	79
Table 5-5 Calibrated Pinching4 Material Model Parameters	84
Table 5-6 Diaphragm Demands and Design.....	85
Table 5-7 Diaphragm Nominal Strength and Backbone Stress Scale Factors.....	86
Table 5-8 Calibrated Steel4 Parameters.....	88

Table 5-9 Masses at Typical Node Locations.....	89
Table 5-10 Far-Field Ground Motions Used for Nonlinear Response History Analysis	94
Table 5-11 Ground Motion Scaling for all Buildings.....	98
Table 5-12 Natural Periods of Archetype Models in OpenSees and SAP2000.....	101
Table 5-13 Base Shear of 4-story Archetype Building with Traditional / Alternative 2 Diaphragm Design under DE and MCE-level Ground Motions.....	109
Table 5-14 Medians of Diaphragm Shear Demand for 4-story Archetype Buildings	117
Table 5-15 Collapse ratios for buildings under three levels of ground motions.....	123
Table 5-16 Spectral acceleration of MCE and DE ground motions	124
Table 5-17 Summary of collapse performance evaluation of archetype buildings using FEMA P695 procedure.....	125
Table 5-18 Proportion of ground motions causing collapse for models subjected to bidirectional motions.....	131
Table 5-19 Proportion of ground motions causing collapse for all models.....	133
Table 6-1 SSR Test Matrix	141
Table 6-2 Measured Seam Width	144
Table 6-3 Displacement of Middle Purlin When Clips Failed and Load at First Clip Failure...	154
Table 6-4 Maximum Load and Secant Stiffness of Specimens	160
Table 6-5 Maximum Load and Secant Stiffness $K_{40\%}$ of Specimens with Different Panel Types	161
Table 6-6 Maximum Load and Secant Stiffness $K_{40\%}$ of Specimens with Different Clip Types	163
Table 6-7 Maximum Load and Secant Stiffness $K_{40\%}$ of Specimens with or without Thermal Insulation	164
Table 6-8 Provided Strength and Stiffness per Clip of SSR for Lateral Bracing of Rafter.....	170
Table A-1 Beam Sizes of Archetype Buildings.....	191
Table A-2 Column Sizes of Archetype Buildings	192
Table A-3 BRB Core Areas (A_{core}), Yield-to-Length Ratios (YLR) and	193

Table A-4 Pinching4 Material Model Parameters Used for Archetype Building Models....	197
Table A-5 Medians of Peak Story Drifts at Each Story of Archetype Buildings under Three Ground Motion Levels.....	198
Table A-6 Medians of Diaphragm Shear Demands for Archetype Buildings and Comparison to Design Shear	199

CHAPTER 1. INTRODUCTION

1.1. Diaphragms in Steel Buildings

Loads acting on building structures, including gravity loads (such as dead and live loads) and lateral loads (such as wind and earthquake loads) are primarily transferred through the vertical and horizontal lateral force resisting systems (LFRS). While the vertical LFRS serves as an essential role to transfer the loads to the ground, the horizontal LFRS is also a key component of the load path. One of the key elements in a typical horizontal LFRS is the diaphragms, a horizontal structural component that spans the floors and roofs of a building. As shown in Figure 1-1, the key roles of diaphragms in a building system include: 1) transfer lateral inertial forces at each level to the vertical elements, 2) distribute gravity loads, 3) provide lateral support to the vertical elements, and 4) sustain soil loads below grade.

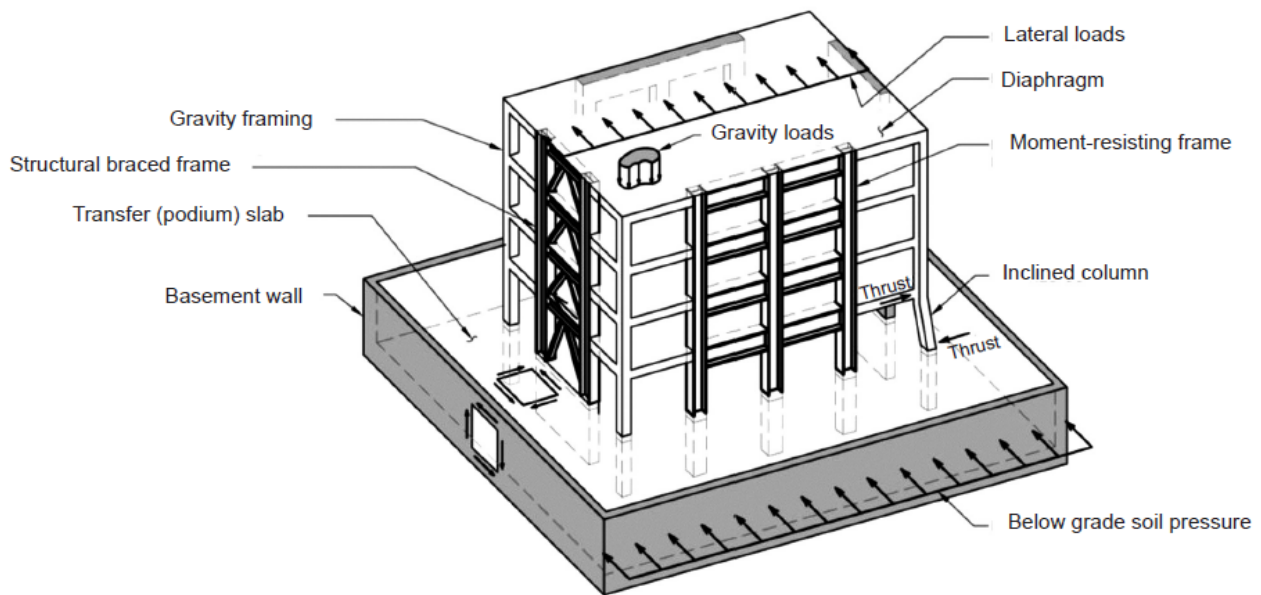


Figure 1-1 Roles of diaphragm in a building system [from Sabelli et al. (2011)]

Figure 1-2 shows the components of a typical diaphragm system, including a steel deck, chords, and collectors. When the diaphragm is subjected to wind and or earthquake-induced lateral loading, the chords of the diaphragm perpendicular to the loading direction are the elements that

resist the flexure due to the lateral loading, while the collectors of the diaphragm parallel to the loading direction are the elements that transfer the loads from the diaphragm to the vertical LFRS such as frames. The shear transfer between the deck and collectors / chords is fulfilled by the shear connectors which are typically headed shear studs.

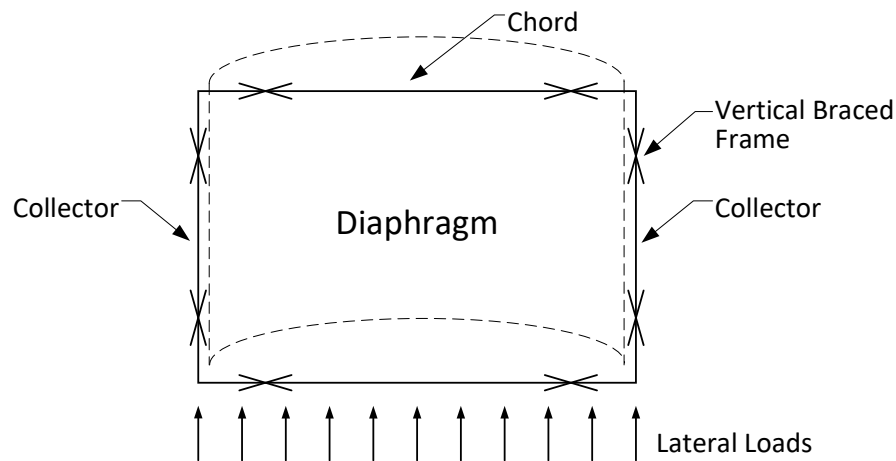


Figure 1-2 Components of a typical diaphragm system

There are three types of floor and roof systems that are widely adopted in steel / metal buildings. The most common type of floor system in steel buildings consists of concrete-filled steel deck. For the roof system, bare steel deck is typically used in multistory steel buildings, while the standing seam roof (SSR) system is usually used for single-story metal buildings. Concrete-filled steel deck and bare steel deck are considered as part of the diaphragm system, but SSR typically is not.

Steel deck diaphragms can be categorized into two groups depending on whether the steel deck is topped with concrete. Bare steel deck diaphragms without concrete fill are typically used for roof systems, and consist of profiled steel sheeting attached to structural beams by various types of fasteners, as shown in Figure 1-3. For floor systems, concrete-filled steel deck diaphragms are commonly used. They can provide considerably larger strength and stiffness than bare steel deck diaphragms for the vertical loading on the floors where the gravity loads are larger, and as a diaphragm they also have significantly larger strength and stiffness for in-plane loading especially when they are connected to the steel beams using headed shear studs. As shown in Figure 1-4, a typical concrete-filled steel deck floor system consists of a concrete slab composite with a profiled

steel deck. The deck acts compositely with the concrete through the embossments on the deck to span between beams. The steel beams also act compositely with the concrete which is facilitated by the shear connectors, typically headed shear studs, that are welded to the top flange of steel beams.

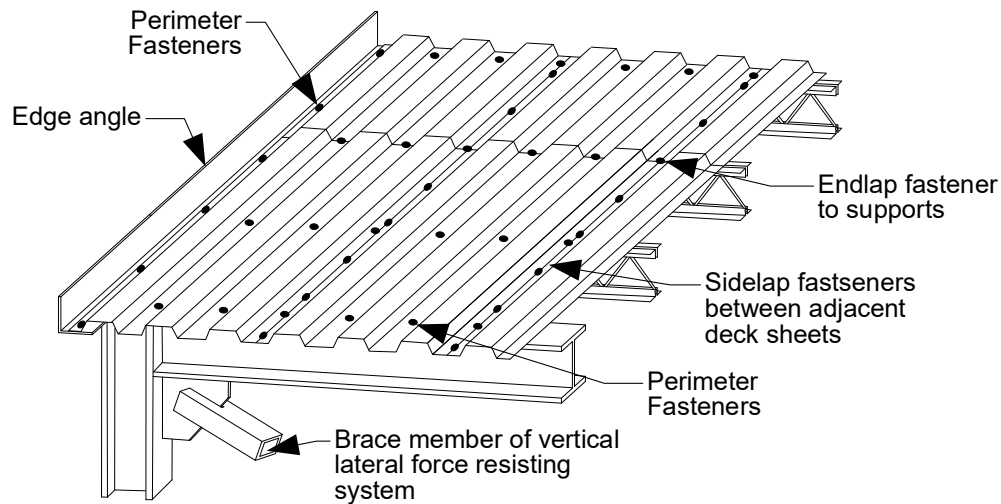


Figure 1-3 Components of a typical bare steel deck roof system

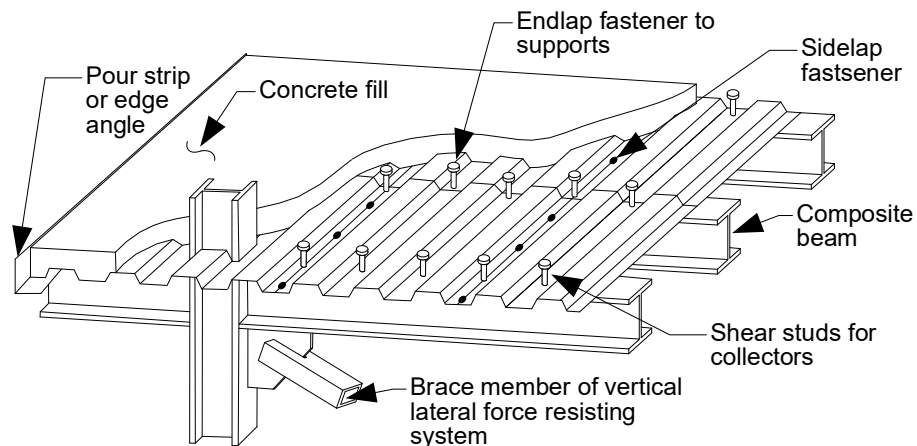


Figure 1-4 Components of a typical concrete-filled steel deck floor system

For low-rise buildings and in particular one-story buildings, metal building systems are popular because of their fast construction and cost efficiency. The roof in metal buildings typically uses a standing seam roof (SSR) system that consists of purlins, clip fasteners, SSR panels, and optional thermal blocks and blanket insulation, as is shown in Figure 1-5. Z-shaped or C-shaped purlins

provide support for the roof and are attached to the frames of the building. The SSR panels are light-gauge corrugated metal sheets which span between the purlins and connect to each other through the standing seam created by roll forming the vertical or trapezoidal legs along the panel edges. Clip fasteners are installed on the purlins and extend up into the standing seam, which after the seam is crimped provides a connection between the roof panels and purlins. Depending on the need for thermal insulation, thermal blocks and blanket insulation can also be installed underneath the roof panels.

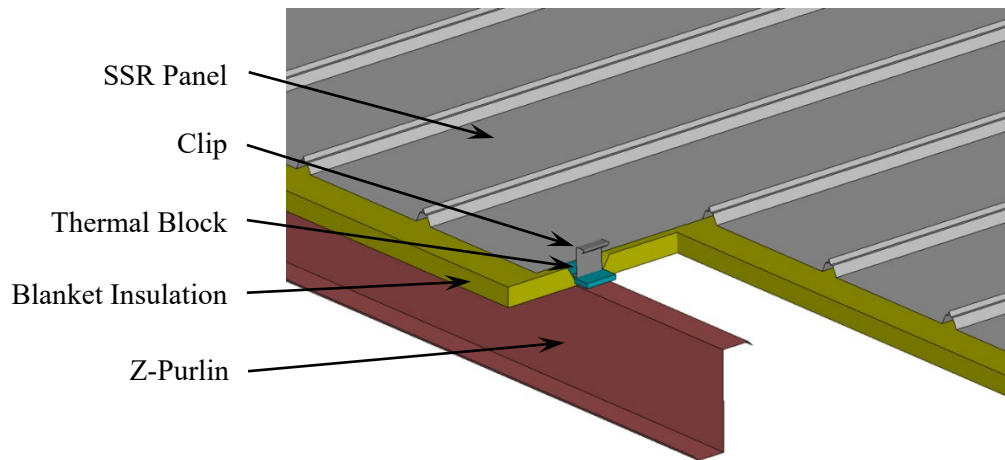


Figure 1-5 Components of a typical standing seam roof system

It should be noted that the SSR system is typically not considered as a diaphragm to resist lateral forces applied to a metal building. The rod bracing in the plane of the roof provides a load path to transfer the lateral loads to the vertical LFRS of the building. However, diaphragm action in the SSR is implicitly relied upon to brace the beams (commonly referred to as rafters in the metal building industry) as will be discussed in the next section.

1.2. Motivation and Research Gaps

Steel buildings have relied on diaphragms since the first building codes a century ago. Contemporary roof and floor systems in steel buildings have been unchanged for decades. However, many unanswered questions remain about the behavior of diaphragms during earthquakes, appropriate design of diaphragms for earthquakes, and the use of SSR diaphragm action in design, including:

- 1) What is the appropriate way for engineers to model diaphragms when evaluating existing structures and what are the appropriate acceptance criteria?
- 2) How should diaphragms be designed to prevent progressive shear connection failure and localized concrete failure along collectors and near reentrant corners and openings?
- 3) How do buildings behave as a 3D structure when subjected to earthquakes with inelasticity in the LFRS and the diaphragm?
- 4) How should buildings be designed to have adequate behavior / collapse performance for seismic hazards?
- 5) How does the 3D behavior of the building or the diaphragm modeling assumptions relate to the collapse performance of the building?
- 6) How can the SSR diaphragm action be used to laterally brace the roof beams of a metal building?

Simplified models, such as those used by engineers where the diaphragms are represented by rigid or semi-rigid shell-like elements without inelasticity, are not capable of capturing diaphragm demands (e.g. see Celebi et al. 1989). In addition, the complex nonlinear behavior of the diaphragms cannot be captured by this type of models. Currently there exists no guide for how to appropriately model the diaphragms for seismic design and analysis in practice. Developing a modeling scheme that can capture the nonlinear hysteretic behavior of the diaphragms during earthquakes while maintaining a relatively low computational cost therefore becomes necessary. Moreover, diaphragms are often treated as elastic elements in the current provisions for seismic evaluation and retrofit that may lead to uneconomical or inefficient design and retrofits. It is therefore beneficial to develop the acceptance criteria and modeling protocols for practicing engineers to design and evaluate diaphragms in steel buildings in a more economical way.

In addition to modeling the diaphragms for the seismic evaluation of building structures, it is also important to understand the distribution of shear forces in the diaphragm and the shear transfer and progression of failure for the shear connections along the collectors and near the reentrant corners and openings of diaphragms under the lateral loading. Current building code and design provisions do not specifically address the details for such behavior of diaphragms with different configurations. Research is needed to fill this gap in order to properly design the diaphragms to prevent potential types of failure.

Conventional diaphragm design of steel buildings assumes that inelasticity occurs primarily in the vertical LFRS. However, it has been shown that diaphragms designed using traditional design procedures may be subject to inelasticity during design level earthquakes (Rodriguez et al, 2007), and in the extreme may cause collapse such as happened for several concrete parking garages with precast concrete diaphragms during the 1994 Northridge earthquake (EERI, 1996). While the majority of the steel deck diaphragm systems have not been reported to undergo severe damage during earthquakes in the past, gaps exist in the research that the seismic behavior and performance of 3D building structures considering diaphragm inelasticity is not well understood.

Challenges also exist in the design of buildings to have adequate behavior and collapse performance for seismic hazards. Research in the past mainly focused on the design and analysis of the vertical LFRS of buildings. For diaphragms, the current U.S. seismic design provisions ASCE 7-16 (ASCE, 2016) provide two methodologies for seismic design: traditional diaphragm design procedures using forces reduced by the response modification factor, R , associated with the vertical system, and alternative diaphragm design procedures using larger and more accurate elastic design forces. The alternative diaphragm design procedures incorporate a diaphragm design force reduction factor, R_s , that reduces the diaphragm demands based on the ductility and overstrength in the diaphragm. However, currently there is no R_s factor currently available for steel deck or concrete on steel deck diaphragms, although values for R_s have been proposed for inclusion in the upcoming edition of NERHRP Recommended Seismic Provisions including $R_s = 2.5$ for steel deck diaphragms satisfying specific special detailing requirements and $R_s = 2.0$ for composite concrete on steel deck diaphragms. Research is needed to examine the consideration of diaphragm inelasticity in seismic design of buildings and what values for R_s are appropriate.

For the evaluation of building seismic performance, two-dimensional (2D) frame analysis has been used extensively to predict the seismic response (e.g., peak story drifts and collapse) of buildings subjected to earthquake excitation. In this type of analysis, the vertical lateral force resisting systems (LFRS) in the building (i.e., braced frames or moment frames) are modeled assuming the horizontal diaphragm system is rigid and that the vertical LFRS responds independent of the rest of the building. In an actual building, however, the diaphragm system deforms contributing to lateral drifts, and interacts dynamically with the vertical LFRS. Furthermore, inelasticity and failure of the diaphragm can have substantial effect on the seismic

behavior and collapse of the overall building. It has remained unknown how the 3D behavior of the building and the diaphragm modeling assumptions can affect the seismic collapse performance of the building.

For metal building systems, the in-plane bracing in the roof plane is provided by tension rod or cable bracing. Additionally, the SSR system provides some restraint against the lateral motion of roof beams (rafters) through a diaphragm action because it resists relative longitudinal motion of purlins. However, the in-plane resistance of the SSR panels and their ability to brace the rafter have not been previously studied and therefore warrant further investigation.

1.3. Objectives and Scope

There are four studies presented in this work that address different research gaps related to diaphragms in steel buildings, including concrete-filled steel deck diaphragms and bare steel deck diaphragms in a typical steel building system, and standing seam roof diaphragms in a typical metal building system.

Study 1 – Guidance for Evaluation of Existing Diaphragms

The objective of the first study is to develop guidance for how practicing engineers can design and evaluate diaphragms in steel buildings. ASCE 41 (ASCE, 2017) for the seismic evaluation and retrofit of existing buildings essentially requires that steel deck diaphragms be designed as elastic elements. This potentially results in large economic and design inefficiencies. To address this issue, existing data on steel deck diaphragms was analyzed for the purposes of determining acceptance criteria for linear and nonlinear analyses consistent with the performance-based seismic design levels of ASCE 41. A series of m-factors (ductility measures) and nonlinear modeling parameters (multi-linear cyclic backbone curves) were determined for bare steel deck diaphragms and concrete-filled steel deck diaphragms. The resulting analyses form the basis for new ASCE 41 provisions and are aligned with the strength and stiffness provisions provided by AISI S310 (AISI, 2016) and AISC 341 (AISC, 2016a). The provisions are recommended for adoption in the first edition of AISC 342 (AISC, 2019), which is intended to replace ASCE 41 Chapter 9 for structural steel systems.

Study 2 – Understanding the Inelastic Behavior of Diaphragms Up to Failure

The second study aims to explore the nonlinear behavior of typical concrete-filled steel deck diaphragms subjected to increasing in-plane lateral loading up to failure. Nonlinear finite element models that can capture the nonlinear behavior of shear stud connections and tension cracking of the concrete slab using solid elements for concrete-filled metal deck, spring elements for shear stud connections, and shell elements for steel beams, were developed and validated against experimental pushout tests and cantilever diaphragm tests. The validated modeling scheme was then applied to diaphragm configurations with various lengths of collectors and geometric irregularities such as reentrant corners and openings to further investigate the shear force distribution and the progression of shear connection failure in these types of diaphragms.

Study 3 – Evaluation of Building Seismic Behavior and Performance Considering Diaphragm Inelasticity

The objectives of the third study include: 1) to understand the extents of diaphragm inelasticity at specified diaphragm hazard levels and its interaction with inelasticity in the vertical lateral force resisting system of the building, 2) to investigate the seismic collapse performance of buildings designed with different diaphragm design approaches, 3) to evaluate whether the use of proposed values of R_s are sufficient for bare steel deck and concrete-filled steel deck diaphragms, and 4) to understand the effects of 3D analysis, different diaphragm modeling assumptions, and unidirectional vs. bidirectional seismic excitation on the seismic collapse response of buildings. A computational study using three-dimensional (3D) building models that capture nonlinear diaphragm behavior and its interaction with the nonlinear vertical LFRS was conducted. A series of 1, 4, 8, and 12-story archetype buildings were designed with buckling-restrained braced frames (BRBF) for the vertical system and three designs for the diaphragms. The modeling scheme uses computationally efficient calibrated frame and truss elements to capture the realistic nonlinear behavior of both the BRBFs and the diaphragms. Modal analysis, nonlinear static pushover analyses, and nonlinear response history analyses using 44 ground motion records scaled to three hazard levels were performed to investigate the behavior and seismic performance of the buildings.

Study 4 – Investigation on SSR Diaphragm Action for Lateral Bracing of Rafters

The objective of the fourth study is to investigate the effects of different standing seam roof configurations on the in-plane stiffness and strength of the SSR system and its use in lateral bracing of rafters. Eleven full-scale in-plane bending tests on SSR systems were conducted. Each specimen was a 22 ft long SSR unit with panels and purlins that represent the portion of a roof between points where roof X bracing attaches to the rafter. Each specimen had a unique configuration to study the effect of different parameters including SSR panel profiles, SSR panel width, type of clip and screw fasteners, clip standoff, use of thermal blocks, and presence of blanket insulation. Results from the tests were analyzed and compared to the bracing requirements in AISC 360-16 (AISC, 2016b) for an example rafter.

1.4. Major Contribution

The major contribution of this work to the field of structural engineering is listed as follows:

- 1) A series of m-factors and nonlinear modeling parameters were determined for steel deck diaphragms, which lead to new provisions ready for adoption in ASCE 41 / AISC 342.
- 2) The distribution of lateral forces along the collectors, reentrant corners, and openings of a diaphragm and the progression of shear connector failure in these regions were obtained and used to provide recommendations for the design of diaphragms with various lengths of collectors and geometric irregularities.
- 3) A new understanding was created about the interaction between inelasticity in the diaphragm system and inelasticity in the vertical lateral force resisting system. This includes discoveries about how inelasticity can be mobilized in both systems simultaneously, how inelasticity in the two systems interacts near collapse, how higher modes can activate diaphragm inelasticity, and the level of forces that can be created in diaphragms during earthquakes.
- 4) Different diaphragm design approaches in the current seismic design provisions were evaluated with regard to the seismic collapse performance of the building. Values of R_s for the alternative diaphragm design procedure in ASCE 7-16 were proposed based on past work and validated for bare steel deck and concrete-filled steel deck diaphragms.

- 5) The shortcomings of 2D analyses / analyses with rigid diaphragms were identified and associated modeling recommendations were provided through analyzing the effects of 3D analysis vs. 2D analysis and different diaphragm modeling assumptions on the evaluation of seismic collapse performance of buildings.
- 6) Experimental tests were conducted to study the in-plane behavior of standing seam roof assemblies and a design approach was proposed for using standing seam roof in the lateral bracing of metal building rafters.

1.5. Organization of This Dissertation

This dissertation is organized as follows:

- Chapter 1 introduces the background, motivation, scope, and objectives of the research presented in this work.
- Chapter 2 reviews past computational and experimental studies in the literature on composite diaphragms, steel buildings with BRB systems, and standing seam roof systems.
- Chapter 3 describes the application of an existing cantilever diaphragm test database for the development of recommended seismic design provisions for steel deck diaphragms utilizing ASCE 41 / AISC 342.
- Chapter 4 describes a computational study on the behavior of concrete-filled steel deck diaphragms using nonlinear finite element models.
- Chapter 5 presents a computational study on the seismic behavior of steel buildings with BRB lateral force resisting system considering diaphragm inelasticity.
- Chapter 6 presents an experimental study on the in-plane behavior of standing seam roof diaphragm and its use in lateral bracing of rafters.

CHAPTER 2. LITERATURE REVIEW

2.1. Introduction

This Chapter reviews the past studies in the literature that relate to the research presented in this dissertation, which are divided into two categories: computational studies and experimental tests. For computational studies, finite element analysis of composite beams and diaphragms and seismic performance assessment of steel buildings using the FEMA P695 methodology (FEMA, 2009) are discussed. For experimental tests, shear connector pushout tests and diaphragm tests of various types (e.g., cantilever diaphragm tests, standing seam roof tests) are included and reviewed.

2.2. Computational Studies

Computational studies have been widely conducted to study the behavior of structures at different scales, ranging from composite beams and diaphragms to full building structures. Compared to experimental tests, the computational simulation approach is much less expensive and labor-intensive, and can enable the investigation of interested behavior at a more detailed level (for example, the stress and strain at certain locations using finite element methods). However, this approach can sometimes be challenging, as material nonlinearity, geometric nonlinearity, and nonlinear contact definition are typically involved in an involved computational study. Validating the computational modeling scheme against observed experimental test results is usually considered the most challenging task in a computational study. This section reviews the relevant computational studies including finite element analysis on composite beams and diaphragm and assessment of seismic performance of steel buildings using the FEMA P695 methodology.

2.2.1. Finite Element Analysis of Composite Beams and Diaphragms

This section provides a review of past studies on the behavior of composite beams and diaphragms using FE approach. Four topics that are of interest and considered most important in these types of studies are discussed in the following subsections, including the modeling of shear connectors, modeling of other components, modeling of contact, and the different applications of the FE study.

2.2.1.1. Modeling of Shear Connectors

In a composite beam or diaphragm, shear connectors (e.g., headed shear studs) play a key role in facilitating the composite action and serving as a load path between the concrete slab, steel deck and steel beams. Capturing the nonlinear behavior of shear connectors is essential to achieve accurate modeling of steel-concrete composite structures. Different types of elements have been used to simulate shear connectors in composite beams and diaphragms, such as beam elements, spring elements, and solid elements. This section summarizes some applications of these types of elements in modeling shear connectors.

One of the commonly used types of elements for modeling shear connectors is the spring element. Each shear connector can be represented by a nonlinear spring with predefined constitutive relationship that captures the shear behavior of the connector, which is computationally efficient. For example, Sebastian and McConnel (2000) used stub element with combined rotational and axial springs to model shear studs in steel-concrete composite structures. In-plane shear behavior of the connection was captured by axial springs along orthogonal directions using empirical force-slip relationship, while very high stiffness was assumed for the vertically oriented axial spring and rotational springs to simulate the composite action along the shear stud longitudinal direction. Queiroz et al. (2007) also modeled shear studs using nonlinear springs with user-defined uniaxial tension-compression constitutive in the shear direction and coupled nodes in the normal direction (see Figure 2-1). Similarly, Nie et al. (2012) used spring elements to model shear connectors between steel beams and concrete slabs with openings. However, an elastic constitutive model was adopted for the spring elements acting only in the longitudinal direction of the steel beam (with coupled nodes for the transverse and normal directions), since the goal of their study was to investigate the elastic rigidity (i.e. bending stiffness) of composite beams with full width slab openings.

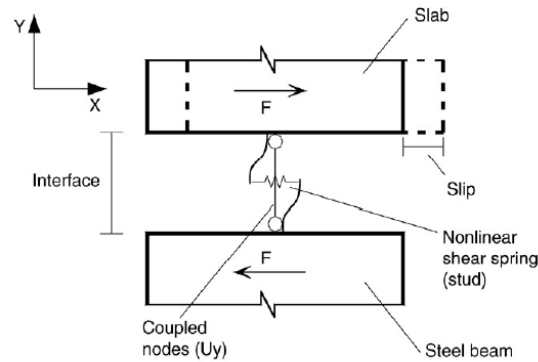


Figure 2-1 Shear stud represented by spring element in the finite element model of a composite beam [from (Queiroz et al., 2007)]

Beam elements have also been used to simulate shear connectors in composite beams and diaphragms. These elements are typically embedded in the concrete slab (usually modeled with solid elements) and attached to the steel beam, and a nonlinear material model is calibrated to capture the shear and axial behavior of the shear connectors. For example, Baskar et al. (2002) used beam elements with modified cross-sectional areas for tensile and shear forces acting on the stud to model the bond strength of concrete in addition to the strength of the studs. Sadek et al. (2008) assumed a fully bonded interface between shear studs and concrete by embedding the beam elements representing shear studs in the concrete slab. A bilinear stress-strain relationship was adopted for the beam elements. Experimental test results were used to calibrate a failure criterion based on shear forces to represent the interface with weld connection between the concrete slab, the shear studs, the metal deck, and the top flanges of the beams. Alashker et al. (2010) found that using individual beam elements to model shear connectors might lead to premature failure of concrete near the connections. To alleviate this issue, each stud was represented by a cluster of four beam elements, and the strength and stiffness of each beam element are equal to a quarter of those of the stud. These beam elements are placed on a 2×2 grid with a spacing of 44 mm in both directions.

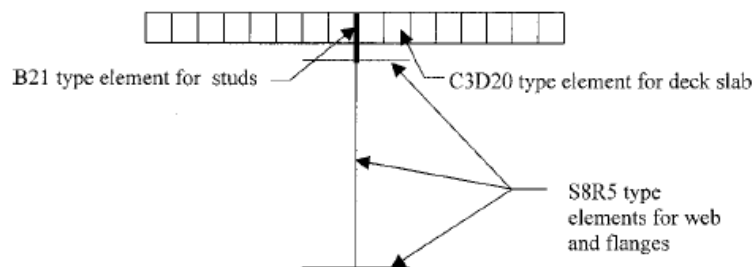


Figure 2-2 Shear stud represent by beam element in the finite element model of a composite plate girder [from (Baskar et al., 2002)]

Another widely used type of element for shear connector modeling is the solid element. This approach features a finite element model of the shear connectors with solid elements, which typically has a refined mesh and thus is computationally more expensive; however, different from the other two aforementioned types of elements, it can capture more detailed behavior of the shear studs explicitly (such as fracture) with properly defined material models (e.g., see Figure 2-3). With the commercial software for finite element modeling, Abaqus (Dassault Systèmes Simulia, 2013), 3D eight-node quadratic solid / brick elements with reduced integration (C3D8R) have been used to model shear connectors in composite structures (Ellobody and Young, 2006; Katwal et al., 2018; Katwal et al., 2020; Rana et al., 2015; Shen et al., 2020; Tahmasebinia et al., 2013). In Mirza and Uy (2010), a twenty-node quadratic solid element with reduced integration (C3D20R) was also used which was found to be capable of capturing stress concentrations.

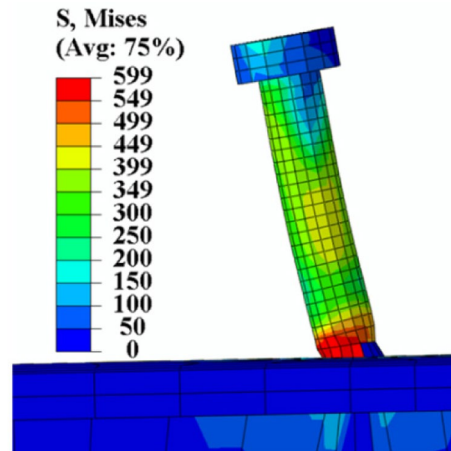


Figure 2-3 Predicted stud failure in a composite beam finite element model using solid elements for shear studs [from (Katwal et al., 2020)]

2.2.1.2. Modeling of Other Components

Different levels of modeling refinement have also been implemented in the literature for other components of the finite element models for composite beams or diaphragms, including concrete slabs, steel deck and steel beams, etc. Using solid elements for modeling the concrete slab and the steel beam and shell elements for modeling the steel deck is the most commonly adopted approach (e.g., Katwal et al., 2018; Mirza and Uy, 2010; Rana et al., 2015; Shen et al. 2020). There are also some variations to this approach, such as using solid elements to model all these components (Ellobody and Young, 2006; Tahmasebinia et al., 2013), using shell elements to model the steel beam (Sadek et al., 2008), and using beam elements to model the steel beam (Yu et al., 2010).

In addition to using 3D solid elements for the concrete slab, two-dimensional (2D) membrane/shell elements also have been used. As shown in Figure 2-4, four-node membrane elements with layered-section were used to model a ribbed reinforced concrete slab on steel sheeting. Similarly, in Galal et al. (2019), shell elements were also used to model a reinforcement concrete slab.

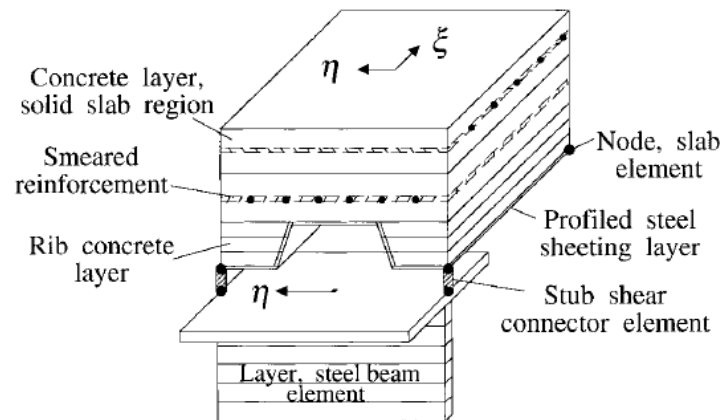


Figure 2-4 Assembly of finite element model for a composite slab using membrane elements for concrete slab [from Sebastian and McConnel (2000)]

With further simplification, one-dimensional (1D) beam elements can also be used to represent a composite beam as a whole. To achieve that, Ayoub and Filippou (2000) performed a segment equilibrium analysis and developed the formulation of an inelastic beam using uniaxial hysteretic material models, as shown in Figure 2-5. Computational efficiency was one of the advantages of using beam formulation to model the whole composite beam at the time that study was conducted; however, this may be less crucial nowadays, and it is not possible to achieve a detailed investigation on the behavior of different components using this approach.

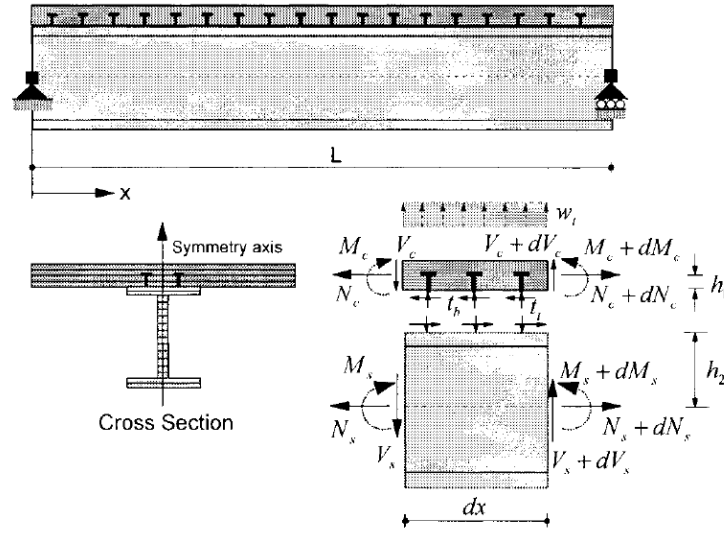


Figure 2-5 Segment equilibrium analysis for a composite beam [from (Ayoub et al., 2000)]

2.2.1.3. Material Models

Material nonlinearity is typically considered in the FE analysis of composite beams and diaphragms to capture their nonlinear structural behavior. For the concrete material, the concrete damage plasticity (CDP) model with the yield function developed by Lubliner et al. (1989) and Lee and Fenves (1998) that considers crushing in compression and cracking in tension as the two primary failure mechanisms, has been commonly used (Katwal et al., 2018; Shen et al., 2020; Tahmasebinia et al., 2013). In Ellobody and Young (2006), the Drucker-Prager yield criterion (Drucker and Prager, 1952) was adopted for the yielding part of the concrete constitutive. In Yu et al. (2010), the Johnson–Holmquist damage model suitable for simulating brittle materials subjected to large strain, high-strain rate and high pressure, as developed by Johnson and Holmquist (1994), was also used as the constitutive model for concrete.

The concrete material models mentioned above were not able to capture the behavior of crack opening and closing for concrete under stress states dominated by compression while considering the confinement effect on the strength and ductility. The triaxial constitutive model proposed by Moharrami and Koutromanos (2016) specifically resolved these issues. This model uses an elastoplastic formation, where the compression-dominated behavior is captured by a nonassociative flow rule and the tension-dominated behavior is captured by a rotating smeared-

crack model. Validation through analyses with a single element and modeling of structural components against experimental test results has demonstrated the accuracy of this concrete material model (Moharrami and Koutromanos, 2016; Moharrami and Koutromanos, 2017).

For steel materials, the kinematic hardening constitutive model has been commonly used (e.g., Yu et al., 2010). A bilinear stress-strain curve for shear studs calibrated against experimental test data was also used by Ellobody and Young (2006) to model all steel materials in their FE models of pushout test specimens. In addition, elastic-plastic constitutive relationship with strain hardening and fracture was adopted in Katwal et al. (2018), where fracture was modeled with a strain limit equal to 4.5% for profiled steel sheeting and with stress-strain model fitted to test data for shear studs.

2.2.1.4. Modeling of Contact

Definition of contact behavior plays a key role in the accuracy of FE modeling of composite beams and diaphragms. It can serve as a load path for transferring the forces between different components of the structure (such as bearing of shear connectors on the concrete slab). It is also challenging as it is one of the sources of nonlinearities and can cause convergence issues. This section summarizes some techniques for modeling contact in composite structures in the literature.

1. Compression-only Contact with Friction

Compression-only contact has been commonly used in the Sadek et al. (2008) used compression-only contact with friction and a friction coefficient equal to 0.3 to model the contact between concrete and steel. In Tahmasebinia et al. (2013), friction coefficients equal to 0.01 between the steel deck and steel beam and 0.5 between the concrete and steel deck were used. Interaction between shear studs and slab was realized using embedment technique, and the studs were also tied to the top flange of the steel beam. In Rana et al. (2015), a surface-to-surface contact was defined between the top flange of the steel beam and the profiled steel sheeting attached to the concrete slab with a friction coefficient equal to 0.25. In Katwal et al. (2018), hard contact was considered using a friction coefficient equal to 0.01 which was determined based on a sensitivity study. This contact definition was applied for all the contact surfaces, such as concrete to sheeting, concrete to shear studs, and sheeting and steel beams. Nodes on the bottom surface of studs were merged with nodes on the top flange of the steel beam to represent the weld. In Shen et al. (2020), compression-only hard contact was defined with the friction coefficient being 0.5.

2. Bond-slip Model

In Baskar et al. (2002), contact between the steel deck, the concrete slab and the studs were combined into a bilinear slip model with a shear stress limit for friction and stud shear, and tensile stress for studs. Majdi et al. (2014) used contact without friction between steel deck and concrete. A bond-slip cohesive zone material was used to model the interface between channel (shear connector) and concrete. In Rana et al. (2015), contact between the steel sheeting and the concrete slab was modeled using radial-thrust type connector elements (CONN3D2), with the radial displacement representing the interface slip and the load-slip curve was derived from pushout tests, and the thrust displacement simulating the debonding of the concrete and the steel sheeting.

3. Perfect Bond

In Sebastian and McConnel (2000), perfect bond was assumed between steel deck sheets and concrete. In Yu et al. (2010), slip was neglected at the interface between the concrete and the steel deck. In Ellobody and Young (2006), it was assumed that the behavior of the slab is barely affected by the debonding of the concrete slab and the steel sheeting, and therefore a tied contact was defined. However, based on test observation, regions of the steel sheeting near the studs could be detached from the concrete slab, and therefore the perfect bond model may not capture actual behavior after large deformations such as up to and after the peak load.

2.2.1.5. Applications

These types of FE models of composite steel-concrete structures have been used for many different purposes and some of the applications found in the literature are summarized as follows.

1. Investigating Composite Action and Resistance of Shear Connections in Composite Beams and Floors

Most of the FE analyses on composite structures were conducted to study the composite action and strength of shear connectors in composite beams. For example, Mirza and Uy (2010) performed finite element modeling of composite beams to investigate the behavior of shear connections in composite beams under axial and shear loading. In Ellobody and Young (2006), finite element analysis was conducted to study the effects of different configurations, such as geometries of the profiled steel sheeting, shear stud diameters and heights, dimensions of the concrete slab, and concrete strength, on the strength and behavior of shear connections in composite beams. In Tahmasebinia et al. (2013), a probabilistic finite element analysis using

Monte Carlo simulation was conducted to understand the effects of material uncertainties on the behavior of composite floors with concrete slab and profiled steel sheeting. Shen et al. (2020) also performed finite element analysis to study the load transfer mechanism and shear resistance of shear connections in solid concrete and composite slabs.

2. Simulation of Progressive Collapse

Another application of the finite element models is to simulate the progressive collapse of the buildings with composite elements. For example, Sadek et al. (2008) performed a finite element analysis on concrete-filled steel deck diaphragms to study their dynamic behavior after removal of the center column. Alashker et al. (2010) investigated the effects of deck thickness, steel reinforcement, and the numbers of bolts in the shear tab connection on the progressive collapse resistance of steel-concrete composite floor systems. Yu et al. (2010) analyzed the progressive collapse of steel concrete composite structures. with respect to different joint properties, concrete strength, interactions with floor slabs and retrofitting steel cables. Galal et al. (2019) conducted a finite element analysis to examine the catenary action of a steel frame with composite floor during progressive collapse.

2.2.2. Seismic Performance Assessment of Steel Buildings Using FEMA P695 Methodology

The seismic behavior and performance of a building depend on both the vertical lateral force resisting system (LFRS), such as braced frames, and the horizontal LFRS, such as the roof and floor diaphragms. During an earthquake, lateral inertial forces are transferred through the diaphragms to the vertical portions of the LFRS. A useful tool in evaluating seismic collapse performance of buildings is FEMA P695 (FEMA, 2009) which provides a methodology for assessing whether a seismic design approach provides sufficient protection against collapse. In that methodology, the collapse margin ratio (CMR) is introduced to quantify the safety margin of structures against collapse. This section reviews the computational studies that utilized the FEMA P695 methodology to assess the seismic performance of steel framed buildings through 2D and 3D analysis.

2.2.2.1. Computational Studies with 2D Frame Analysis

Historically, seismic design of buildings assumes that the vertical elements of the LFRS control the dynamics of the building (e.g. periods and mode shapes) and that the vertical LFRS is the

primary source of inelastic actions and hysteretic energy dissipation in the structure. In conjunction with this assumption, two-dimensional (2D) frame analysis has been widely used in the design and analysis of buildings.

Ariyaratana and Fahnestock (2011) performed a nonlinear dynamic analysis to examine the performance of BRBF and BRBF-SMRF (special moment resisting frame) systems using beam-column connections with and without moment resistance within the BRBF. It was found that the strength provided by the moment-resisting beam-to-column connections of the BRBF or the SMRF parallel to the BRBF is crucial to the behavior and the seismic performance of BRBFs. BRBFs with beam-to-column connections not capable of resisting moment experienced the largest peak and residual story drifts without observed connection-related failure modes. Smaller residual story drifts and potential connection-related failure modes at design earthquake (DE) and maximum consider earthquake (MCE) were observed for BRBFs with moment-resisting beam-to-column connections. Residual story drifts were found to reduce significantly with the dual system configurations, and such reduction was more obvious when non-moment-resisting beam-to-column connections were used for the BRBF.

Nonlinear static pushover analysis and incremental dynamic analysis (IDA) were conducted by Atlayan and Charney (2014) to study the behavior of typical BRBF systems and hybrid BRBF systems using different steel materials in the core of the brace under earthquake loading. Collapse of the braced frames was assumed to occur when 10% story drift ratio was achieved or failure of the braces due to low cycle fatigue occurred. Results of a case study with a business office building showed that for the standard BRBF system subjected to MCE level earthquake loading, the collapse probability is equal to 3.3%, with an adjusted collapse margin ratio (ACMR) equal to 2.63 that passes the FEMA P695 evaluation criteria. Case study with medical office building: The collapse probability at MCE level for a medical office building is 1.27% (far-field) and 2.38% (near-field) for the standard BRBF, with the ACMR value equal to 3.23 and 2.83, respectively (pass).

In Chen (2010), a computational study was performed to estimate the seismic demands for SCBFs and BRBFs with 3, 6, and 16 stories subjected to earthquake loadings at different hazard levels. The probability of brace buckling is larger than 50% in the 3-story SCBF under service-level earthquake loading. At the MCE hazard level, fracture of the braces is likely to occur in the

SCBFs. Larger story drift demands were observed in the BRBFs than the SCBF which can be attributed to the larger fundamental period of the BRBFs. However, weak story was more likely to develop in the BRBFs compared to the SCBFs. It was also found that the collapse probability of the braced frames with a short period is higher those with a longer period, especially for the SCBFs. For taller structures with longer periods, the SCBFs designed with conventional approaches are likely to exhibit weak story behavior.

In Khorami et al. (2017), incremental nonlinear dynamic analysis was performed on 3-, 5-, and 8-story BRBFs and SCBFs using 28 far-field ground motions. Three performance levels, i.e., immediate occupancy, life safety, and collapse prevention, were selected to define the limit states. For the target performance level, higher capacities were observed in the BRBFs than the SCBFs. Moreover, the taller structures were found to be more likely to achieve the performance level of collapse prevention than the lower structures.

In another study by Özkılıç et al. (2018), 24 archetype BRB frames with different number of stories (3, 6, 9), BRB yield strength, and BRB yield-to-length ratios and sizes, were analyzed using the methodology proposed in FEMA P695. In general, story drifts stayed below 5% at collapse level (ACMR).

2.2.2.2. Computational Studies with 3D Frame Analysis

Implicit assumptions made when using 2D frame analysis, are that the diaphragm system is rigid, the diaphragm dissipates no seismic energy, and the building does not experience torsion. In other words, 2D frame analysis assumes that the vertical LFRS responds independent of the rest of the building. However, real-world building structures behave as a 3D system during earthquakes, and the elastic and inelastic deformations of the diaphragm will interact with the deformations in the vertical LFRS and affect the seismic behavior and performance of the building. To date, there is little data in the literature about how 3D analysis, diaphragm modeling approaches, and bidirectional loading affect seismic collapse.

In Manie et al (2014), a computational study was conducted to evaluate the collapse performance of low-rise buildings with accidental torsion due to mass irregularities of the floor. Incremental dynamic analysis with bidirectional loading was performed on three-dimensional 3- and 6-story buildings with reinforced concrete frames and different levels of mass eccentricities in

the floors to study their nonlinear behavior under seismic loading. Collapse margin ratios for each model were determined using a modified FEMA P695 methodology. It was found that the increase of plan mass eccentricity caused significant changes of building behavior and reduction of CMR that can become unacceptable for life-safety performance level.

In another study by Leng et al. (2020), incremental dynamic analyses with the FEMA P695 far-filed ground motion set were performed using three-dimensional finite element models with various levels of modeling fidelity to simulate a two-story building with cold-formed steel framing designed to current U.S. standards, and the FEMA P695 methodology was used to evaluate its seismic performance. The low-fidelity model only including shear walls led to prediction with unsafe collapse margin ratios. The high-fidelity model that includes the cold-formed steel gravity walls and architectural sheathing showed overall performance comparable to the test results, and results from IDA indicated acceptable collapse margin ratios, which were attributed to large system overstrength. It was shown that current design of the studied archetype was safe in ignoring lateral force resistance of the gravity system and nonstructural components but a system behavior was found to be largely different from the actual.

2.3. Experimental Tests

Experimental tests have been conducted in the past few decades to investigate the shear strength and behavior of building diaphragms. Three types of component testing programs in the literature are reviewed in this section, including shear connector pushout tests, cantilever diaphragm tests, and standing seam roof tests.

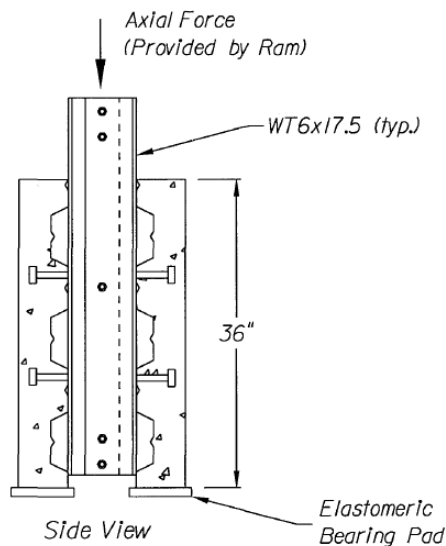
2.3.1. Shear Connector Pushout Tests

Shear connectors, such as headed shear studs, have been extensively used in composite structures to transfer forces and create composite action at the interface between a steel base and the surrounding concrete materials. There exist a wide range of experimental studies on shear connectors in steel-concrete composite components, a majority of which were focused on composite beams or diaphragms and conducted through a testing configuration commonly referred to as pushout tests. This section highlights the past studies with pushout tests for shear connectors in composite beams with concrete-filled steel deck slabs. Pushout tests that involved composite

beams without steel deck (i.e., solid slabs) are outside the scope of the work and have been reviewed in the literature (e.g., Pallarés and Hajjar, 2010).

In a composite beam with concrete-filled steel deck slab, headed shear studs are welded on the top flange of the steel beam through the flutes of the steel deck and embedded in the concrete slab that is cast on the steel deck. Historically, pushout tests have been widely used to investigate the shear strength and stiffness of the shear connectors in this type of composite beams, and many of the prediction equations for shear stud strength and stiffness in code provisions and in the literature are based on the results of pushout tests.

Figure 2-6 shows the typical setup of a pushout test. The test specimen consists of two slabs with concrete fill on the steel deck that are attached to the center beam by the shear studs welded on the flanges of the beam. The center beam is typically fabricated by attaching two WT steel shapes at the webs. Lateral bracing is provided at the surfaces of the concrete slabs. A hydraulic jack is used to apply a vertical load to the top of the center beam until failure of the specimen. Test data including applied load, displacement of the center beam, and relative slip between the center beam and the concrete slabs is collected during the test, which can then be used to obtain the shear strength and stiffness of the shear studs.



(a) Side view



(b) Test setup

Figure 2-6 Typical pushout test setup [from (Rambo-Roddenberry, 2002)]

A total of 526 pushout tests on composite beams with concrete-filled steel deck slabs were found in the literature, including Jayas and Hosain (1988), Robinson (1988), Lloyd and Wright (1990), Mottram and Johnson (1990), Androustos and Hosain (1992), Sublett (1991), Lyons (1994), Gnanasambandam (1995), Johnson and Yuan (1997), Rambo-Roddenberry (2002), Horita et al. (2012), Fan and Liu (2014), Rehman et al. (2016), Nellinger et al. (2017), Shen and Chung (2017), Yanez et al. (2017). Different parameters for the configuration of the test specimens were examined in these pushout tests to study their effects on the shear strength and stiffness of the shear studs, such as deck orientation (perpendicular vs. parallel to the center beam), stud layout and spacing, stud height and diameter, number of studs, concrete and steel deck material properties, etc. Only 24 of these pushout tests were conducted using a cyclic loading protocol. Table 2-1 summarizes the pushout tests found in the literature.

Table 2-1 Shear connector pushout tests in the literature

Reference	# of tests	Deck orientation	Loading protocol	Test parameter
Jayas and Hosain (1988)	15	Perpendicular (10) / Parallel (5)	Monotonic	Stud spacing, stud configuration
Robinson (1988)	49	Perpendicular (41) / Parallel (8)	Monotonic	Rib geometry
Lloyd and Wright (1990)	42	Perpendicular (38) / Parallel (3)	Monotonic	Slab width, concrete reinforcement, type of loading, deck profile and details
Mottram and Johnson (1990)	22	Perpendicular	Cyclic	Concrete strength and density, stud layout and position, number of studs, slab thickness and stud height
Androustos and Hosain (1992)	8	Parallel	Monotonic	Stud spacing
Sublett (1991)	34	Perpendicular	Monotonic	Type of loading, stud position, rib geometry, beam flange thickness, number of studs
Lyons (1994)	87	Perpendicular	Monotonic	Stud height, stud layout and position, deck strength, number of studs
Gnanasambandam (1995)	46	Parallel	Monotonic	Stud spacing, rib geometry
Johnson and Yuan (1997)	34	Perpendicular (16) / Parallel (18)	Monotonic	Concrete reinforcement, stud position and spacing, concrete density, rib geometry
Rambo-Roddenberry (2002)	93	Perpendicular	Monotonic	Stud diameter, concrete strength, friction at deck-beam interface, tension in stud shank
Horita et al. (2012)	26	Perpendicular	Monotonic	Stud height, stud position and spacing, number of studs, deck height
Fan and Liu (2014)	4	Perpendicular (2) / Parallel (2)	Monotonic (2) / Cyclic (2)	Deck orientation, number of studs
Rehman et al. (2016)	12	Perpendicular	Monotonic	Concrete strength and reinforcement, number of studs, shear connector type
Nellinger et al. (2017)	20	Perpendicular	Monotonic	Type of loading
Shen and Chung (2017)	10	Perpendicular	Monotonic	Stud layout, combined shear and tension in stud
Yanez et al. (2017)	24	Perpendicular	Monotonic	Stud diameter, stud position

2.3.2. Cantilever Diaphragm Tests

Research on the strength and stiffness of bare steel deck diaphragms dates back to the 1950's, when the cantilever diaphragm test method was demonstrated to yield the strength and stiffness of

the diaphragm similar to those determined from simply supported diaphragm tests at a larger scale (Nilson, 1960). Since then, a large number of experimental research programs with cantilever diaphragm tests were conducted to understand the static and cyclic behavior of steel deck diaphragms. This type of test was adopted as the standard for diaphragm testing by the American Iron and Steel Institute (AISI, 2013).

A comprehensive review of these experimental research programs was performed by O'Brien et al. (2017) and a steel deck diaphragm test database that includes a total of 753 test specimens was created. These tests can be further classified into two categories depending on whether the diaphragm specimen is topped with concrete fill or not, i.e., bare steel deck cantilever diaphragm tests and concrete-filled steel deck cantilever diaphragm tests.

Figure 2-7 shows a schematic view of the typical test setup for bare steel deck cantilever diaphragm tests idealized as a cantilever deep beam. The steel deck diaphragm is attached to the perimeter framing beams which is subjected to a shear loading on the free (north) end. The fasteners in the specimen play a crucial role in behavior of the steel deck diaphragm. Based on the fastener location, two groups of fasteners are used including the structural fasteners for deck-to-frame connections and sidelap fasteners for panel-to-panel connections (O'Brien et al., 2017). There are three types of structural fasteners commonly used in bare steel deck diaphragms that were included in the database: power actuated fasteners, self-drilling screws, and welds. Power actuated fasteners and self-drilling screws are considered as mechanical fasteners which don't require special training and have smaller variations in strength than welds that are more labor intensive and usually vary in shape and size. For sidelap connections, different types of fasteners such as welds, screws, etc. or mechanical crimping methods such as button punch were used in the test specimens consolidated in the database.

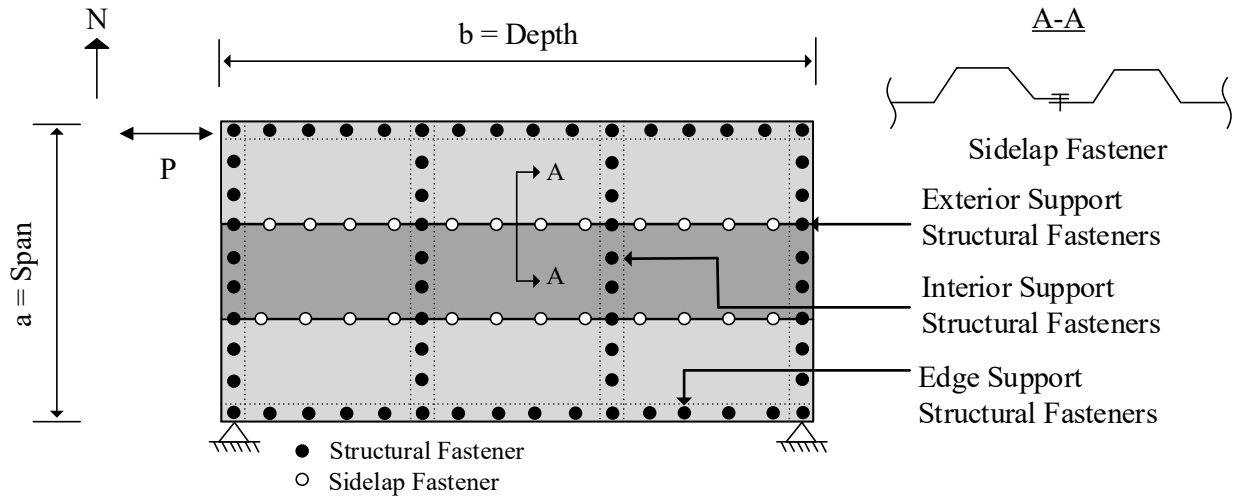


Figure 2-7 Schematic view of bare steel deck cantilever diaphragm test setup [from (O'Brien et al., 2017)]

As shown in Figure 2-8, cantilever diaphragm tests on concrete-filled steel deck diaphragms employ a similar test setup as those on bare steel deck diaphragms. Test specimens are covered with concrete fill and with or without wire mesh reinforcement, and headed shear studs are typically used for the structural fasteners that serve as part of path for the shear load transfer between the frame beams and the concrete-filled steel deck slab. As described in O'Brien et al., the research program conducted at Iowa State University (ISU) in the 1980s with 32 full scale concrete-filled steel deck diaphragm specimens provides an important source of publicly available experimental data for cantilever diaphragm tests on steel deck diaphragms with structural concrete fill (Porter and Greimann, 1980; Porter and Greimann, 1982; Neilson, 1984; Easterling, 1987; Easterling and Porter 1988; Porter and Easterling 1988; Easterling and Porter, 1994a; Easterling and Porter, 1994b; Prins, 1985). Headed shear studs and arc spot welds were used as the structural fasteners of the specimens. These specimens were subjected to a cyclic loading from hydraulic actuators and tested until failure. There were three limit states observed in the tests: diagonal tension cracking of concrete, perimeter fastener failure, and shear transfer failure (Porter and Easterling, 1988).

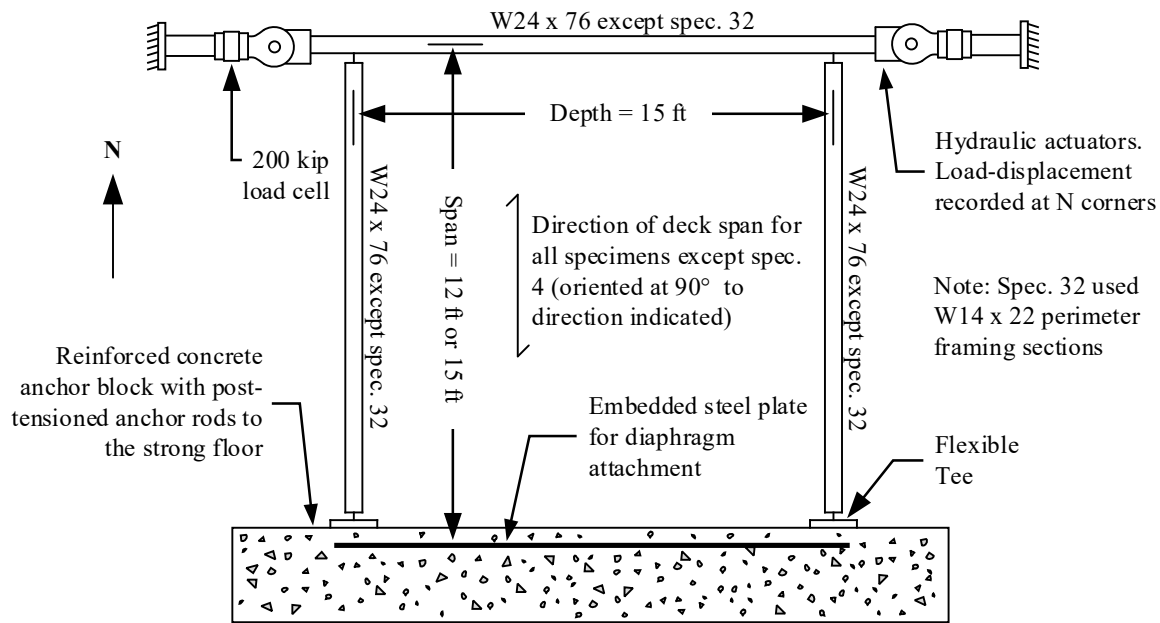


Figure 2-8 Typical test setup for concrete-filled steel deck cantilever diaphragm tests [from (O'Brien et al., 2017)]

The test specimens of the ISU experimental test program did not include configurations of concrete-filled steel deck diaphragms that are most typical in current construction. To further expand the range of cantilever diaphragm tests on concrete-filled steel deck diaphragms and to better understand the behavior of these types of diaphragms, a total of eight specimens were tested by Avelleneda-Ramirez et al. (2021) using a cyclic loading protocol. This testing program conducted at Virginia Tech is part of the SDII research efforts. These tests included varying depths of concrete cover, deck depth, perimeter stud anchor configuration, and concrete density (normal weight and light weight), as shown in Table 2-2. The test specimens were designed with target limit states including diagonal tension cracking and perimeter fastener failure which were also observed in the tests. Reinforcing steel and welded wire fabric were also included in some of the test specimens to investigate the effects of concrete reinforcements on the strength and behavior of the diaphragms. The tests results were used to evaluate the shear strength prediction of the concrete-filled steel deck diaphragm specimens using the equations proposed by O'Brien et al. (2017), which were recently adopted in AISI S310 for its next edition.

Table 2-2 Test Matrix for Cantilever Diaphragm Tests on Concrete-filled Steel Deck Diaphragms by Avelleneda-Ramirez et al. (2021)

Specimen Type	Test Specimen	Total Slab Depth (in.)	Deck Height (in.)	Concrete Type	Expected Failure Mode	Reinforcement
Reinforcing Bars	2/4.5-4-L-RS-DT	4.5	2	LW ¹	DT ³	#4 bars at 12 in. spacing
	3/6.25-4-L-RS-DT	6.25	3	LW	DT	6x6 D2.1xD2.1
	3/7.5-4-N-RS-DT	7.5	3	NW ²	DT	#3 bars at 18 in. spacing
Unreinforced	2/4-4-L-NF-DT	4	2	LW	DT	None
	3/6.25-4-L-NF-DT	6.25	3	LW	DT	None
	3/7.5-4-N-NF-DT	7.5	3	NW	DT	None
Perimeter	3/6.25-4-L-NF-P	6.25	3	LW	P ⁴	None
Fastener Failure	3/7.5-4-N-NF-P	7.5	3	NW	P	None

¹: NW = normal weight concrete

²: LW = light weight concrete

³: DT = diagonal tension cracking

⁴: P = Perimeter fastener failure

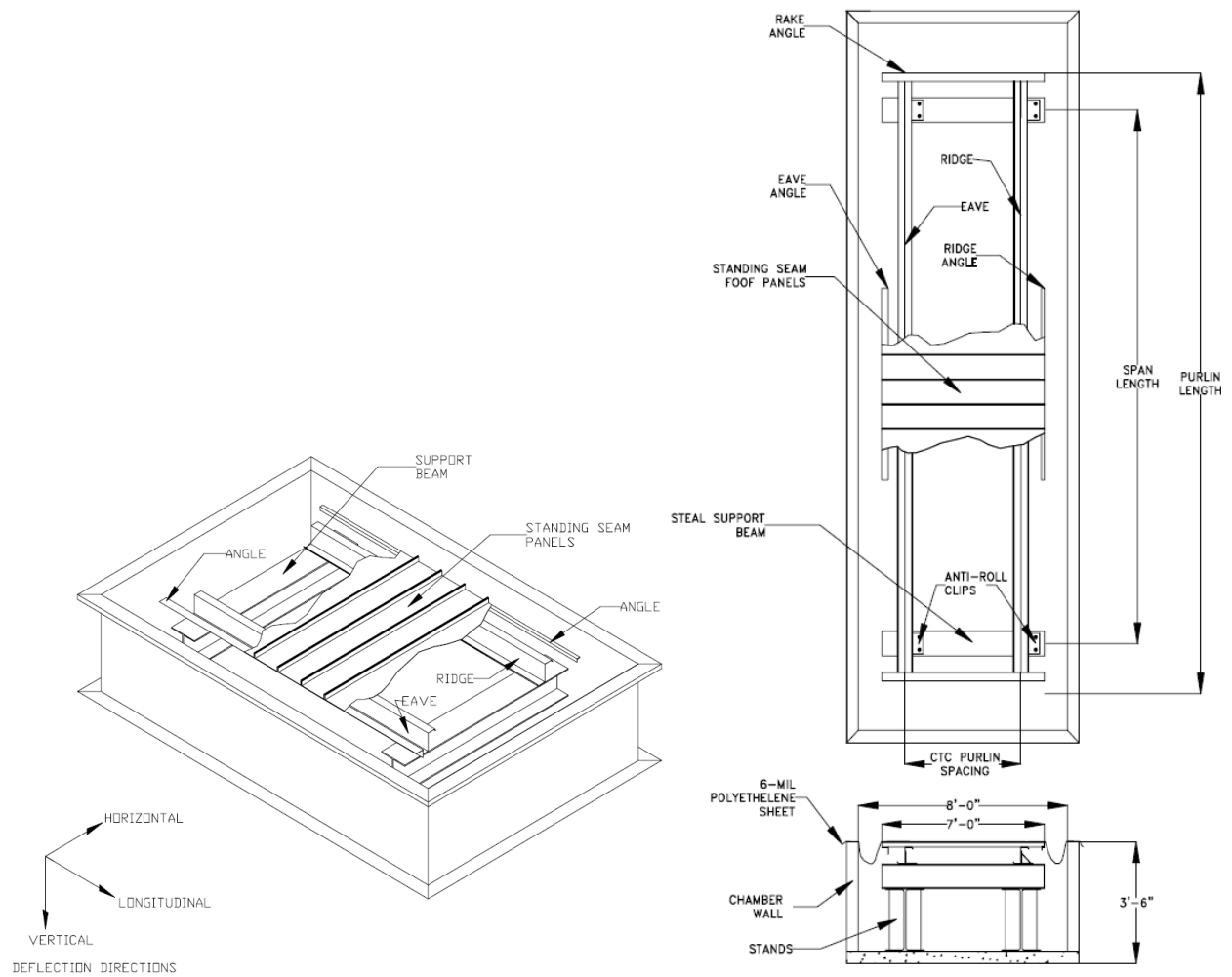
2.3.3. Standing Seam Roof Tests

Another type of experimental tests different from the cantilever diaphragm tests described above is the standing seam roof tests. This type of test can be used to study the behavior of standing seam roof diaphragms that are commonly adopted as the roof system for metal buildings.

An extensive number of gravity-loading experimental tests on standing seam roof specimens, referred as base tests (Carbollo, 1989), were conducted in the 1990s to determine the positive moment strength of a standing seam roof system (Spangler and Murray, 1989; Brooks and Murray, 1990; Earls et al., 1991; Borgsmiller et al., 1994; Bathgate and Murray, 1995; Almoney and Murray, 1998; Bryant et al., 1999; Trout 2000). Different configurations of the standing seam roof specimen were investigated with varying panel types, rib types, clip types, purlin sizes and types, span lengths, loading conditions, and bracing configurations, etc. Figure 2-9 shows the typical test setup of a base test. The standing seam roof specimen was installed on a vacuum chamber where the air was evacuated from inside using vacuum pumps to apply an increasing uniformly distributed gravity load on the specimen.

Diaphragm tests on SSR have also been conducted to determine the shear strength and stiffness of SSR assemblies. Fisher (1998) tested a manufacturer's standing seam panel roof system with four specimens to obtain the in-plane shear strength and stiffness of the test assemblies. In Piotter and Murray (2000), three cantilever diaphragm tests with the same specimen configuration were conducted to determine the shear strength and stiffness of a standing seam metal deck profile oriented perpendicular to a longitudinal ridge cap. In Sockalingam (2015), two diaphragm tests were conducted on a manufacturer's SSR system to determine the shear strength and stiffness of SSR panel construction under simulated loading conditions.

While the large-scale diaphragm tests used specimens that were more representative of a standing seam roof system, they were usually costly and labor intensive. There exists another type of small-scale clip test that was often performed to examine the strength and behavior of clips and their connections in a specimen roof diaphragm. However, many of the clip tests were conducted by the manufacturers of the standing seam roof system and due to the proprietary nature of these tests, the experimental data from clip tests is usually not publicly available. One experimental study by Seek et al. (2021) was found in the literature that used clip tests to determine the effective standoff and rotational stiffness of the panel-slip connections for a variety of clip, panel, seam and insulation configurations.



(a) Test chamber

(b) Test setup cross section and plan view

Figure 2-9 Typical test setup of standing seam roof base test [from (Trout, 2000)]

CHAPTER 3. MODELING PARAMETERS AND ACCEPTANCE CRITERIA FOR SEISMIC EVALUATION OF STEEL DECK DIAPHRAGMS

3.1. Introduction and Motivation

The current (2017) edition of ASCE 41 for the seismic evaluation and retrofit of existing buildings typically requires steel deck diaphragms to be designed (retrofit or new design) as “force-controlled”, i.e., elastic elements. This is similar to $R = 1$ (design without force reductions) in the ASCE 7 seismic design provisions, and potentially results in large economic and design inefficiencies. It also implies that a new steel deck diaphragm design per ASCE 7, evaluated per ASCE 41-17, would likely be flagged as requiring diaphragm retrofit. Recently existing data has been gathered on the cyclic performance of steel deck diaphragms and this data indicates that appreciable ductility can exist in these systems (O’Brien et al., 2017). Such ductility can be harnessed in the seismic design and evaluation of steel deck diaphragm to achieve a more economical and efficient design. However, there currently exists no guidance for what modeling parameters and acceptance criteria can be used in the seismic evaluation of steel deck diaphragms treated as deformation-controlled in the light of their ductility.

The objective of this study is to develop the acceptance criteria and modeling protocols for seismic performance-based design and evaluation of steel deck diaphragms that are aligned ASCE 41 / AISC 342. Following protocols established in ASCE 41, existing data was analyzed to determine the modeling parameters and acceptance criteria for seismic design of steel deck diaphragms supported by linear or nonlinear analysis. The method requires fitting a multi-linear model to the cyclic backbone response of available data – and parametrically characterizing the fit to the extent possible. Differences between ductility of diaphragms in buildings and diaphragms in sub-assembly tests are noted and recommendations made to handle this difference. The modeling parameters and acceptance criteria developed using the proposed method are ready for adoption by AISC 342 / ASCE 41 for bare steel deck diaphragms and concrete-filled steel deck diaphragms.

3.2. Background

A database on the past performance of steel deck diaphragms was recently compiled and analyzed (O'Brien et al. 2017). This database provides academic and proprietary test results on steel deck diaphragms. A subset of the data includes cantilever diaphragm tests with complete load-history information. Such tests were processed per Figure 3-1, and an assessment of overstrength and ductility in these systems was provided.

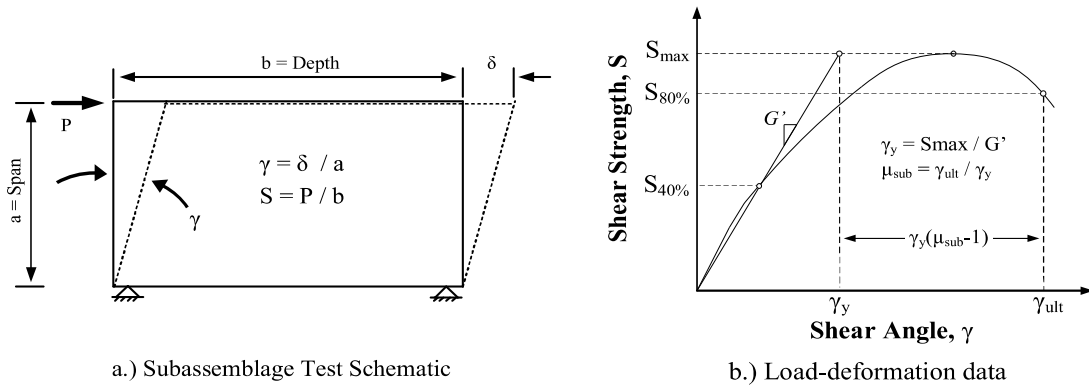


Figure 3-1 Definition of (a) shear angle (γ) and normalized load (S) in typical cantilever test and (b) stiffness (G'), strength (S_{max}), and subassembly ductility (μ_{sub}) evaluated from test results [from (O'Brien et al., 2017)]

For cyclic cantilever tests (monotonic tests have observably lower ductility) of bare steel deck diaphragms the response is summarized in O'Brien et al. (2017) and reprinted here as Table 3-1. The results indicate that ductility is available in these subassemblies, and that decks connected with mechanical systems (PAF/Screw) have the highest observed ductility.

Direct comparison between like cantilever deck specimens for PAF vs. arc spot weld structural connectors is limited, but a small subset of monotonic tests as detailed in O'Brien et al. (2017) indicates decks with PAF structural connectors and screwed sidelaps have $\mu_{sub} = 3.33$, while similar decks with arc spot weld structural connectors and screwed sidelaps (same pattern as the PAF tests) have $\mu_{sub} = 2.47$. Mechanical structural connectors are superior, but not the only path to subassembly ductility.

Table 3-1 Cyclic cantilever diaphragm test results for bare deck from O'Brien et al. (2017)

Reference	Spec. ID	G' kip/in	S _{max} kip/ft	γ_y Rad*1000	γ_{ult} Rad*1000	Subassemblage Ductility, μ_{sub}
PAF/Screw; n¹ = 21						
Martin 2002	28	12.1	0.96	6.62	13.1	1.97
Martin 2002	29	15.3	0.92	5.02	6.54	1.30
Martin 2002	31	65.3	1.81	2.31	10.1	4.37
Martin 2002	33	114	2.40	1.75	9.89	5.66
Martin 2002	34	24.7	1.16	3.90	11.9	3.04
Martin 2002	35	26.5	1.18	3.71	5.90	1.59
Essa et al. 2003	8	16.2	0.85	4.38	13.1	2.98
Essa et al. 2003	18	26.3	1.07	3.39	13.5	4.00
Yang 2003	38	23.1	1.03	3.73	13.1	3.50
Yang 2003	40	10.6	0.88	6.95	15.2	2.19
Beck 2008	S 03	72.3	3.96	4.56	14.6	3.20
Beck 2008	S 04	44.9	3.43	6.37	15.3	2.41
Beck 2008	S 05	46.1	3.48	6.29	14.2	2.26
Beck 2008	S 06	73.5	4.33	4.92	13.6	2.76
Beck 2008	S 07	59.6	2.08	2.90	11.0	3.79
Beck 2008	S 08	45.6	1.93	3.53	5.80	1.64
Beck 2013a	C 01	48.7	4.11	7.03	13.3	1.88
Beck 2013a	C 02	61.6	3.93	5.31	12.9	2.42
Beck 2013a	C 03	57.2	5.77	8.42	20.2	2.40
Beck 2013b	S 02C	58.4	3.47	4.95	12.4	2.50
Beck 2013b	S 03C	49.5	4.09	6.88	14.3	2.08
Average		45.3	2.52	4.90	12.4	2.76
Std. dev.		25.1	1.47	1.71	3.30	1.02
Weld/BP²; n¹ = 6						
Martin 2002	20	16.8	0.67	3.33	5.04	1.51
Martin 2002	21	15.2	0.93	5.11	6.51	1.27
Martin 2002	36	14.0	0.67	3.99	5.82	1.46
Essa et al. 2003	2	12.3	0.52	3.50	5.06	1.44
Yang 2003	42	11.2	0.70	5.18	12.2	2.36
Yang 2003	48	4.20	0.48	9.47	10.9	1.15
Average		12.3	0.66	5.10	7.60	1.53
Std. dev.		4.06	0.15	2.08	2.88	0.39
Weld/Screw n¹ = 2						
Essa et al. 2003	14	18.3	0.884	4.02	8.04	2.00
Essa et al. 2003	16	16.0	1.30	6.77	12.6	1.86
Weld/Weld; n¹ = 4						
Martin 2002	23	33.0	2.35	5.94	13.1	2.20
Martin 2002	24	26.7	2.27	7.09	10.0	1.40
Essa et al. 2003	12	14.0	0.71	4.25	11.1	2.62
Essa et al. 2003	13	11.2	0.89	6.58	13.1	2.00
Average		21.2	1.55	5.97	11.8	2.06
Std. dev.		8.94	0.76	1.07	1.35	0.44

¹ n = number of tests in respective group² "BP" for button punch

In addition to the database of cantilever diaphragm tests, a series of tests on the cyclic shear performance of sidelap and structural connectors for deck were recently conducted (Torabian et al. 2018a, 2018b). Specimens were tested to the AISI S905 standard and included 18 - 22 gauge B deck with sidelap connectors: screwed, top arc seam, or button punch; and structural connectors: PAF, arc spot, or arc seam. The results indicate that on the fastener level welds have remarkable strength, but little to no individual connection ductility. PAFs have the highest ductility, though

the hysteretic response is highly pinched. A useful feature of the tests is the ability to better understand the absolute deformation the connections can maintain, and what level of post-peak capacity exists at extremely high levels of deformation for any given connection.

The database assembled by O'Brien et al. (2017) also includes cantilever tests on concrete-filled steel deck diaphragms (see Table 3-2). The results are limited and more tests have been recently conducted by Avelleneda-Ramirez et al. (2021). Nonetheless some interesting observations can be made from the existing data. First and foremost, the absolute shear angles available in these systems are small, but in some realistic cases ductility – as traditionally defined – is available. Ductility, in part due to the very small γ_y , is highly variable. It should also be noted that many of the specimens which are welded only (no shear studs) use detailing that is not considered practical for construction. Considering only those specimens which fail in diagonal tension cracking (DT) or on the perimeter (P), the average ductility is $\mu_{sub} = 3.33$, regardless of whether shear studs are included or not.

Table 3-2 Cyclic cantilever diaphragm test results for filled deck from O'Brien et al. (2017)
note all tests in this table from Porter and Easterling (1988)

Spec. ID	Failure Mod ¹	G' kip/in	S _{max} kip/ft	γ_y Rad*1000	γ_{ult} Rad*1000	Subassemblage Ductility, μ_{sub}
Welded; n¹ = 14						
11	S	1770	6.34	0.30	2.25	7.53
12	DT	1710	12.1	0.59	2.30	3.92
13	DT	2020	16.8	0.69	2.23	3.23
14	S	1840	14.0	0.64	5.64	8.85
15	S/DT	1130	6.84	0.50	2.41	4.78
16	DT	920	8.01	0.73	2.39	3.29
17	S	1600	9.70	0.51	5.61	11.1
18	DT	1580	10.7	0.56	2.27	4.03
19	DT	1820	16.5	0.76	1.06	1.40
20	S/P	1300	6.21	0.40	2.25	5.65
21	S/P	870	8.16	0.78	2.56	3.27
22	DT	1650	10.5	0.53	2.09	3.95
23	S/P	1370	7.09	0.43	5.29	12.3
24	DT	1330	11.2	0.71	2.96	4.20
Average		1490	10.3	0.58	2.95	5.53
Std. dev.		338	3.44	0.14	1.39	3.08
Welds with Headed Shear Studs; n¹ = 6						
25	DT	1730	12.0	0.58	2.26	3.92
26	DT	1590	5.80	0.30	1.35	4.45
27	P	1751	6.07	0.29	1.38	4.76
28	P	1580	7.98	0.42	1.41	3.37
29	DT	1890	9.00	0.40	1.24	3.13
30	P	1530	7.69	0.42	1.37	3.27
Average		1670	8.09	0.40	1.50	3.82
Std. dev		131	2.06	0.09	0.34	0.62

¹DT = Diagonal tension cracking, P = Perimeter fastener failure, S = Shear transfer mechanism failure

Taken together existing research indicates ductility exists in steel deck diaphragm assemblies. Further, sufficient data exists to provide a more informed cyclic backbone response for these systems and thus supply m-factors and nonlinear modeling parameters for ASCE 41-based design.

3.3. Development of Subassembly m-factors

ASCE 41 uses the component demand modification factor, or m-factor, that accounts for expected ductility at certain structural performance level for the acceptance criteria in performance-based evaluation of building components with linear analysis procedures. For nonlinear analysis procedures, modeling parameters instead of m-factors should be defined. Whether determining m-factors or nonlinear modeling parameters, the core of ASCE 41 analysis is understanding the (cyclic) backbone response of the components that comprise a building. This response is generically characterized as shown in Figure 3-2.

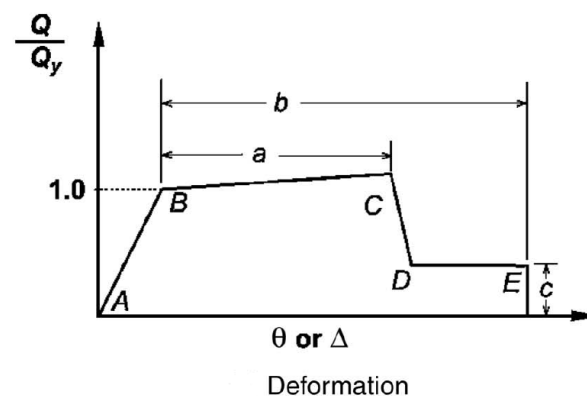


Figure 3-2 ASCE41 Multi-linear Backbone Response Curve (ASCE 41-17)

Given a measured cyclic response, judgment / analysis is still required to convert the nonlinear backbone response to the multi-linear segments of ASCE 41. ASCE 41-17 Section 7.6 provides some guidance in the process as the conversion has a definite impact on the m-factors and nonlinear modeling parameters derived.

Utilizing the data from O'Brien et al. (2017), a backbone curve was first developed from the available cyclic data. As depicted in Figure 3-3 (red line is the backbone, blue the data), two definitions were initially considered: peak-to-peak of each cycle and cyclic envelope. The cyclic envelope is closer to the available data, but causes greater complication when fitting the multi-

linear segments, and is more dependent on the cyclic loading protocol, particularly in the post-peak characterization. The peak-to-peak model was ultimately selected for the evaluation.

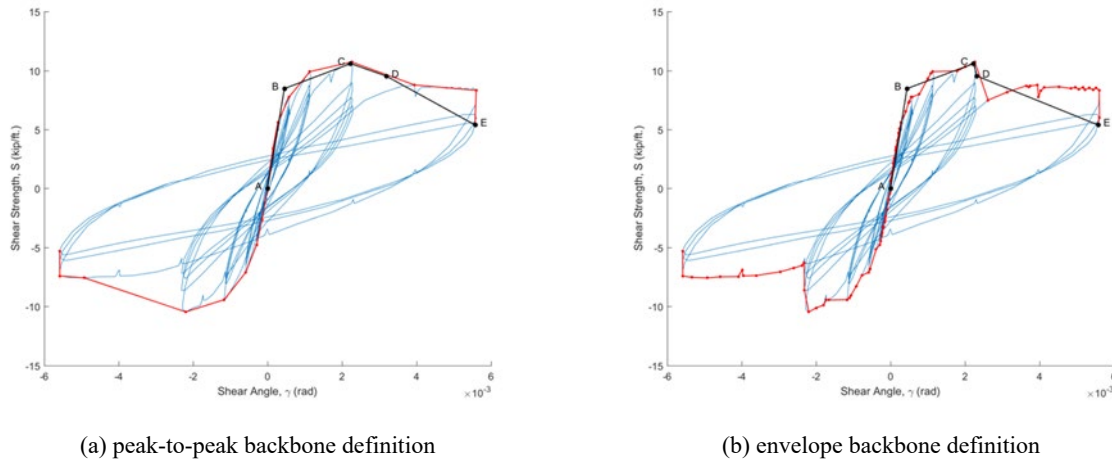


Figure 3-3 Backbone definitions (red curves) initially considered in development for example data of a concrete-filled steel deck

With the nonlinear backbone established, the ASCE 41 multi-linear segment of Figure 3-2 must be fit. The initial multi-linear fit set point C at the maximum strength point (in force and displacement) from the test, set B as along the stiffness established at 40% pre-peak but balanced so the energy under ABC is the same as the backbone of the test data, set point D at the 80% post-peak point (in force and displacement) and set point E as the last point available in the data. An example of this form of multi-linear segment fitting is shown in Figure 3-4a.

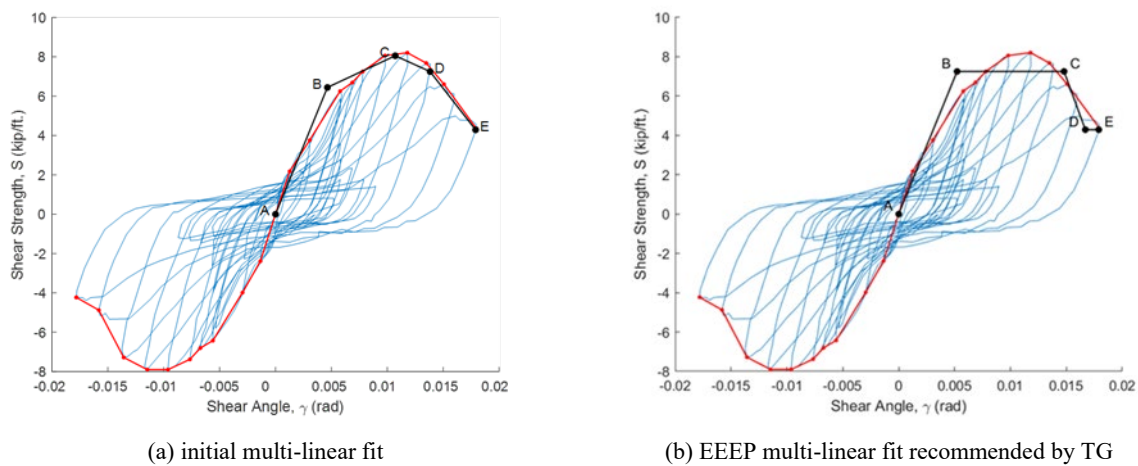


Figure 3-4 Comparison of (a) initial bounding fit developed by the team and (b) EEEP fit recommended by TG for bare steel deck diaphragm with PAF structural connector and screwed sidelaps

Based on input from the AISC 7 TG, a different multi-linear segment fit approach was also considered and ultimately utilized, which is more consistent with an Equivalent Energy Elastic Plastic (EEEP) model (but with a degraded post-peak branch). The model parameters are defined in Figure 3-5. Both pre-peak and post-peak energy is balanced. An example result of this manner of multi-linear fitting is provided in Figure 3-4b.

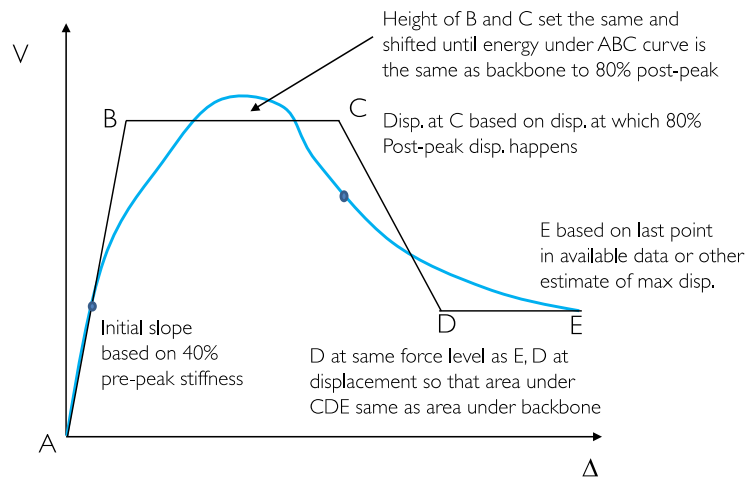
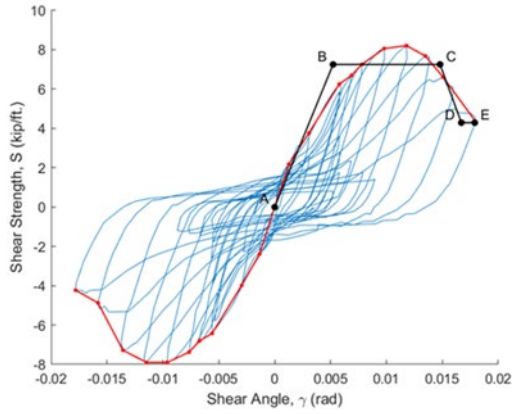
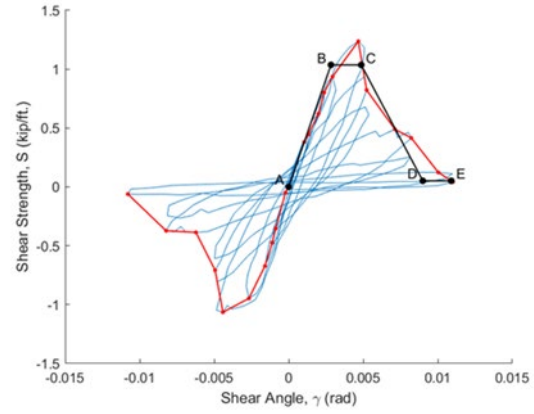


Figure 3-5 Details of the EEEP model

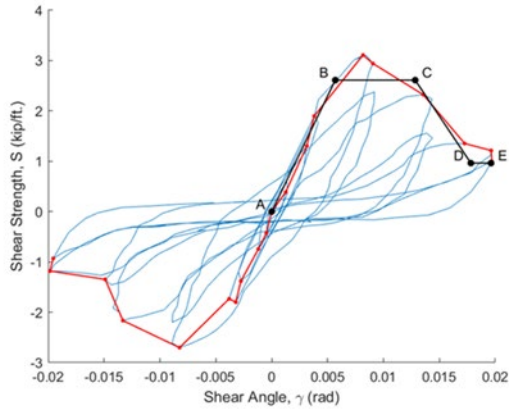
Examples of the ASCE 41 multi-linear segment fit to different bare steel deck diaphragm systems in cantilever diaphragm tests is provided in Figure 3-6. Care should be taken in comparing across systems as the axes limits are not equal. Points D and E are sensitive to the amount of post-peak data collected - in some cases the data is quite limited and particularly point E must be used with some level of caution / judgment in subsequent analyses. It is also worth noting that backbones are established for the first and third quadrant, but only the first quadrant results are depicted. Ultimately for determining m-factors and nonlinear modeling parameters, symmetry in the response is assumed and the values across the 1st and 3rd quadrant are averaged.



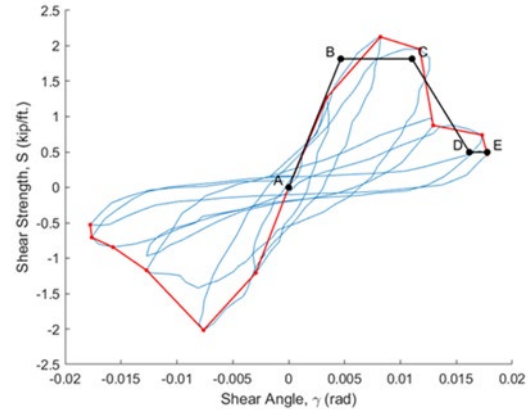
(a) PAF/screw



(b) Weld/Button Punch



(c) Weld/Screw



(d) Weld/Weld

Figure 3-6 Example EEEP fits to available data for bare steel deck diaphragms

Once the multi-linear backbone curves are established, determining the acceptance criteria for linear analysis procedures, i.e. m-factors, is relatively straightforward. ASCE 41-17 Section 7.6 establishes the basic acceptance criteria for three structural performance levels, i.e., immediate occupancy (IO), life safety (LS), and collapse prevention (CP) in linear analysis procedures as a function of the deformation at points B, C, and E. These criteria are summarized in Table 3-3.

Table 3-3 Definition of m-factors per ASCE41-17 (reproduced from Ayhan et al. 2016)

	Primary	Secondary
m_{IO}	$0.75 \times 0.75 \times 0.67 \times \Delta_C / \Delta_B$	$0.75 \times 0.67 \times \Delta_C / \Delta_B$
m_{LS}	$0.75 \times 0.75 \times \Delta_C / \Delta_B$	$0.75 \times 0.75 \times \Delta_E / \Delta_B$
m_{CP}	$0.75 \times \min \left(\Delta_C / \Delta_B, 0.75 \times \Delta_E / \Delta_B \right)$	$0.75 \times \Delta_E / \Delta_B$

Utilizing the data gathered in O'Brien et al. (2017), the raw estimates for m-factors were developed as provided in Table 3-4. It should be noted that the values were obtained directly from the analysis of test data and may not match exactly the final implementation in ASCE 41 / AISC 342; however, these values provide the basis for selecting the m-factors. As can be observed in Table 3-4 the raw m-factors are greater than 1.0 for life safety and collapse prevention cases (and typically immediate occupancy) indicating the benefit of inelastic response and ductility in these assemblies.

Table 3-4 Developed subassembly m-factors from existing test data for steel deck diaphragms
(a) primary components

Type	Load	Fastener	Count	m _{IO}		m _{LS}		m _{CP}	
				avg.	std. dev.	avg.	std. dev.	avg.	std. dev.
Bare	Mono-tonic	PAF / Screw	20	1.5	0.8	2.2	1.1	2.9	1.5
		Weld / BP	8	1.0	0.2	1.4	0.3	1.8	0.5
		Weld / Screw	9	1.2	0.3	1.8	0.4	2.1	0.6
		Weld / Weld	14	1.0	0.3	1.6	0.5	1.8	0.4
	Cyclic	PAF / Screw	21	1.0	0.3	1.5	0.4	1.9	0.5
		Weld / BP	6	0.7	0.1	1.0	0.1	1.3	0.1
		Weld / Screw	2	1.0	0.3	1.6	0.4	2.1	0.5
		Weld / Weld	4	0.9	0.3	1.3	0.4	1.6	0.7
Concrete-filled	Cyclic	Welds only	14	3.8	1.4	5.6	2.1	6.5	2.2
		Welds+studs	6	2.1	0.8	3.2	1.1	4.3	1.5

(b) secondary components

Type	Load	Fastener	Count	m _{IO}		m _{LS}		m _{CP}	
				avg.	std. dev.	avg.	std. dev.	avg.	std. dev.
Bare	Mono-tonic	PAF / Screw	20	2.0	1.0	3.2	1.8	4.3	2.4
		Weld / BP	8	1.3	0.3	2.1	0.6	2.8	0.8
		Weld / Screw	9	1.6	0.4	2.4	0.8	3.2	1.1
		Weld / Weld	14	1.4	0.4	2.0	0.6	2.7	0.8
	Cyclic	PAF / Screw	21	1.3	0.3	2.1	0.7	2.8	1.0
		Weld / BP	6	0.9	0.1	1.8	0.4	2.4	0.5
		Weld / Screw	2	1.4	0.4	2.7	1.1	3.6	1.5
		Weld / Weld	4	1.2	0.4	2.0	1.2	2.7	1.6
Concrete-filled	Cyclic	Welds only	14	5.0	1.9	7.2	2.3	9.6	3.0
		Welds+studs	6	2.9	1.0	11.1	2.7	14.8	3.6

3.4. Correction from Sub-assembly to Full Diaphragm

Direct application of the m-factors derived in the previous section implies that the test used to develop the component ductility is under the same conditions and demands as the component in the actual building. For a shear wall, with demands imparted at the floor levels, typical tests impart the same demands as the components. For a diaphragm the situation is more complex. A cantilever diaphragm test imparts uniform shear, as shown in Figure 3-7a, while an actual diaphragm sees a varying shear demand across its width, as shown in Figure 3-7b. Deformations of the diaphragm, e.g. at midspan, are less than those developed from the uniform shear case that is tested, and these reduced deformations imply a reduced ductility for the diaphragm as a system and thus reduced m-factors.

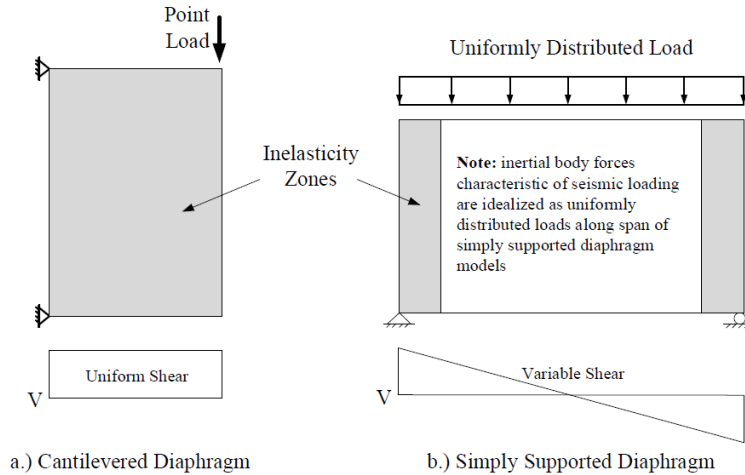


Figure 3-7 Shear distribution in (a) sub-assembly test and (b) prototypical diaphragm span in a building

O'Brien et al. (2017) developed a correction between sub-assembly cantilever diaphragm tests and simply supported diaphragms. The deformation at midspan was approximated by the summation of an inelastic end zone over diaphragm length L_p and elastic shear deformations over length L . The result of the derivation is that deformation-based ductility of the system may be expressed as:

$$\mu_{system} = 1 + 4(\mu_{sub} - 1) \frac{L_p}{L} \quad (3-1)$$

Where μ_{sub} is the ductility in the cantilever diaphragm test as reported in Table 3-1 and Table 3-2. ASCE 41 m-factors similarly use ratios of deformation to establish ductility and therefore the correction for m-factors is the same:

$$m_{system} = 1 + 4(m_{sub} - 1) \frac{L_p}{L} \quad (3-2)$$

There is limited information on the width of the plastic zone in actual diaphragms. O'Brien et al. (2017) and Schafer et al. (2018) provided preliminary examinations on L_p/L . A reasonable lowerbound estimate for steel deck diaphragms is $L_p/L \sim 0.1$. This results in system level m-factors as:

$$m_{system} = 0.6 + 0.4m_{sub} \quad (3-3)$$

Table 3-5 provides the m-factors that would be applicable at the system level assuming $L_p/L = 0.1$. It should be noted that such correction for system-level diaphragms may not be considered in the final implementation of ASCE 41 / AISC 342.

Table 3-5 System level m-factors from existing test data for steel deck diaphragms assuming $L_p/L = 0.1$

(a) primary components

Type	Load	Fastener	Count	m_{IO}		m_{LS}		m_{CP}	
				avg.	std. dev.	avg.	std. dev.	avg.	std. dev.
Bare	Mono-tonic	PAF / Screw	20	1.2	0.3	1.5	0.4	1.8	0.6
		Weld / BP	8	1.0	0.1	1.2	0.1	1.3	0.2
		Weld / Screw	9	1.1	0.1	1.3	0.2	1.4	0.2
		Weld / Weld	14	1.0	0.1	1.2	0.2	1.3	0.2
	Cyclic	PAF / Screw	21	1.0	0.1	1.2	0.2	1.4	0.2
		Weld / BP	6	0.9	0.0	1.0	0.0	1.1	0.0
		Weld / Screw	2	1.0	0.1	1.2	0.2	1.4	0.2
		Weld / Weld	4	1.0	0.1	1.1	0.2	1.2	0.3
Concrete-filled	Cyclic	Welds only	14	2.1	0.6	2.8	0.8	3.2	0.9
		Welds+studs	6	1.4	0.3	1.9	0.4	2.3	0.6

(b) secondary components

Type	Load	Fastener	Count	m_{IO}		m_{LS}		m_{CP}	
				avg.	std. dev.	avg.	std. dev.	avg.	std. dev.
Bare	Mono-tonic	PAF / Screw	20	1.4	0.4	1.9	0.7	2.3	1.0
		Weld / BP	8	1.1	0.1	1.4	0.2	1.7	0.3
		Weld / Screw	9	1.2	0.2	1.6	0.3	1.9	0.4
		Weld / Weld	14	1.2	0.2	1.4	0.2	1.7	0.3
	Cyclic	PAF / Screw	21	1.1	0.1	1.4	0.3	1.7	0.4
		Weld / BP	6	1.0	0.0	1.3	0.2	1.6	0.2
		Weld / Screw	2	1.2	0.2	1.7	0.4	2.0	0.6
		Weld / Weld	4	1.1	0.2	1.4	0.5	1.7	0.6
Concrete-filled	Cyclic	Welds only	14	2.6	0.8	3.5	0.9	4.4	1.2
		Welds+studs	6	1.8	0.4	5.0	1.1	6.5	1.4

3.5. Development of Nonlinear Modeling Parameters and Acceptance Criteria

ASCE 41 supports direct use of component backbone curves (Figure 3-2) in nonlinear static (pushover) analysis. When such analysis is performed, ASCE 41 provides the nonlinear modeling parameters that define the backbone instead of the m -factors. Typically, this is in the form of parameters, d , e , and c , as illustrated in Figure 3-8. Note that alternative plastic rotation parameters a and b are sometimes used in place of d and e , where $a = d - 1$ and $b = e - 1$. Development of the backbone curve also requires definition of the initial stiffness and the maximum strength. These are defined for each diaphragm type in relation to AISI S310-16. The initial diaphragm stiffness is selected as G' from AISI S310-16 and the diaphragm strength is the nominal strength S_n from AISI S310-16. R_Ω of Figure 3-8 refers to correction of the nominal strength to the EEEP strength as derived in Section 3.3. The values of the nonlinear modeling parameters developed from existing test data are provided in Table 3-6. It should also be noted that the values were obtained directly from the analysis of test data and may not match exactly what are ultimately suggested by ASCE 41 / AISC 342.

For steel deck diaphragms with reinforced structural concrete topping (i.e. including rebar specifically included to enhance strength or provide chord or collector performance), there is not quality test or design guidance. Therefore, it is recommended that the ASCE 41 provisions for reinforced structural concrete slabs provided in ASCE 41 Section 10.10.2.3 be utilized for nonlinear modeling and acceptance criteria in this case.

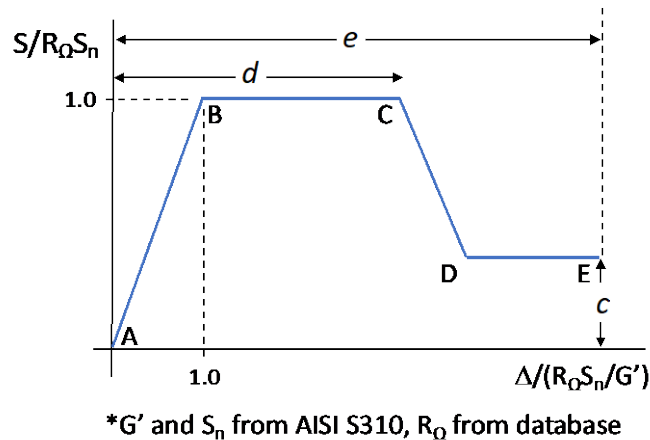


Figure 3-8. Normalized backbone curve for steel deck diaphragm

Table 3-6. Developed nonlinear modeling parameters and acceptance criteria from existing test data for steel deck diaphragms

(a) nonlinear modeling parameters

Type	Load	Fastener	Count	d		e		c^I		R_Ω	
				avg	std. dev.	avg.	std. dev.	avg.	std. dev.	avg.	std. dev.
Bare	Mono-tonic	PAF / Screw	20	4.0	2.0	5.8	3.3	0.6	0.2	1.2	0.3
		Weld / BP	8	2.6	0.5	3.7	1.1	0.5	0.2	1.0	0.2
		Weld / Screw	9	3.2	0.8	4.3	1.4	0.6	0.2	1.0	0.2
		Weld / Weld	14	2.8	0.9	3.6	1.1	0.7	0.2	0.9	0.2
	Cyclic	PAF / Screw	21	2.7	0.7	3.7	1.3	0.6	0.2	1.2	0.2
		Weld / BP	6	1.7	0.2	3.1	0.7	0.2	0.2	0.9	0.2
		Weld / Screw	2	2.8	0.7	4.8	2.0	0.3	0.1	0.9	0.1
		Weld / Weld	4	2.3	0.8	3.6	2.1	0.3	0.3	0.8	0.4
Concrete-filled	Cyclic	Welds only	14	10.0	3.8	12.8	4.0	0.4	0.1	1.8	0.4
		Welds+studs	6	5.7	2.0	19.8	4.9	0.3	0.0	1.5	0.9

(b) acceptance criteria

Type	Load	Fastener	Count	IO		LS		CP	
				avg	std. dev.	avg.	std. dev.	avg.	std. dev.
Bare	Mono-tonic	PAF / Screw	20	2.0	1.0	4.0	2.0	5.8	3.3
		Weld / BP	8	1.3	0.3	2.6	0.5	3.7	1.1
		Weld / Screw	9	1.6	0.4	3.2	0.8	4.3	1.4
		Weld / Weld	14	1.4	0.4	2.8	0.9	3.6	1.1
	Cyclic	PAF / Screw	21	1.3	0.3	2.7	0.7	3.7	1.3
		Weld / BP	6	0.9	0.1	1.7	0.2	3.1	0.7
		Weld / Screw	2	1.4	0.4	2.8	0.7	4.8	2.0
		Weld / Weld	4	1.2	0.4	2.3	0.8	3.6	2.1
Concrete-filled	Cyclic	Welds only	14	5.0	1.9	10.0	3.8	12.8	4.0
		Welds+studs	6	2.9	1.0	5.7	2.0	19.8	4.9

1. residual capacity, c , is strongly influenced by last data point available in data. Values of c provided here are not recommended for design without adjustment.

3.6. Conclusions

Seismic retrofit and rehabilitation following ASCE 41-17 essentially requires that steel deck diaphragms be designed as elastic elements. Substantial experimental evidence exists that steel

deck diaphragms have useful levels of ductility in many configurations. Existing data on steel deck diaphragms was analyzed for the purposes of determining acceptance criteria and modeling parameters for linear and nonlinear analyses consistent with the performance-based seismic design levels of ASCE 41. A series of m-factors (ductility measures) and nonlinear modeling parameters (multi-linear cyclic backbone curves) were determined for bare steel deck diaphragms and concrete-filled steel deck diaphragms.

It should be noted that neither the m-factors nor the nonlinear modeling parameters developed herein may be used directly for design. Judgment is required to incorporate the fact that the sample sizes are relatively small, that the data is limited particularly for large post-peak excursions, and that the new provisions must work and be aligned with existing related provisions. Because of such limitations, the final implementation for ASCE 41 / AISC may not match exactly the values for m-factors and nonlinear modeling parameters proposed in this study. Nevertheless, steel deck diaphragms have appreciable ductility and the developed provisions fill a gap in steel systems seismic design and allow engineers utilizing ASCE 41 / AISC 342 to consider this fact in their designs and retrofits.

CHAPTER 4. FINITE ELEMENT MODELING OF CONCRETE-FILLED STEEL DECK DIAPHRAGMS UP TO FAILURE

4.1. Goals and Scope

Concrete-filled steel deck diaphragms are highly indeterminate systems with a large number of shear stud connections between the concrete-filled steel deck and the underlying steel beams. It is almost impossible to predict the actual distribution of shear forces among shear studs in real buildings with geometric / material imperfections, built-in stresses, differential deformations, and nonuniform loading. For this reason, engineers use highly idealized models to represent the shear force distribution in studs such as assuming equal stud forces along a collector or chord. Most of these idealized models have not been validated, especially not as the diaphragm is subjected to increasing lateral loads up to failure. For the purposes of collector design, Sabelli et al. (2011) suggests that the shear may be assumed as either distributed uniformly along the depth of the diaphragm or concentrated near the vertical elements of the lateral force-resisting system to reduce collector demands. If a uniform distribution is assumed, ductile elements and connections should be used for the diaphragm; for semirigid diaphragm models, the non-uniform shear stress obtained from the analysis can be integrated over a length of five to ten feet for design purposes. Research is needed to provide support for these assumptions in practice.

The proposed computational study on the inelastic behavior of concrete-filled steel deck diaphragms aims to answer the following research questions:

- 1) How do the lateral forces accumulate in long collector lines through the shear connectors considering the nonlinear behavior of the shear connectors up to failure? How long can a collector line be and still have effective shear transfer to the diaphragm? Over what distance can the collector spread the load relatively evenly to the shear connectors before they start to fail?
- 2) How do reentrant corners and openings in the diaphragm affect the distribution of shear forces in the shear connectors in these regions? Engineers running semi-rigid diaphragm models (typically elastic diaphragms) may get stress concentrations in the diaphragm around these reentrant corners and openings. The shear stresses are often averaged along

an entire line emanating from the corner. Is this practice appropriate? There is also a concept of subdiaphragm design which is to split the diaphragm on either side of the corner into rectangular pieces, create free-body diagrams and design for the shear of these diaphragm pieces. Is this practice appropriate?

- 3) Do shear connectors have the propensity to fail sequentially (unzip until complete failure) along the edges of the reentrant corners and openings or along long collector lines?

The goals of this computational study are to understand the nonlinear behavior of diaphragm with complex configurations (e.g., long collector lines, having reentrant corners and openings, etc.), such as the distribution of shear forces transferred through the shear connectors along the collectors and near the reentrant corners and openings, and the progression of shear connector failure in these regions. This can be facilitated using finite element models that are capable of capturing the potential limit states of such diaphragms under lateral loading, including tension cracking of the concrete slab and the progressive failure of shear connectors. The framework of this study includes: 1) using cyclic push-out test data, calibrate a user-defined material model for springs using the *Pinching4* hysteretic model which was originally developed for *OpenSees* and can capture the nonlinear behavior of shear stud connections under cyclic loading, 2) validate the modeling scheme through finite element modeling of cantilever diaphragm tests, and 3) create and analyze finite element models of diaphragms with both simple and complex configurations using the validated modeling scheme to explore their nonlinear behavior subjected to lateral loading.

4.2. Representation of Shear Stud Connections Using Nonlinear Springs

Headed shear studs are typically used as shear connectors in concrete-filled steel deck diaphragms. They play a crucial role in the shear strength and behavior of the diaphragm. The accuracy of the finite element modeling of concrete-filled steel deck diaphragms heavily relies on the capability of the models to capture the shear transfer behavior of the shear stud connectors at the interface between the concrete slab, the steel deck, and the structural support (typically steel beams). One option to represent the shear studs in the finite element model of a concrete-filled steel deck diaphragm is to use solid elements (Katwal et al., 2020). However, because of the geometry and the small size of shear studs, creating a mesh that can capture the geometry of the studs and the interface between the studs and the surrounding concrete materials would require a

significantly large number of elements, especially when the represented diaphragm has a large size. To reduce the computational cost while being able to capture the complex behavior of the component in the finite element model, a more attractive alternative, and the approach adopted herein, is to represent the shear stud connectors using spring elements with a user defined material model.

The *Pinching4* material model in *OpenSees* is capable of capturing pinched load-deformation response including cyclic strength and stiffness degradation when subjected to cyclic loading. Zero-length spring elements with this type of material model that can deform in the plane of the shear loading are found to be suitable for modeling the shear stud connections in concrete-filled steel deck diaphragms. As shown in Figure 4-1, the multilinear backbone curve of the *Pinching4* material model is characterized by four force-deformations pairs on both the positive and the negative branches, which results in a total of 16 backbone curve parameters (i.e., ePd_i , ePf_i , eNd_i , eNf_i , $i = 1, 2, 3, 4$) and allows the represented force-deformation behavior to be asymmetric. The pinching behavior of the hysteretic response is simulated by the material unloading and reloading paths, which are defined by six parameters (i.e., $rDispP$, $rForceP$, $uForceP$, $rDispN$, $rForceN$, $uForceN$) and also dependent on the loading history (i.e., maximum and minimum historic deformation, d_{max} and d_{min}). Other parameters are also included in the *Pinching4* material model to simulate cyclic degradation of strength and stiffness including unloading stiffness degradation, reloading stiffness, and strength degradation.

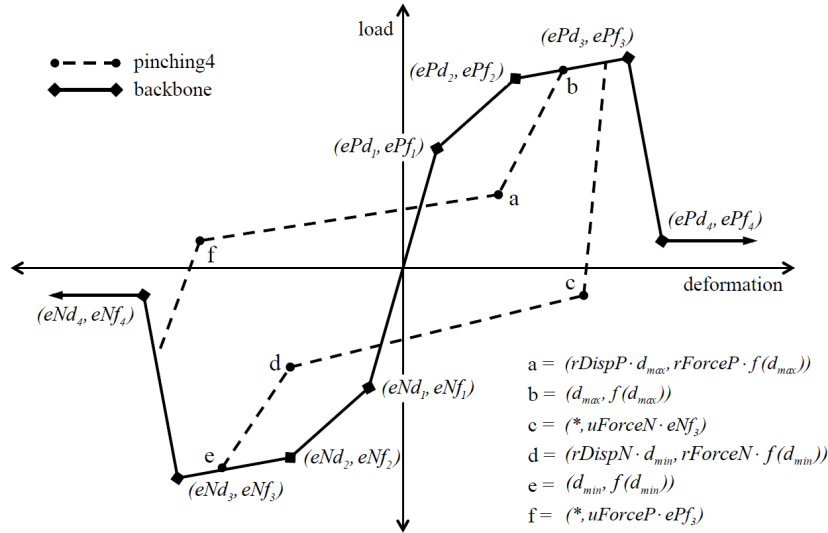


Figure 4-1 Backbone curve and pinching paths of Pinching4 material model [from (Ding, 2015)]

A general-purpose finite element software, *ABAQUS*, was used for the finite element modeling. The *Pinching4* material model has been programmed to create a user-defined spring element (Ding 2015) and was adopted in this study. Specifically, a coupled spring pair consisting of user elements (UEL) oriented in orthogonal directions with the same *Pinching4* material parameters was used to represent a shear stud connection. Figure 4-2 shows the stress-strain curve of a unit-length UEL in ABAQUS with the same *Pinching4* material parameters as SP2 given in Table 5-5 subjected to an arbitrary displacement-controlled cyclic loading protocol, and the correctness and accuracy of the user subroutine for the UEL is verified against the corresponding simulation result from OpenSees.

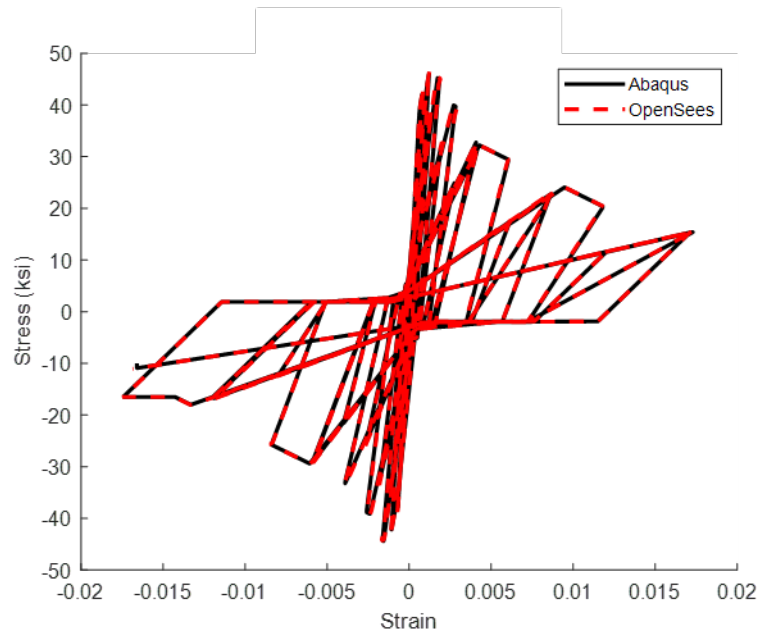


Figure 4-2 Verification of UEL with Pinching4 material in ABAQUS

The parameters of the *Pinching4* material model were calibrated against shear connector pushout tests conducted at Northeastern University (NEU) which are part of the SDII efforts (Briggs et al., 2021). Different from the traditional pushout tests reviewed in Section 2.3.1, these tests used a new testing rig as shown in Figure 4-3. The test matrix for these tests was developed with varying parameters including deck orientation, slab position, stud grouping and location, and loading protocol, resulting in a total of 16 pushout tests which are summarized in Table 4-1. The variation in design of these specimens covered common configurations of shear stud connectors used in practice, and the test results of these tests were used to calibrate the *Pinching4* material parameters for the corresponding shear stud connector configurations. Specifically, the

experimental data from the monotonic tests were used to estimate the backbone curve parameters of the Pinching4 model, whereas the other parameters for cyclic degradation of strength and stiffness were calibrated against the cyclic test results.

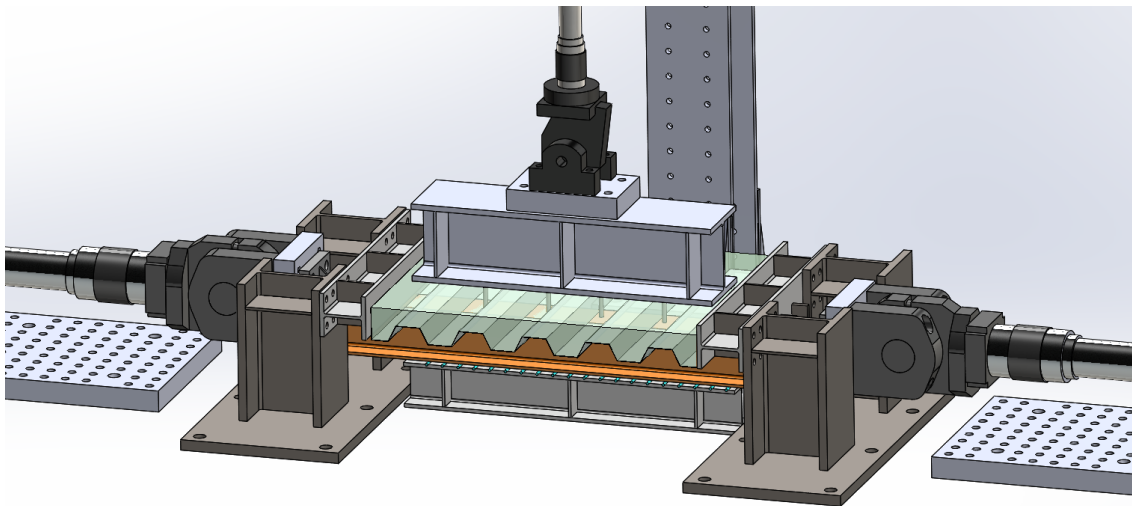
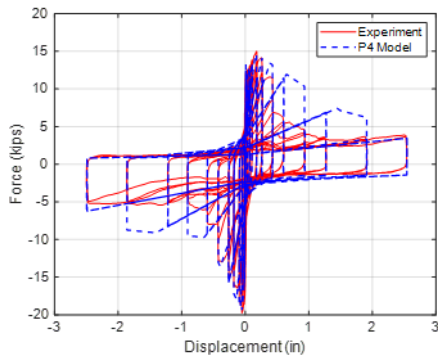


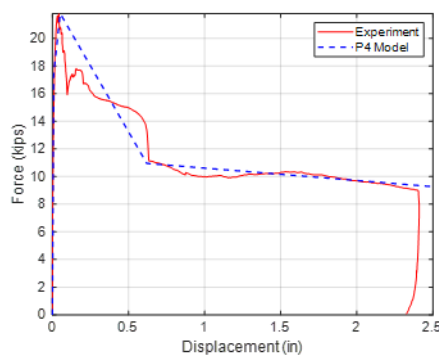
Figure 4-3 NEU shear connector pushout test setup [from (Briggs et al., 2021)]

Table 4-1 NEU Pushout Test Matrix [from (Briggs et al., 2021)]

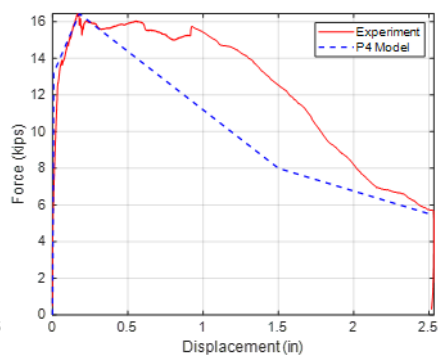
Test #	Deck Orientation	Concrete Weight	Slab Position	Stud Grouping	Stud Location	Loading
P1	Perpendicular	Lightweight	Center	One @ 12" O.C.	All strong	Monotonic
P2	Perpendicular	Lightweight	Center	One @ 12" O.C.	All weak	Monotonic
P3	Perpendicular	Lightweight	Center	One @ 12" O.C.	All Weak	Cyclic
P4	Perpendicular	Lightweight	Center	One @ 12" O.C.	50-50	Monotonic
P5	Perpendicular	Lightweight	Center	One @ 12" O.C.	50-50	Cyclic
P6	Perpendicular	Lightweight	Center	Two @ 12" O.C.	All strong	Monotonic
P7	Perpendicular	Lightweight	Center	Two @ 12" O.C.	All weak	Monotonic
P8	Perpendicular	Lightweight	Center	Two @ 12" O.C.	All Weak	Cyclic
P9	Perpendicular	Lightweight	Center	Two @ 12" O.C.	50-50	Monotonic
P10	Perpendicular	Lightweight	Center	Two @ 12" O.C.	50-50	Cyclic
P11	Parallel	Lightweight	Center	One @ 12" O.C.	Alternate sides	Monotonic
P12	Parallel	Lightweight	Center	One @ 12" O.C.	Alternate sides	Cyclic
P13	Parallel	Normal-weight	Center	One @ 12" O.C.	Alternate sides	Monotonic
P14	Parallel	Normal-weight	Center	One @ 12" O.C.	Alternate sides	Cyclic
P15	Parallel	Lightweight	Edge	One @ 12" O.C.	Alternate sides	Monotonic
P16	Parallel	Lightweight	Edge	One @ 12" O.C.	Alternate sides	Cyclic



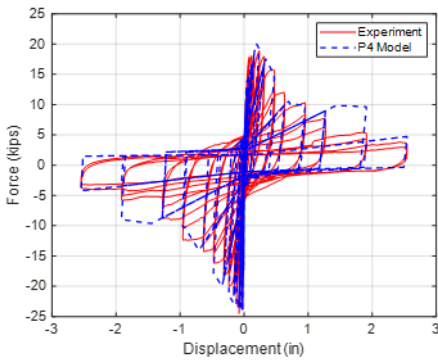
(a) P3 - one@12", all weak/strong, cyclic



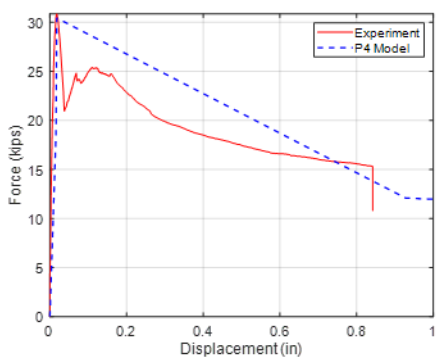
(b) P1 - one@12", all strong, monotonic



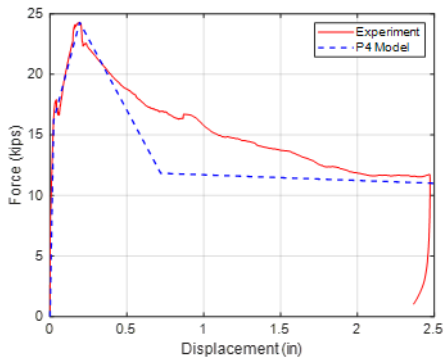
(c) P2 - one@12", all weak, monotonic



(d) P8 - two@12", all weak/strong, cyclic



(e) P6 - two@12", all strong, monotonic



(f) P7 - two@12", all weak, monotonic

Figure 4-4 Calibration of Pinching4 material model against pushout tests

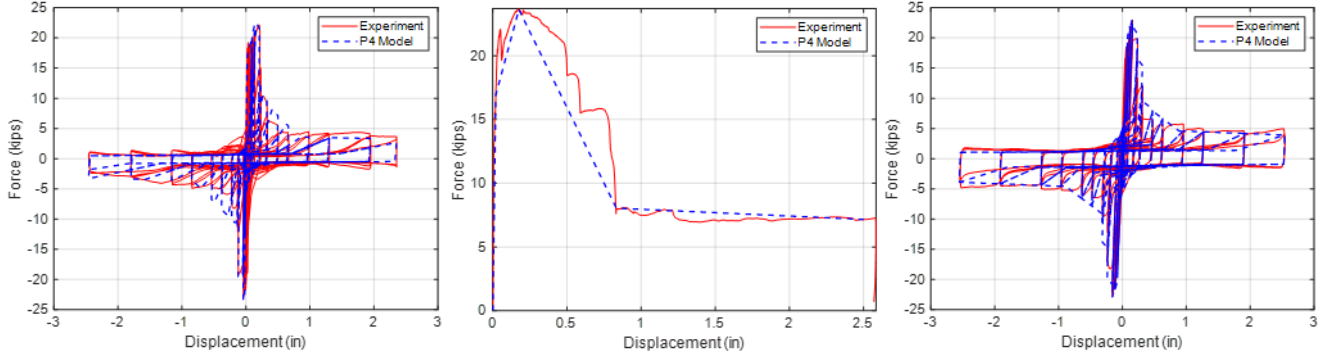


Figure 4-4 (Continued) Calibration of Pinching4 material model against pushout tests

Table 4-2 Calibrated Pinching4 Material Parameters

Test #	Parameters
P1/P2/P3 (deck perpendicular, one stud per rib)	Weak: $d_1 = 0.0074 \text{ in.}$, $f_1 = 13.2 \text{ kip}$, $d_2 = 0.178 \text{ in.}$, $f_2 = 16.4 \text{ kip}$, $d_3 = 1.5 \text{ in.}$, $f_3 = 8 \text{ kip}$, $d_4 = 2.71 \text{ in.}$, $f_4 = 5 \text{ kip}$, Strong: $d_1 = 0.00807 \text{ in.}$, $f_1 = 17.4 \text{ kip}$, $d_2 = 0.0511 \text{ in.}$, $f_2 = 21.8 \text{ kip}$, $d_3 = 0.62 \text{ in.}$, $f_3 = 10.9 \text{ kip}$, $d_4 = 2.8 \text{ in.}$, $f_4 = 9 \text{ kip}$, $r_{\Delta^+} = 0.02$, $r_{\Delta^-} = 0.02$, $r_{F^+} = 0.3$, $r_{F^-} = 0.3$, $u_{F^+} = 0.18$, $u_{F^-} = 0.18$, $gF_1 = 0.0139$, $gF_2 = 0.397$, $gF_3 = 0.0857$, $gF_4 = -0.0156$, $gF_{lim} = 0.111$, $gK_1 = 1.26$, $gK_2 = 0.936$, $gK_3 = 0.45$, $gK_4 = 0.986$, $gK_{lim} = -0.153$, $gD_1 = 0.203$, $gD_2 = 0.371$, $gD_3 = 0.0975$, $gD_4 = 0.122$, $gD_{lim} = 0.936$, $gE = 5.03$
P6/P7/P8 (deck perpendicular, two studs per rib)	Weak: $d_1 = 0.024 \text{ in.}$, $f_1 = 16.2 \text{ kip}$, $d_2 = 0.193 \text{ in.}$, $f_2 = 23.1 \text{ kip}$, $d_3 = 0.72 \text{ in.}$, $f_3 = 11.8 \text{ kip}$, $d_4 = 2.5 \text{ in.}$, $f_4 = 10 \text{ kip}$, Strong: $d_1 = 0.008 \text{ in.}$, $f_1 = 14.8 \text{ kip}$, $d_2 = 0.018 \text{ in.}$, $f_2 = 28.9 \text{ kip}$, $d_3 = 0.928 \text{ in.}$, $f_3 = 12.1 \text{ kip}$, $d_4 = 1.99 \text{ in.}$, $f_4 = 10.3 \text{ kip}$, $r_{\Delta^+} = 0.00112$, $r_{\Delta^-} = 0.00112$, $r_{F^+} = 0.181$, $r_{F^-} = 0.181$, $u_{F^+} = 0.0573$, $u_{F^-} = 0.0573$, $gF_1 = 0.000208$, $gF_2 = 0.141$, $gF_3 = 0.0157$, $gF_4 = 0$, $gF_{lim} = 0.413$, $gK_1 = 2.15$, $gK_2 = 1.09$, $gK_3 = 0.343$, $gK_4 = 1.14$, $gK_{lim} = 0.85$, $gD_1 = 0.209$, $gD_2 = 0.324$, $gD_3 = 0.111$, $gD_4 = 0.133$, $gD_{lim} = 1.05$, $gE = 4.84$
P11/P12 (deck parallel, center slab position)	$d_1 = 0.02 \text{ in.}$, $f_1 = 16.9 \text{ kip}$, $d_2 = 0.176 \text{ in.}$, $f_2 = 23.6 \text{ kip}$, $d_3 = 0.833 \text{ in.}$, $f_3 = 8.04 \text{ kip}$, $d_4 = 2.5 \text{ in.}$, $f_4 = 7.16 \text{ kip}$, $r_{\Delta^+} = 0.55$, $r_{\Delta^-} = 0.55$, $r_{F^+} = 0.25$, $r_{F^-} = 0.25$, $u_{F^+} = 0.15$, $u_{F^-} = 0.15$, $gF_1 = -0.0201$, $gF_2 = 164$, $gF_3 = -0.00328$, $gF_4 = 2.48$, $gF_{lim} = 0.553$, $gK_1 = 2.15$, $gK_2 = 1.91$, $gK_3 = 1.15$, $gK_4 = 1.02$, $gK_{lim} = 0.85$, $gD_1 = -0.0528$, $gD_2 = 0.61$, $gD_3 = 0.247$, $gD_4 = 0.434$, $gD_{lim} = 0.347$, $gE = 4.44$
P16 (deck parallel, edge slab position)	$d_1 = 0.082 \text{ in.}$, $f_1 = 20.3 \text{ kip}$, $d_2 = 0.158 \text{ in.}$, $f_2 = 23.7 \text{ kip}$, $d_3 = 0.887 \text{ in.}$, $f_3 = 10.5 \text{ kip}$, $d_4 = 3 \text{ in.}$, $f_4 = 8.5 \text{ kip}$, $r_{\Delta^+} = 0.7$, $r_{\Delta^-} = 0.7$, $r_{F^+} = 0.35$, $r_{F^-} = 0.35$, $u_{F^+} = 0.28$, $u_{F^-} = 0.25$, $gF_1 = -0.0129$, $gF_2 = 10.2$, $gF_3 = 0$, $gF_4 = 1.71$, $gF_{lim} = 0.562$, $gK_1 = 2.16$, $gK_2 = 2.46$, $gK_3 = 2.39$, $gK_4 = 3.27$, $gK_{lim} = -0.227$, $gD_1 = -0.0507$, $gD_2 = 0.605$, $gD_3 = 0.29$, $gD_4 = 0.355$, $gD_{lim} = 0$, $gE = 4.44$

4.3. Finite Element Modeling of Concrete-filled Steel Deck Cantilever Diaphragm Tests

Finite element models were developed using ABAQUS to simulate the cantilever diaphragm tests on concrete-filled steel deck diaphragms conducted at Virginia Tech (Avelleda-Ramirez et al., 2021). Among the eight test specimens as provided in Table 2-2, Specimen 3/6.25-4-L-NF-P and Specimen 3/6.25-4-L-NF-DT that exhibited a failure mode of perimeter (P) fastener failure and diagonal tension (DT) cracking, respectively, were selected for use in the validation analysis.

Figure 4-5 shows the finite element modeling scheme of a concrete-filled steel deck diaphragm assembly in a cantilever diaphragm test. The concrete-filled steel deck is represented by a solid slab consisting of eight-node hexahedral (brick) elements with reduced integration (C3D8R), with a uniform thickness of 5.073 in. which is equal to the average thickness of the concrete fill plus the transformed equivalent thickness of the steel deck. The concrete damage plasticity material model offered in ABAQUS was used to capture the inelastic behavior of the concrete. The equivalent uniaxial stress-strain curve for the constitutive model is shown in Figure 4-6, which was found to be suitable for simulating concrete material in shear connector pushout tests (Nguyen and Kim, 2009). The compressive branch consists of an elastic part up to a stress limit equal to 40% of the compressive strength of the concrete, a parabolic nonlinear portion determined based on the Eurocode 2 (European Committee for Standardization, 1992), and a linear descending part up to a strain limit equal to five times the strain value at the compressive strength. The tensile branch consists of two linear segments. For lightweight concrete of the specimens modeled in this study, the compressive and tensile strength are taken as 4 ksi and 0.2 ksi, respectively, while the elastic modulus and Poisson's ratio are assumed to be 2408 ksi and 0.2, respectively. For the concrete damage plasticity model, the material dilation angle, eccentricity, the ratio of biaxial compressive strength to uniaxial compressive strength, and the viscosity parameter were taken as 30°, 0.1, 1.16, and 0.0001 respectively. The details for the values of other variables in Figure 4-6 were adopted to be the same as Nguyen and Kim (2009). It was found in the finite element analysis that including the damage variables had a small effect on the hysteretic curve of the simulation and might cause convergence difficulty, and therefore they are not included in the proposed modeling scheme.

Perimeter steel beams were modeled using 4-node shell elements with reduced integration (S4R) and kinematic-hardening material. The yield stress of the steel material is assumed to be 50

ksi, and the modulus of elasticity and hardening modulus are set equal to 29,000 ksi and 100 ksi, respectively.

Shear studs between the composite deck and the steel beams were modeled using zero-length nonlinear springs with Pinching4 material that was calibrated against push-out tests as described in Section 4.2 to represent the nonlinear behavior of the connections. In the vicinity of shear connections, elements of the composite slab are constrained as rigid bodies to reference points located at the mass centers of the rigid bodies, which are attached to one end of the UEL representing the shear connections. To cancel the offset from the reference points to the top flange of the perimeter steel beams, rigid beam elements were created with one end attached to the UEL and the other end attached to the top of the steel beams. The UEL for the shear connections consist of an uncoupled orthogonal spring pair with zero initial length and can deform in any direction in the plane of the shear loading, which are expected to capture all the nonlinear behavior of the shear stud connections (including failure of the shear studs and the surrounding concrete material which was captured in calibration tests). Based on the configurations of the shear connections, the calibrated Pinching4 material parameters provided in Table 4-2 were used for the simulation of the two cantilever diaphragm tests. Specifically, the parameters for Specimens P3 (deck perpendicular with one stud per rib) and P8 (deck perpendicular with two studs per rib) were used to simulate strong / weak position shear connections on the steel beams parallel to the loading direction for Specimen 3/6.25-4-L-NF-P and Specimen 3/6.25-4-L-NF-DT, respectively, while the parameters for Specimen P16 (deck parallel with edge slab position) were used to simulate the shear connections on the other two steel beams for both specimens.

A general hard contact with a friction coefficient equal to 0.1 was defined for the interface between the bottom of the composite slab and the top flanges of the steel beams. Translational degrees of freedom for the nodes of the steel beams near the locations of the supports are restrained accordingly to simulate the boundary conditions. Multi-point constraints (MPC) were defined for nodes on the plate at one end of the loading beam and a displacement-controlled loading protocol was applied to the reference node of the MPC to simulate the loading from the actuator in the tests.

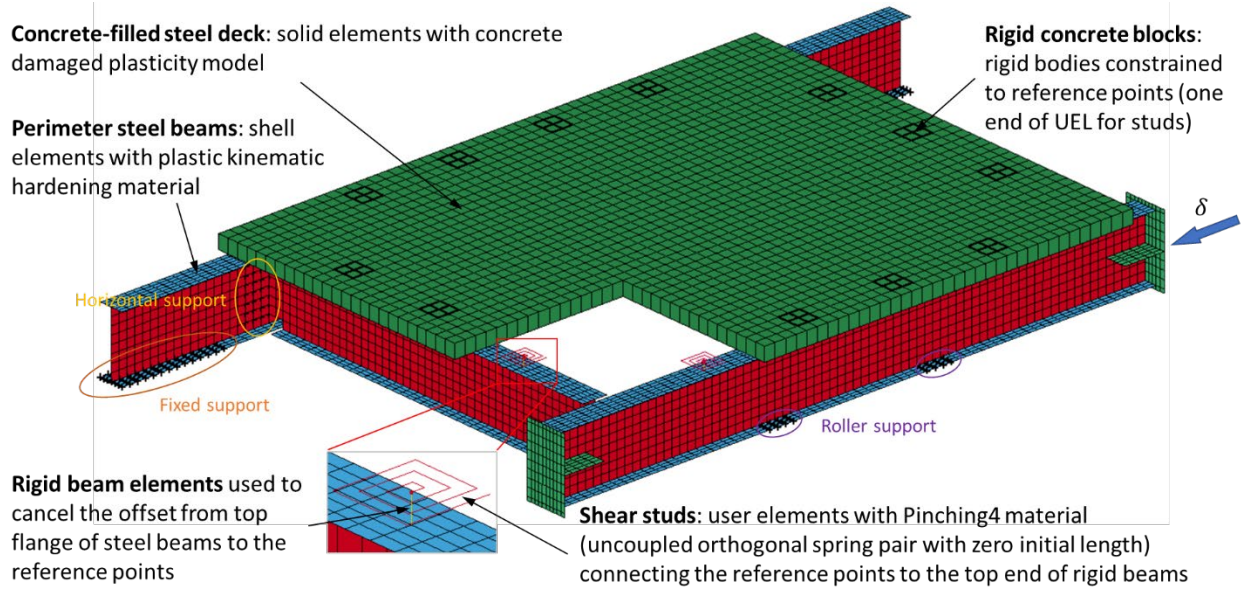


Figure 4-5 Finite element modeling scheme of a concrete-filled steel deck diaphragm assembly

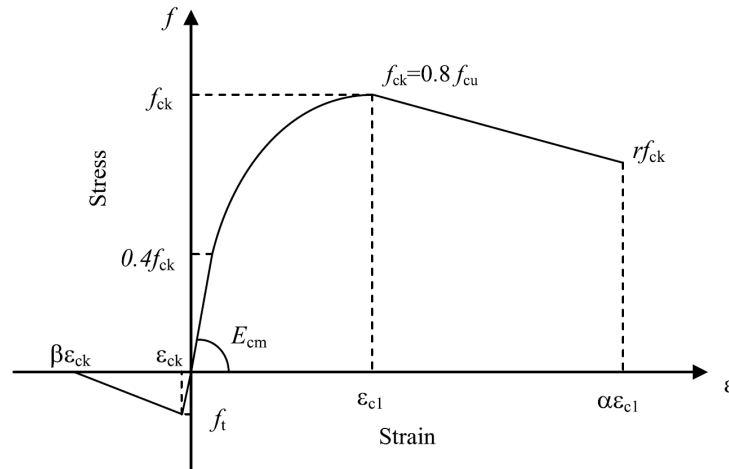


Figure 4-6 Schematic stress-strain relationship for concrete damage plasticity model [from (Nguyen and Kim, 2009)]

The proposed modeling scheme were used to study the diaphragm behavior under a monotonically increasing lateral load. Therefore, the validation analysis focused on the simulation of monotonic loading to the cantilever diaphragm specimens as opposed to the actual cyclic loading during the experimental tests. Figure 4-7 shows the load-deformation plots from the simulation and the experimental tests for the two specimens of interest. The contours of maximum principal stress in the composite slab of the two specimens are shown in Figure 4-8. For Specimen 3/6.25-4-L-NF-P, it was observed that composite slab remained relatively elastic during the analysis, with the maximum principal stress smaller than the tensile strength of the concrete, while

the stud connections on the loading beam reached a deformation larger than the deformation value of the fourth characteristic point of the Pinching4 backbone curve, indicating a failure mode of fastener failure that matches the test results. However, the strength of the specimen obtained from the simulation is 10% higher than that from the test. This is likely due to the fact that the width of the overhang of concrete on the steel beams parallel to the loading direction is smaller than that of the pushout test specimens, which makes the shear strength of the stud connections in the cantilever diaphragm test specimen smaller than those in the pushout tests that were used for the simulation. For Specimen 3/6.25-4-L-NF-DT, the peak strength obtained from the simulation is 15% higher than the experimental strength. Figure 4-8b shows a limit state of diagonal tension cracking that matches the actual failure mode of the specimen. It should be noted that the material properties of the components in the specimens, such as the ultimate tensile strength of shear studs and compressive strength of the concrete, were not specifically calibrated to match the cantilever diaphragm tests, which may also lead to the discrepancy between the predicted strength from the simulations and the actual strength obtained from the tests.

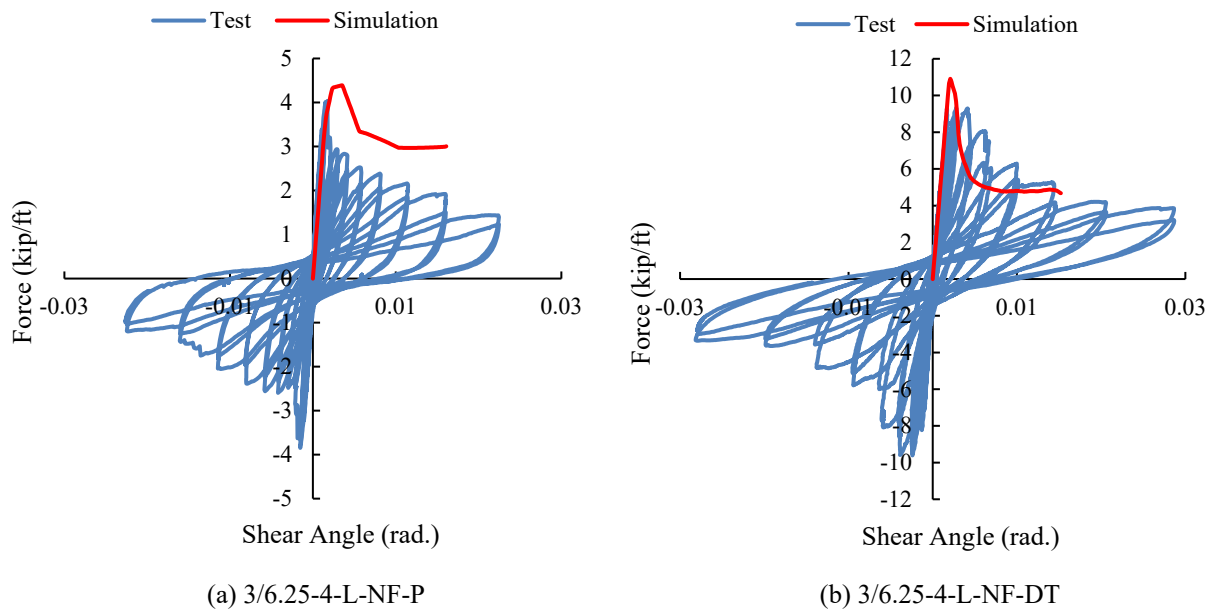


Figure 4-7 Load-deformation plots of cantilever diaphragm tests

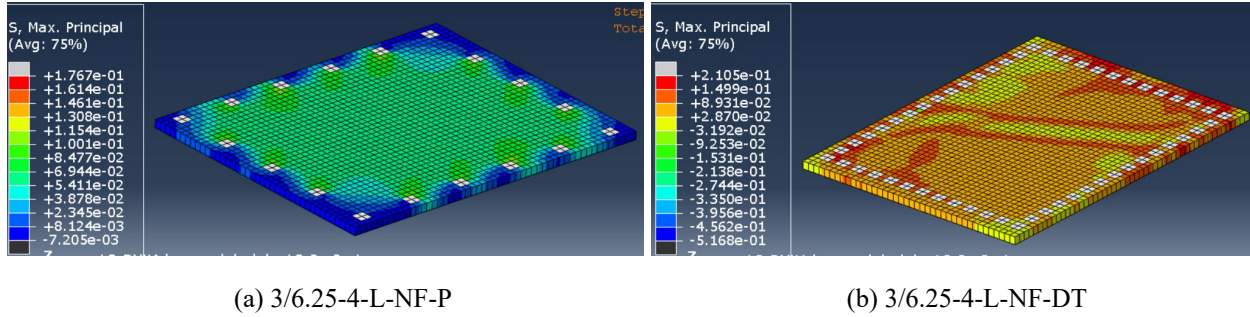


Figure 4-8 Contours of maximum principal stress in composite slab of cantilever diaphragm test specimens

4.4. Computational Study of Diaphragm Behavior Using Finite Element Models

The proposed finite element modeling scheme validated against experiment tests was used to perform a computational study to further investigate the behavior of diaphragm systems with more complex configurations. Details of the computational study is provided in this section.

4.3.1. Prototype Structures and Modeling Scheme

To answer the research questions described in Section 4.1, a series of 300 ft $\times$ 100 ft prototype diaphragm structures were developed to perform finite element analysis on diaphragm behavior using the modeling scheme validated in Section 4.3. Figure 4-9 shows a schematic layout of the prototype diaphragm structures designed to address a specific research question individually. Specifically, prototype diaphragm A in Figure 4-9a and prototype diaphragm B in Figure 4-9b were used to investigate how the lateral forces are spread over the collectors of the diaphragm. Prototype diaphragm C in Figure 4-9c and prototype diaphragm D in Figure 4-9d were used to study the distribution of shear forces and the unzipping behavior of shear stud connections near a reentrant corner and near an opening, respectively.

The diaphragm prototypes have similar configurations to the floor diaphragms of the archetype buildings described in Section 5.2 (see Figure 5-1). Steel beams attached to the diaphragms were included and the prototype structures were modeled using the validated modeling scheme. Columns and frames were not explicitly modeled, and instead fixed boundary constraints on the out-of-plane translational degree of freedom were applied to nodes on the bottom flange of the steel beams near the locations of joints where braces and columns are attached. At the locations of joints where braces are attached, an additional constraint was applied on the translational degree of freedom parallel to the braced frame direction, as indicated in Figure 4-9. Pushover analysis

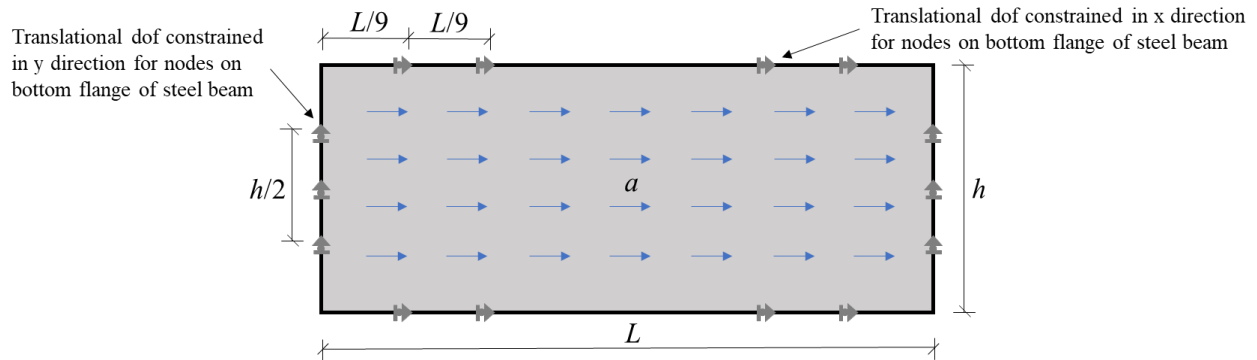
was performed with in-plane distributed lateral loads applied to the diaphragm through a monotonically increasing acceleration loading that acted on the composite slab until convergence failure occurred during the analysis. Figure 4-10 shows a schematic view of the finite element model for the prototype diaphragms. The mesh size of the composite slab is approximately 8 in., whereas the mesh size of the steel beams is approximately 6 in.

There are some important limitations associated with this modeling scheme, which are listed as follows:

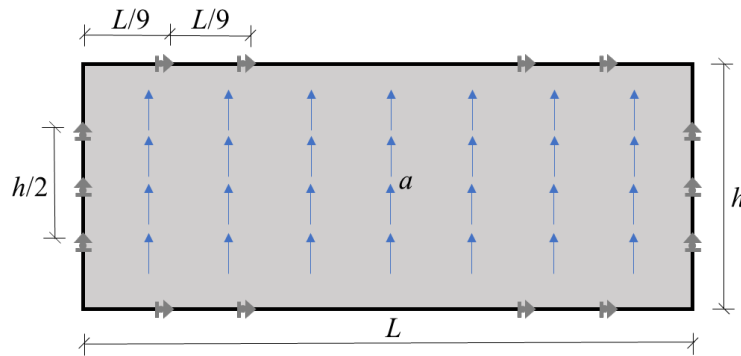
1. Beam-to-girder connections are simulated by using shared nodes on the webs of the beams and girders where they are attached. Actual connections using bolted angles or connection plates are expected to add additional axial flexibility, particularly along collector lines. This approximation of the connection may affect the distribution of shear transfer forces along collector and chord lines.
2. In buildings, the concrete floor slab is sometimes cast right up against the columns, allowing bearing of the column faces directly on the diaphragm. This can create large shear transfer strength at column locations, which is neglected in the current modeling scheme.
3. The nonlinear springs that represent the shear stud connections are not affected by damage in the surrounding concrete elements. The springs are attached to a group of concrete elements constrained as rigid bodies. In buildings, there may be an interaction between diagonal tension cracking of the concrete and local failure of shear stud connections, but that is neglected in these models.
4. The shear stud connections are represented by spring pairs with two orthogonal springs that share the same Pinching4 material model. When forces occur at an angle, the stud may exhibit greater strength and stiffness than expected in real stud connections (Ding, 2015). In addition, while the behavior of the shear stud connections is typically dominated by the behavior of the spring oriented in the direction of loading, using the same material parameters for the other spring in the perpendicular direction may not be accurate if large shear forces occur in this direction.
5. The concrete-filled steel deck is idealized as a solid concrete slab with an equivalent thickness that is modeled using solid elements. This neglects the interaction between the concrete and the steel deck which can affect the behavior of the diaphragm when large deformation occurs.

For example, change in the stiffness of the diaphragm due to the loss of shear bond between the concrete and the steel deck cannot be captured; the residual strength provided by the steel deck is also neglected.

6. Concrete cracking is not explicitly modeled. The concrete damage plasticity model without including the damage variables is used to model the concrete-filled steel deck. Element removal is not considered during the analysis. Individual cracks and the kinematic behavior of crack opening and closing may not be captured.
7. The width of the overhang of the concrete slab on the steel beams (i.e. the distance from the edge of the concrete slab to the center line of the steel beams) may be different from that of the specimens considered for calibrating the spring models of the shear connections. This can lead to slightly different strengths of shear connections as discussed in Section 4.3.

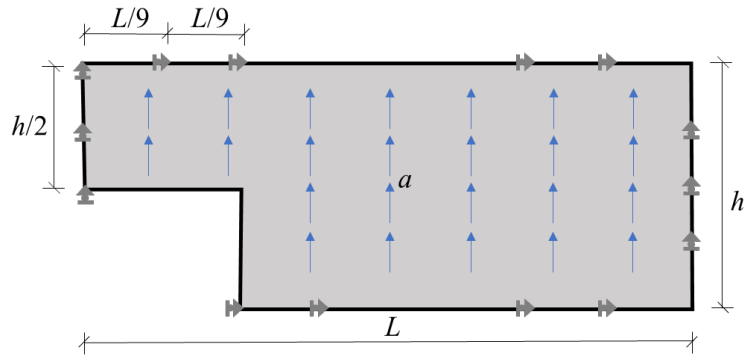


(a) Prototype diaphragm A for investigating distribution of collector forces: longitudinal lateral loading

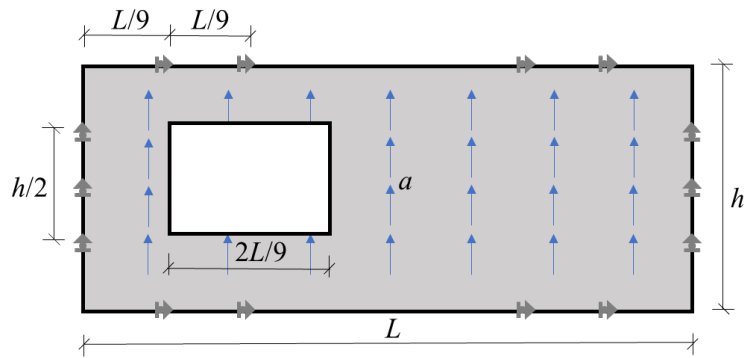


(b) Prototype diaphragm B for investigating distribution of collector forces: transverse lateral loading

Figure 4-9 Prototype diaphragm structures for finite element analysis



(c) Prototype diaphragm C for investigating distribution of shear forces near reentrant corner



(d) Prototype diaphragm D for investigating distribution of shear forces near opening

Figure 4-9 (Continued) Prototype diaphragm structures for finite element analysis

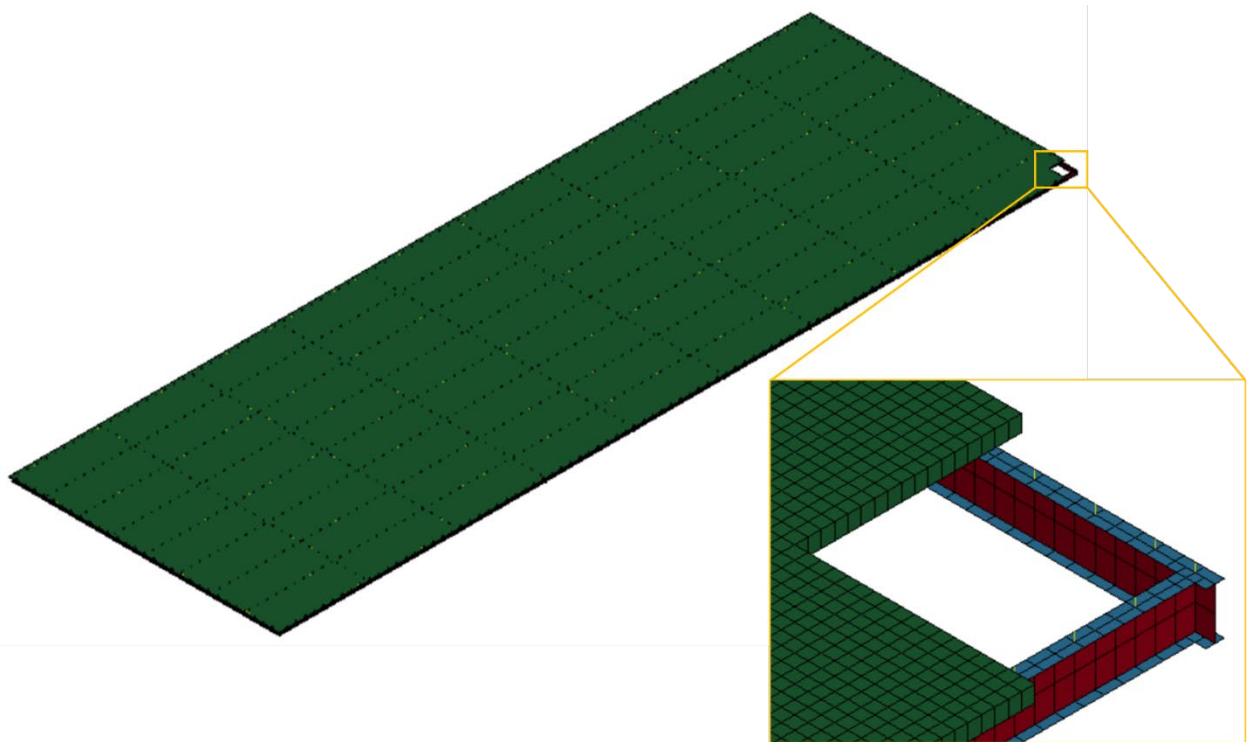
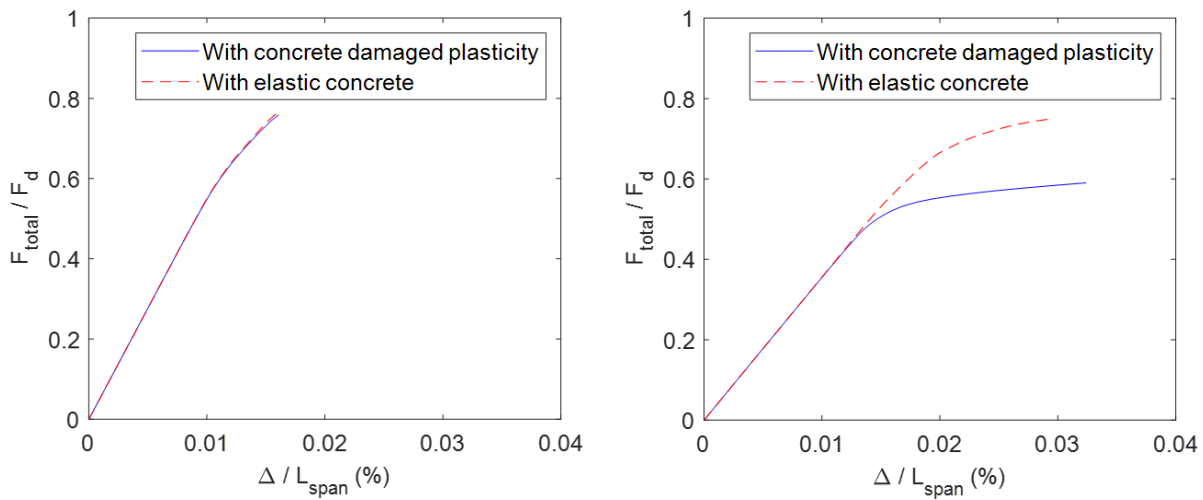


Figure 4-10 Finite element models for prototype floor diaphragms

4.3.2. Results and Discussion

This section presents the results of the finite element modeling of the prototype diaphragms. It should be noted that all figures shown in this section are plotted at or up to the last point in the analysis where convergence failed and the applied load reached the peak. To identify the potential failure modes that may cause convergence failure of the analyses and to study the progression of shear connection failure, an accompanying run of pushover analyses were also conducted on the same prototype diaphragms but with elastic material for the composite slab.

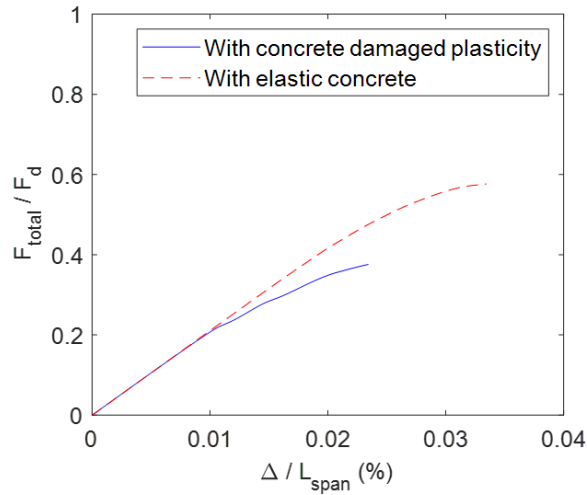
Figure 4-11 shows the force-displacement plots of prototype diaphragms. The vertical axis represents the total inertial force applied to the diaphragm, F_{total} , normalized by the design diaphragm strength with a controlling limit state of fastener failure assuming all the shear connections along the collector lines reach the peak strength together, F_d . The horizontal axis represents the displacement at the center of the diaphragm, Δ , normalized by the span of the diaphragm perpendicular to the loading, L_{span} . Because the loads were applied in a force-controlled pattern, behavior of the diaphragms after reaching the peak load was not captured. It can be observed that a larger peak load was achieved for prototype diaphragms B, C, and D with an elastic composite slab than those with the concrete damage plasticity material for the composite slab, indicating that the convergence failure in the pushover analyses of the latter is likely associated with the limit state of the composite slab in these diaphragms.



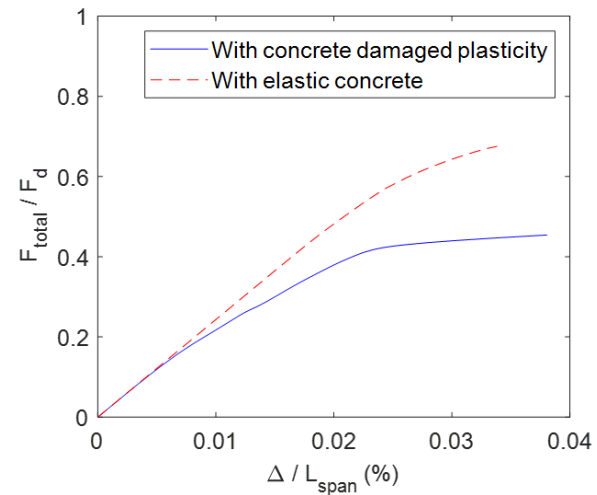
(a) Prototype diaphragm A with long collectors

(b) Prototype diaphragm B with short collectors

Figure 4-11 Force-displacement plots for prototype diaphragms



(c) Prototype diaphragm C with reentrant corner



(d) Prototype diaphragm D with opening

Figure 4-11 (Continued) Force-displacement plots for prototype diaphragms

The initial stiffness of the prototype diaphragms is obtained from the force-deformation curves and the values are given in Table 4-3, which also provides values of the peak inertial force applied to the diaphragms with concrete damage plasticity, F , the peak inertial force applied to the diaphragms with elastic composite slab, F_e , the design diaphragm strength with a controlling limit state of fastener failure assuming all the shear connections along the collector lines reach the peak strength together, F_d , and the ratio of F_e to F_d . Comparing the results of prototype diaphragms B, C and D leads to a conclusion that the existence of reentrant corner or opening can substantially reduce the stiffness (up to 41%) and shear strength (up to 36%) of the diaphragm. The ratio of F_e to F_d ranges from 76% to 86% for all the prototype diaphragms, with an average ratio equal to 80%, which implies that the design diaphragm strength considering the peak strength of all the shear connections along the collector lines is somewhat unconservative. It should be noted that some limitations of the proposed modeling scheme as described in Section 4.3.1 may affect the ratio of F_e to F_d in real diaphragm structures. For instance, neglecting the bearing of the concrete slab on the columns may provide additional shear transfer strength along the collector. Conversely, neglecting the flexibility of beam-to-column connections may overpredict the rigidity of the framing elements and shorten the distance over which shear forces are effectively transferred along the collectors.

Table 4-3 Initial stiffness and peak forces of prototype diaphragms

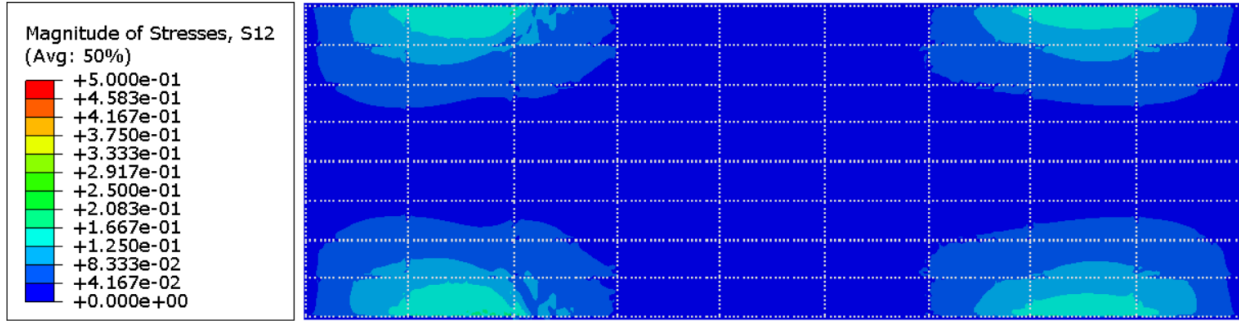
Prototype Diaphragm	Initial Stiffness (kip/in.)	F^1 (kip)	F_e^2 (kip)	F_d^3 (kip)	$\frac{F_e}{F_d}$
A (with long collectors)	17820	2930	2954	3862	76.5%
B (with short collectors)	3819	2280	2901	3457	83.9%
C (with reentrant corner)	2259	1451	2227	2605	85.5%
D (with opening)	2610	1754	2609	3457	75.5%

¹: F = peak inertial force applied to the diaphragms with elastic composite slab

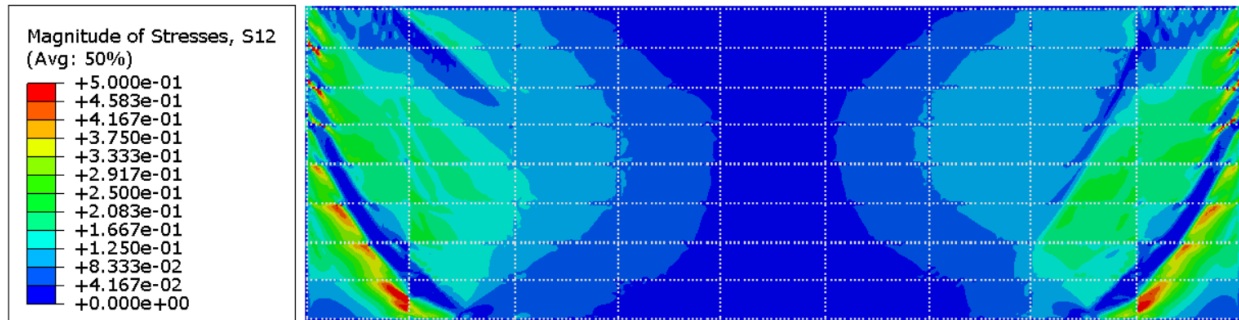
²: F_e = peak inertial force applied to the diaphragms with elastic composite slab

³: F_d = design diaphragm strength with a controlling limit state of fastener failure assuming all the shear connections along the collector lines reach the peak strength together

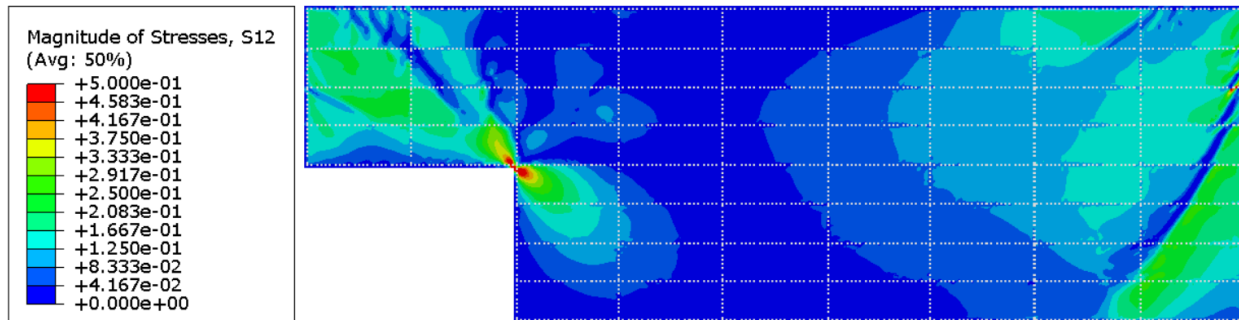
Figure 4-12 shows the distribution of shear stress in the composite slab of the prototype diaphragms with concrete damage plasticity. It can be observed that for diaphragms without geometric irregularities, the magnitude of shear stress is much larger near the vertical lateral force resisting system (i.e., braced frames) than elsewhere in the diaphragm. For prototype diaphragm A with long collectors, the shear stress of the diaphragm mainly concentrates near the braced frames, whereas for prototype diaphragm B with relatively short collectors, the shear stress is distributed more uniformly along the entire length of the collectors. For the prototype diaphragm with a reentrant corner or opening, stress concentration is observed at the corners of such geometric irregularities, indicating a potential of localized concrete cracking / crushing in these regions. This implies the need for reinforcing at the corner in design practice. It can also be observed from Table 4-3 that prototype diaphragm C reached a peak force that is 64% of prototype diaphragm B. However, with the assumption in design practice that the diaphragm shear strength is proportional to the depth of the diaphragm along the shear loading, one might think that prototype diaphragms C should reach a shear strength equal to 75% of B since it only has half the diaphragm depth on one side (same depth on the other), which is larger than the value observed from the analysis. This also indicates the unconservative nature of such an assumption.



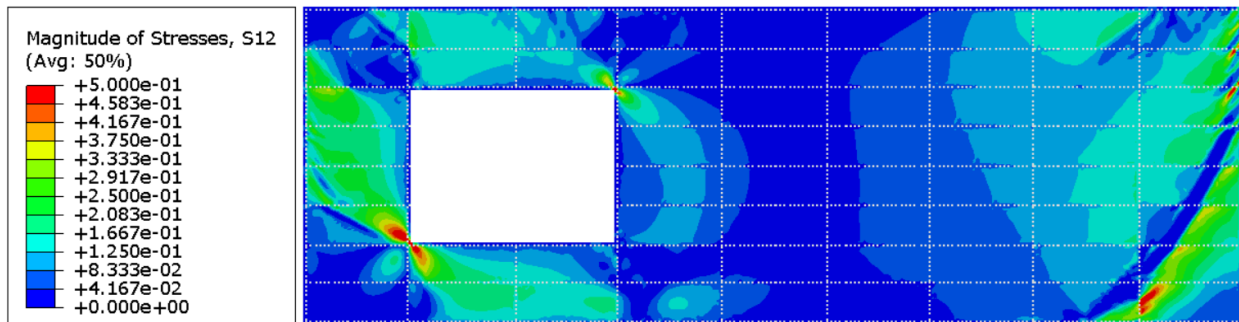
(a) Prototype diaphragm A with long collectors



(b) Prototype diaphragm B with short collectors



(c) Prototype diaphragm C with reentrant corner



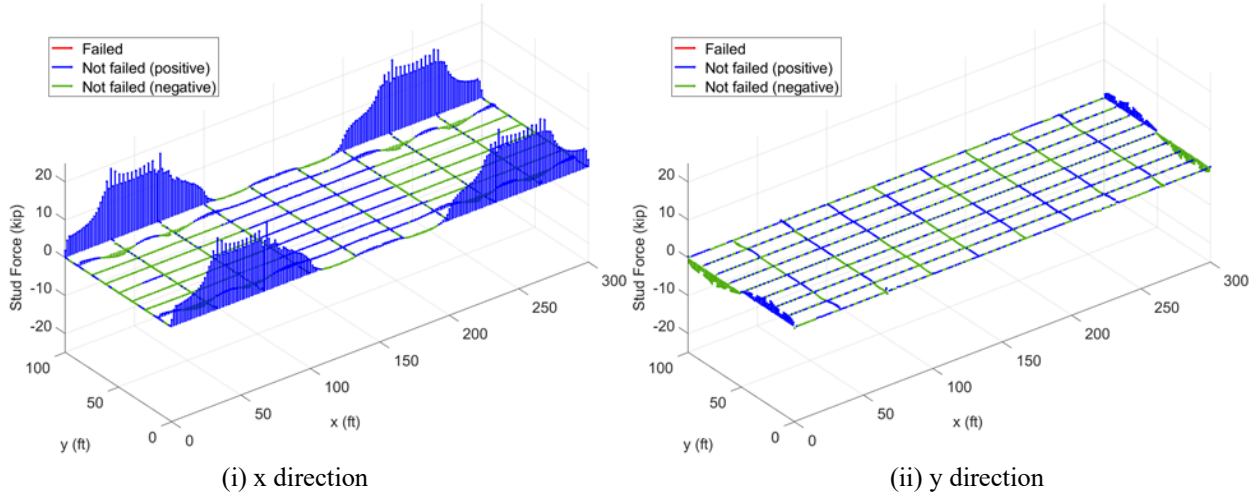
(d) Prototype diaphragm D with opening

Figure 4-12 Contour of shear stress magnitude in the composite slab of prototype diaphragms with concrete damage plasticity

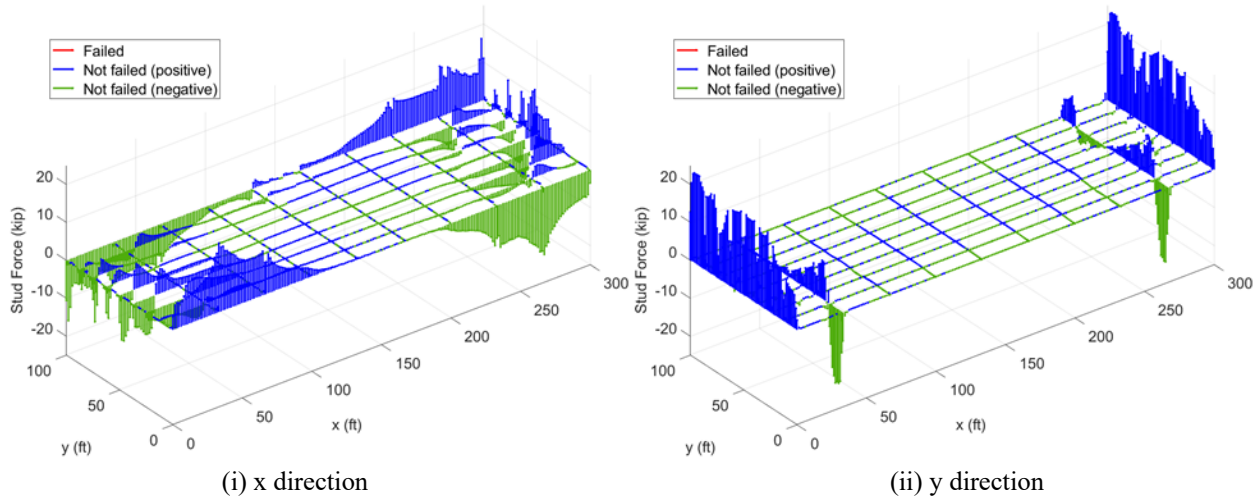
Figure 4-13 shows the distribution of shear connection forces in the x and y directions for the prototype diaphragms with concrete damage plasticity at the end of the analysis. For the purposes of plotting, shear connections are assumed to fail when the UEL deformation in the loading direction exceeds the deformation at 80% post-peak strength of the Pinching4 backbone curve. For prototype diaphragm A, loads are applied along the long dimension of the diaphragm (x direction), and the shear forces in this direction are transferred through the studs primarily near the braced frames, with negligible stud forces in the y direction. The jagged shapes of the stud force distribution in the x direction near the braced frames are due to the different positions of the studs (strong or weak).

For prototype diaphragm B where loads act in the y direction, shear forces are spread out along the collectors with shorter length. It can be observed from Figure 4-13(b)(i) that a significant amount of shear forces in the perpendicular x direction exist in the shear connections, which can be attributed to the long span of the diaphragm that results in a larger force in the chords. The direction / sign of shear connection forces along the chords follows a pattern similar to a simply supported beam as depicted in Figure 4-14. Different from prototype diaphragm A, the shear connections along the collectors in prototype diaphragm B are all in parallel position; however, a jagged shape of shear force distribution is observed along these collectors as shown in Figure 4-13-b-ii. This is because the spacing of the beams in the x direction is relatively small, and a portion of the lateral forces are transferred through the shear connections on these intersecting beams near the beam-collector joints, which leads to a decreased shear force in the shear connections at the locations of these joints.

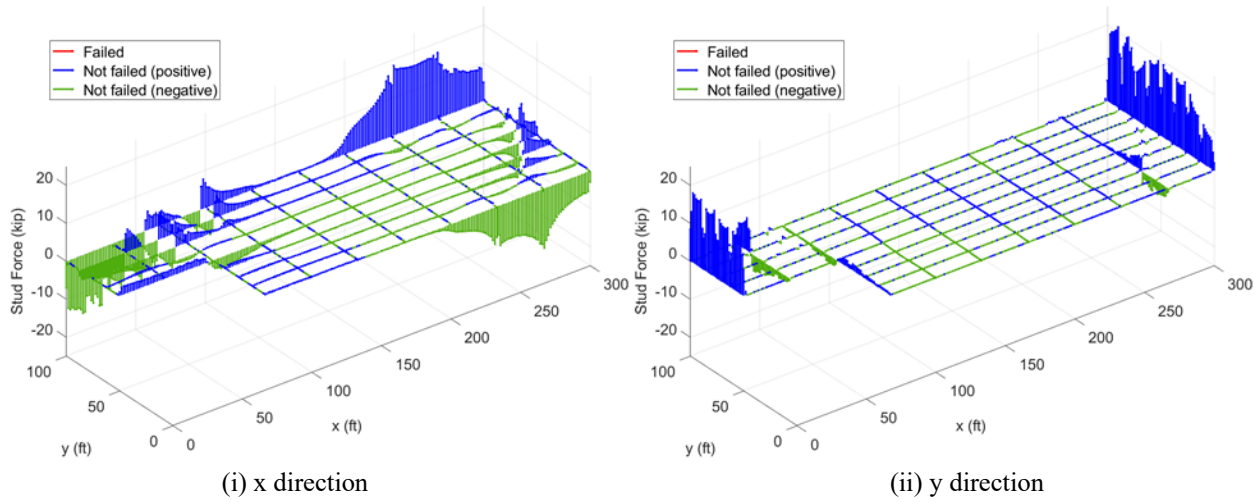
For the prototype diaphragm with a reentrant corner or an opening, a similar pattern to that of prototype diaphragm B is observed for the distribution of shear connection forces, i.e., the shear forces are transferred through the entire length of the collectors along the short dimension of the diaphragm. It should be noted that forces in the shear connections near the edge of the reentrant corner and the opening are significantly smaller than those on the collectors. This indicates that the shear loads near the edge of the reentrant corner and the opening are primarily carried by the composite slab and barely transferred to the framing elements through the shear connections. Considering the observation of shear stress localization in the composite slab in Figure 4-12, special detailing should be provided in these regions to control crack sizes and limit damage.



(a) Prototype diaphragm A with long collectors



(b) Prototype diaphragm B with short collectors



(c) Prototype diaphragm C with reentrant corner

Figure 4-13 Distribution of shear connection forces for prototype diaphragms with concrete damage plasticity

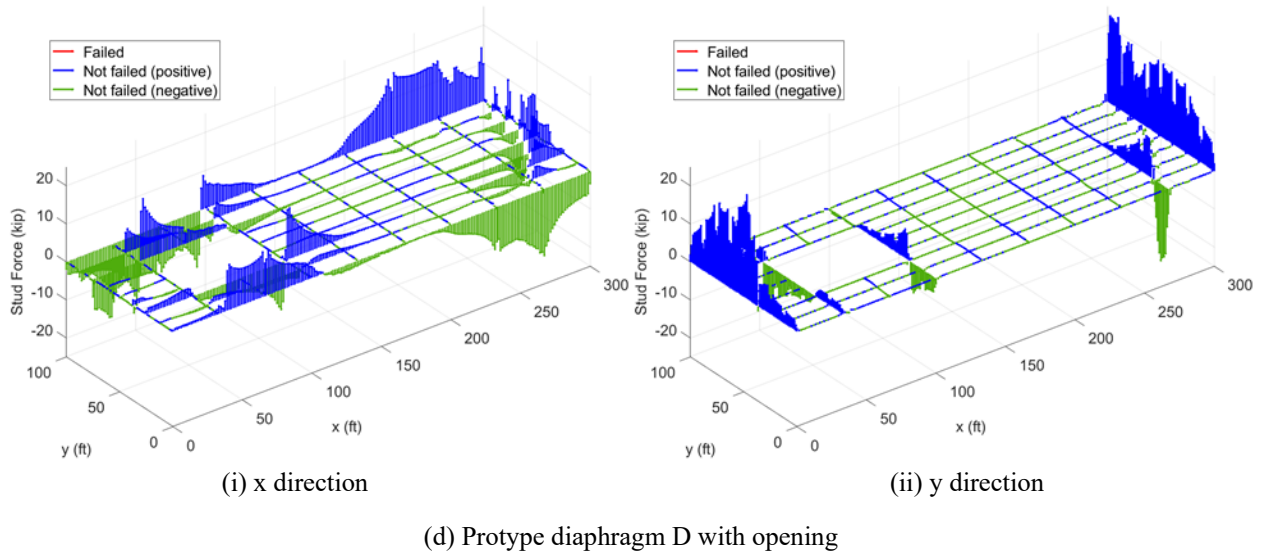


Figure 4-13 (Continued) Distribution of shear connection forces for prototype diaphragms with concrete damage plasticity

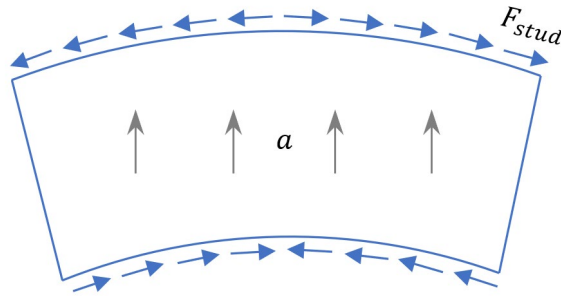


Figure 4-14 Free-body diagram of prototype diaphragm B

To answer the research question about the effective length of collectors over which the shear stud connection forces are relatively uniform, the effective lengths of shear transfer along the collectors near the braced frames are estimated for prototype diaphragms A and B with concrete damage plasticity. As depicted in Figure 4-15, assuming equal area under the shear connection force distribution curve, the effective length of shear transfer is obtained by the ratio of the total area under the shear connection force distribution curve along the entire collector line to the average shear forces of the shear connections within the braced frames where the collectors are attached. The effective length of shear transfer for prototype diaphragms A and B is 146 ft and 99 ft, respectively. This means that the shear loads are transferred over a distance of 20 to 25 ft through

the collectors on each side of the braced frames, which can be considered as the effective length of collectors in design practice.

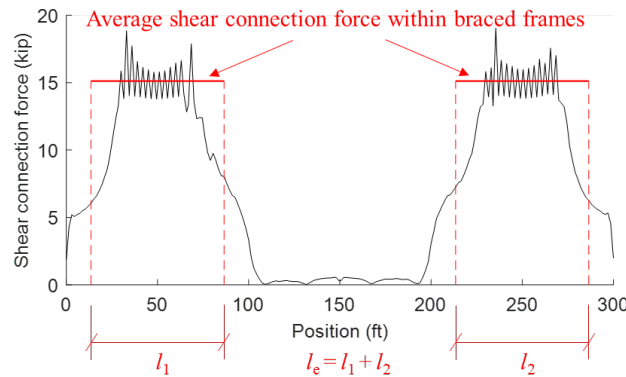


Figure 4-15 Example of effective shear transfer length calculation for prototype diaphragm A with concrete damage plasticity

Shear connection failure was not observed during the pushover analyses of the prototype diaphragms with concrete damage plasticity, as shown in Figure 4-13. The distribution of shear connection forces in the x and y directions at the end of the analysis is depicted in Figure 4-16 for the prototype diaphragms with elastic composite slab. Shear connection failure can be observed (shown in red) for all the prototype diaphragms except prototype diaphragm A, which again indicates that the convergence failure of diaphragms B, C and D were likely associated with the nonlinear behavior of the composite slab. The forces in the shear connections near the reentrant corner and the opening remained relatively small and no shear connection failure was observed at the end of the pushover analysis when shear connection failure occurred near the braced frames.

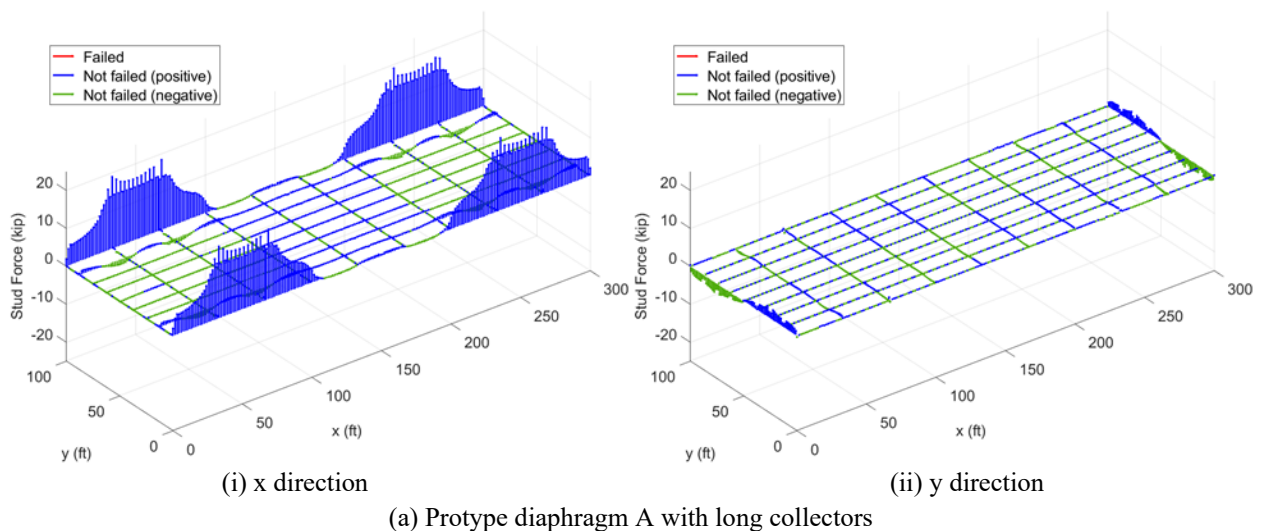
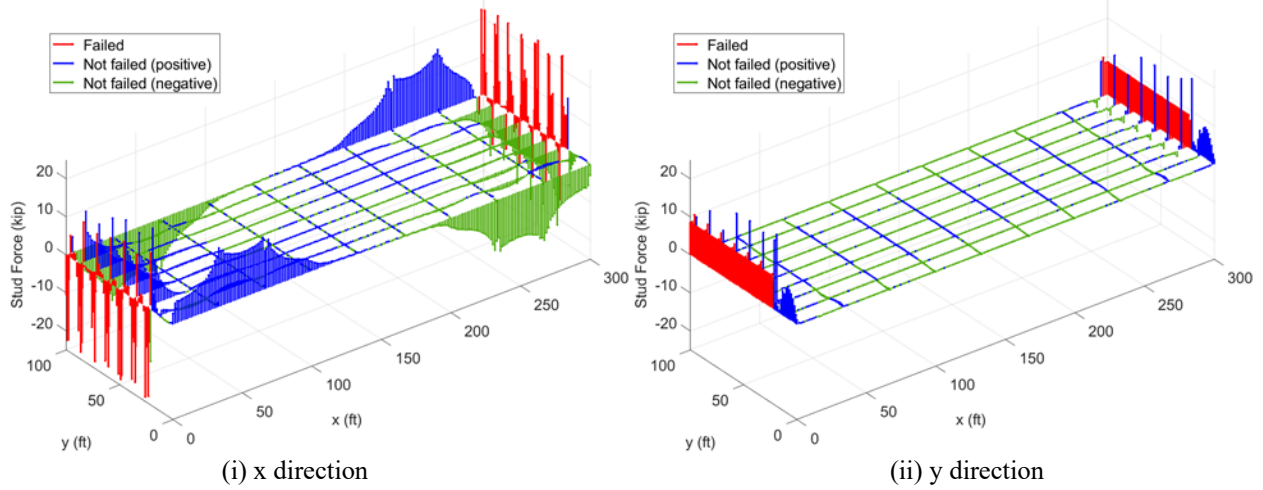
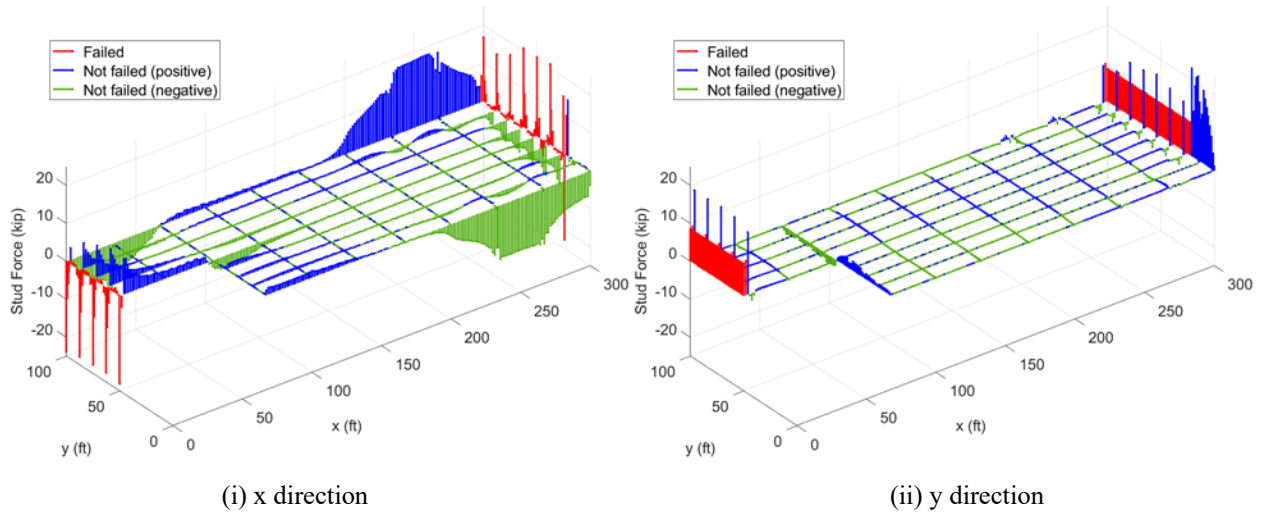


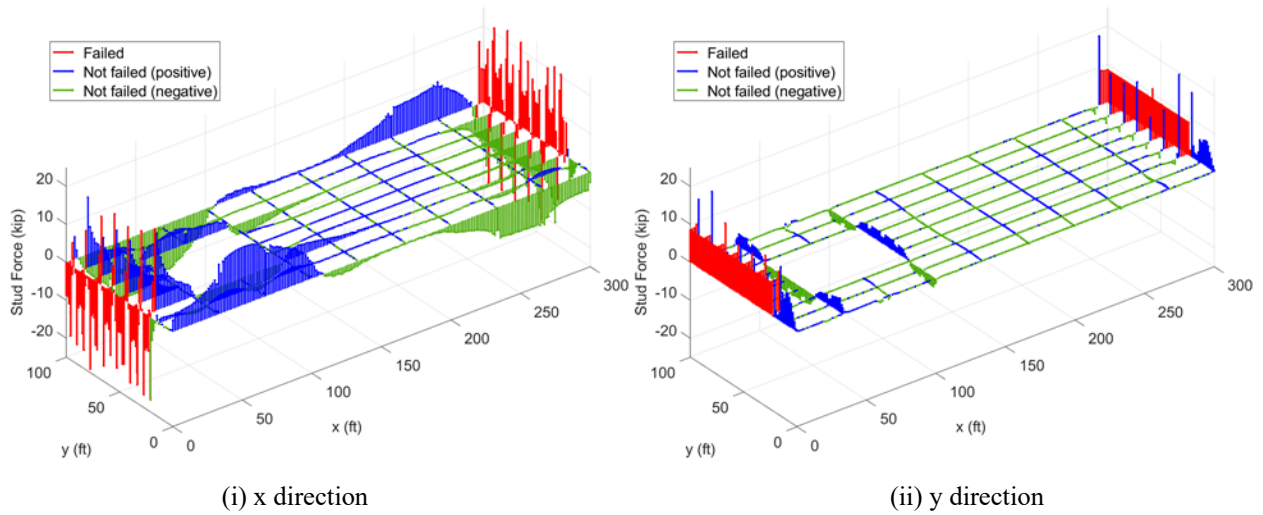
Figure 4-16 Distribution of stud forces for prototype diaphragms with elastic composite slab



(b) Prototype diaphragm B with short collectors



(c) Prototype diaphragm C with reentrant corner



(d) Prototype diaphragm D with opening

Figure 4-16 (Continued) Distribution of stud forces for prototype diaphragms with elastic composite slab

Figure 4-17 shows the progression of shear connection failure along the left edge of prototype diaphragm B with elastic composite slab. It can be observed that the failure of shear connection starts from within the braced frame and progresses outwards to the collectors on both sides. At the end of the analysis when convergence failure occurred, almost all the failed shear connections reached a deformation on the last branch of the Pingching4 backbone with a shear force equal to the residual strength of the shear connection equal to 8.5 kip. It should be noted that the inertial forces applied to the diaphragm at the first shear connection failure and at the end of the analysis are 2894 kips and 2901 kips, respectively, which indicates that the progression of shear connection failure occurred rapidly after it was initiated (i.e., unzipping).

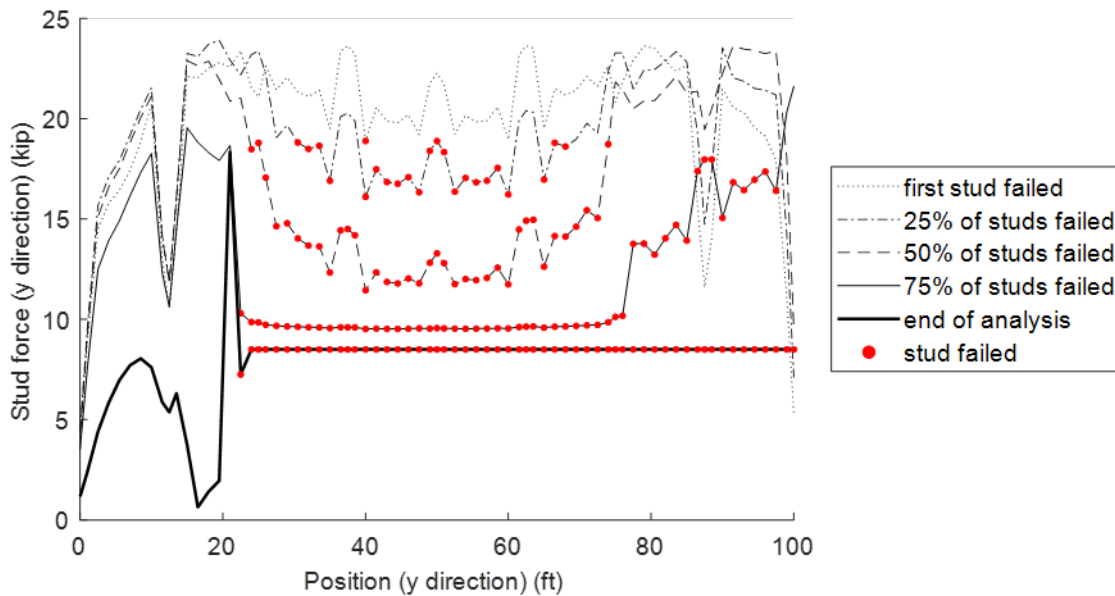


Figure 4-17 Progression of shear connection failure along the left edge of prototype diaphragm B with elastic composite slab

4.5. Conclusions

A computational study was conducted using finite element models in ABAQUS to understand the nonlinear behavior of concrete-filled steel deck diaphragm with different configurations such as long collector lines, having reentrant corners and openings, including the distribution of shear forces transferred through the shear connections along the collectors and near the reentrant corners and openings, and the progression of shear connector failure in diaphragms. A finite element modeling scheme was proposed to capture the potential limit states of such diaphragms under

lateral loading, including tension cracking of the concrete slab and the progressive failure of shear connections. Specifically, user elements of uncoupled zero-length spring pair oriented in orthogonal directions with the Pinching4 material model that can deform in the plane of the shear loading were used to model the shear stud connections; the concrete-filled steel deck was modeling using solid elements with a uniform thickness equal to the effective transformed thickness of the composite slab and with the concrete damage plasticity material; the perimeter steel beams were represented with shell elements that use an elastoplastic kinematic-hardening material. The Pinching4 material models for shear connections with different configurations were calibrated against experimental data from monotonic and cyclic pushout tests on shear connectors, and the proposed modeling scheme was validated through the modeling of cantilever diaphragm tests.

A series of prototype diaphragm structures were developed to perform finite element analysis on diaphragm behavior, which include prototype diaphragms with long collectors, with short collector, with a reentrant corner, and with an opening, respectively. Pushover analyses were performed using the validated modeling scheme, with in-plane inertial loads proportional to the diaphragm weight that were applied to the composite slab incrementally until convergence failure occurred during the analysis.

It should be noted that there are some limitations associated with the proposed modeling scheme, including: 1) beam-to-girder connections are simulated by using shared nodes on the webs of the beams and girders where they are attached, which may affect the flexibility of these connections and the distribution of shear transfer forces along collector and chord lines; 2) the bearing of the concrete floor slab against the column faces can create large shear transfer strength at column locations, but is neglected in the finite element analysis; 3) the nonlinear springs that represent the shear stud connections are not affected by damage in the surrounding concrete elements, whereas in real buildings, there may be an interaction between diagonal tension cracking of the concrete and local failure of shear stud connections; 4) shear stud connections are represented by spring pairs with two orthogonal springs that share the same Pinching4 material model, which may exhibit greater strength and stiffness than expected in real stud connections and may not be accurate if large shear forces occur in the direction perpendicular to the shear loading; 5) the concrete-filled steel deck is idealized as a solid concrete slab with an equivalent thickness without considering the interaction between the concrete and the steel deck and the residual

strength of the steel deck, which can affect the behavior of the diaphragm when large deformation occurs; and 6) the concrete damage plasticity model without including the damage variables and element removal is used to model the concrete-filled steel deck, and therefore individual cracks and the kinematic behavior of crack opening and closing may not be captured.

Results from the finite element analyses showed that the shear stress of the composite slab concentrate in the end zone of the diaphragm, i.e., near the vertical lateral force resisting system (braced frames). For diaphragms with geometric irregularities such as reentrant corner and opening, significant shear stress concentration also occurred near the corners of such irregularities. The shear forces in the direction of inertial loads are transferred through the studs primarily near the braced frames regardless of whether reentrant corner or opening is included. For prototype diaphragm B, C and D with relatively small spacing (12.5 ft) of the beams along the perpendicular direction to the applied loads, a portion of the shear forces were transferred through the shear connections on these beams near the beam-collector joints, leading to a decreased shear force in the shear connections at the locations of these joints. Due to the large axial forces in the chords in these prototype diaphragms with a long span (300 ft), a significant amount of shear connection forces was also observed in the direction perpendicular to the loading direction. For diaphragms with geometric irregularities, forces in the shear connections near the edge of the reentrant corner and the opening were significantly smaller than those on the collectors, indicating that the shear loads near the edges of reentrant corners and openings are primarily carried by the composite slab and barely transferred to the framing elements through the shear connections. The shear loads were found to be transferred over a distance of 20 to 25 ft through the collectors on each side of the braced frames, which can be considered as the effective length of collectors in design practice.

Results from pushover analyses on the same prototype diaphragms with elastic material for the composite slab implied that the convergence failure of the analysis is likely due to the failure of the composite slab for the prototype diaphragms with the concrete damage plasticity material model. It was found that the existence of reentrant corner or opening can substantially reduce the initial stiffness (up to 41%) and shear strength (up to 36%) of the diaphragm. The ratio of the total inertial force applied to the diaphragms with elastic composite slab, F_e , to the design diaphragm strength with a controlling limit state of fastener failure assuming all the shear connections along the collector lines reach the peak strength together, F_d , ranges from 76% to 86% for all the prototype

diaphragms, with an average ratio equal to 80%, indicating that the design diaphragm strength considering the peak strength of all the shear connections along the collector lines is somewhat unconservative.

Analysis on the progression of shear connection failure showed that the failure of shear connection started from within the braced frame and progresses outwards to the collectors on both sides. It was also noticed that a small increase of the applied load after the first shear connection failure led to the failure of the majority of all shear connections, which indicates that the progression of shear connection failure occurred rapidly once initiated.

CHAPTER 5. COMPUTATIONAL STUDY USING NONLINEAR 3D BUILDING MODELS

5.1. Introduction

Steel building systems with braced frames, steel deck roof diaphragms, and composite concrete on steel deck floor diaphragms are one of the most common structural systems in North America. During an earthquake, lateral inertial forces are transferred through the diaphragms to the vertical portions of the lateral force resisting system (LFRS). Conventional seismic design of these steel buildings assumes that the vertical elements of the LFRS control the dynamics of the building and that they are also the primary source of inelastic actions and hysteretic energy dissipation in the structure. However, it has been shown that diaphragms designed using traditional design procedures may be subject to inelasticity during design level earthquakes (Rodriguez et al, 2007), and in the extreme may cause collapse such as happened for several concrete parking garages with precast concrete diaphragms during the 1994 Northridge earthquake (EERI, 1996).

Current U.S. seismic design provisions ASCE 7-16 (ASCE, 2016) provide two methodologies for seismic design of diaphragms: traditional diaphragm design procedures using forces reduced by the response modification factor, R , associated with the vertical system, and alternative diaphragm design procedures using larger and more accurate elastic design forces. The alternative diaphragm design procedures incorporate a diaphragm design force reduction factor, R_s , that reduces the diaphragm demands based on the ductility and overstrength in the diaphragm. However, currently there is no R_s factor available for steel deck or concrete on steel deck diaphragms, although values for R_s have been proposed for inclusion in the upcoming edition of NERHRP Recommended Seismic Provisions including $R_s = 2.5$ for steel deck diaphragms satisfying specific special detailing requirements and $R_s = 2.0$ for composite concrete on steel deck diaphragms.

To explore the impact of different diaphragm design procedures on the seismic performance of building systems, a computational study using three-dimensional (3D) building models that capture nonlinear diaphragm behavior and its interaction with the nonlinear vertical LFRS was conducted. This report presents details of the study starting with definition of a series of 1, 4, 8,

and 12-story archetype buildings with buckling-restrained braced frames (BRBF) for the vertical system and three designs for the diaphragms. The modeling scheme uses computationally efficient calibrated frame and truss elements to capture the realistic nonlinear behavior of both the BRBFs and the diaphragms. Modal analysis, nonlinear static pushover analyses, and nonlinear response history analyses using 44 ground motion records scaled to three hazard levels were performed to investigate the behavior and seismic performance of the buildings.

The objectives of this study include: 1) to examine the effect of diaphragm rigidity on the dynamic properties of buildings, 2) to understand the extents of diaphragm inelasticity at specified diaphragm hazard levels, 3) to investigate the probability of collapse for buildings designed using different diaphragm design approaches, and 4) to evaluate whether the use of proposed values of R_s for steel deck and concrete on steel deck diaphragms has a significant effect on the seismic behavior of buildings.

5.2. Development of Archetype Buildings

A series of 1, 4, 8, and 12-story steel buildings with BRBFs for the vertical LFRS were selected as archetype buildings for this study and designed to the current US building code (Torabian et al, 2019). Three different diaphragm design scenarios were considered: 1) Traditional Design using conventional diaphragm design procedures from Section 12.10.1 of ASCE 7-16 (ASCE, 2016), 2) Alternative 1 using the alternative diaphragm design procedures from Section 12.10.3 of ASCE 7-16 with $R_s = 1.0$, and 3) Alternative 2 using the alternative diaphragm design procedures with $R_s = 2.0$ for composite concrete on steel deck diaphragm and $R_s = 2.5$ for bare steel deck diaphragm.

For 1-story buildings, two different types of roof system were considered, i.e., a composite concrete on steel deck roof, and a bare steel deck roof. The 1-story buildings with composite concrete on steel deck roof may be less common than those with bare steel deck roof, but they were included to enable comparison to multi-story buildings with composite concrete on steel deck floors. For all other multi-story buildings, bare steel deck roof and composite concrete on steel deck floors were used. Table 5-1 shows a list of the buildings analyzed in this study. Note that the diaphragm force demands in the traditional design and the alternative design with $R_s = 2.0$ for composite floors and $R_s = 2.5$ for the bare steel deck roof (Alternative 2) are controlled by the

minimum value allowed for diaphragm design forces (see Table 5-4 for details), and therefore the archetype buildings designed with these two diaphragm design procedures were identical.

Table 5-1 List of BRB Archetype Buildings for the Study

Building Number	Number of Stories	Diaphragm Design
1	1	Traditional / Alternative 2 ($R_s = 2.5$ with bare steel deck roof)
2	1	Alternative 1 ($R_s = 1$ with bare steel deck roof)
3	1	Traditional / Alternative 2 ($R_s = 2.0$ with composite concrete on steel deck roof)
4	1	Alternative 1 ($R_s = 1$ with composite concrete on steel deck roof)
5	4	Traditional / Alternative 2 ($R_s = 2$ for floors, $R_s = 2.5$ for roof)
6	4	Alternative 1 ($R_s = 1$ for floors and roof)
7	8	Traditional / Alternative 2 ($R_s = 2$ for floors, $R_s = 2.5$ for roof)
8	8	Alternative 1 ($R_s = 1$ for floors and roof)
9	12	Traditional / Alternative 2 ($R_s = 2$ for floors, $R_s = 2.5$ for roof)
10	12	Alternative 1 ($R_s = 1$ for floors and roof)

Table 5-2 provides the loading information used in the design of the archetype buildings and associated typical seismic weights. Detailed site information and design parameters are given in

Table 5-3, including the location, risk category, importance factor I_e , spectral response acceleration parameter at short periods S_s , spectral response acceleration parameter at a period of 1 sec S_1 , site class, response modification coefficient R , overstrength factor Ω_0 , deflection amplification factor C_d , approximate fundamental period of the building T_a , upper limit on approximate fundamental period $C_u T_a$, fundamental period of the building obtained from a SAP2000 model T_{model} , effective seismic weight of the building W , and design base shear V .

Table 5-2 Archetype Building Loading and Design Information

Composite Concrete on Steel Deck Floor / Roof	Bare Steel Deck Roof	Seismic Weight	Site Information	Design Parameters
Dead Load = 56.5 psf slab + 22 psf superimposed = 78.5 psf Live Load = 50 psf + 15 psf partition = 65 psf Exterior wall = 40 psf	Dead Load = 3 psf slab + 22 psf superimposed = 25 psf Live Load = 20 psf + 15 psf partition = 35 psf Exterior wall = 40 psf	Typical Floor = 2545 kips Composite Concrete on Steel Deck Roof = 2630 kips Bare Steel Deck Roof = 1271 kips	Irvine, CA Risk Category 2 $I_e = 1.0$ $S_s = 1.55$ $S_1 = 0.57$ Site Class D	$R = 8$ $\Omega_0 = 2.5$ $C_d = 5$

Table 5-3 Prototype Building Seismic Design Information

Design Parameter	1-story Bare Deck Roof	1-story Composite Roof	4-story	8-story	12-story
T_a (sec)	0.22	0.22	0.58	0.96	1.30
$C_u T_a$ (sec)	0.30	0.30	0.81	1.34	1.81
T_{model} (sec)	0.46	0.41	0.76	1.31	2.00
W (kip)	1271	2630	8906	19086	29266
V (kip)	164	339	830	1036	1326

The buildings all use the same plan dimensions, shown in Figure 5-1 and Figure 5-2, 300 ft by 100 ft with a story height of 14 ft at the first story and 12.5 ft for a typical story. Four bays of BRBFs are located on the perimeter of the building in each orthogonal direction and Figure 5-3 shows an elevation view of the BRBFs in the 4-story building. Typical details for the floor and roof diaphragms are given in notes on Figure 5-1 and Figure 5-2, as designed based on the diaphragm design forces tabulated in Table 5-4. Members sizes for each archetype building are provided in Table A-1, Table A-2, and Table A-3 in Appendix A. Additional details for the design of the archetype buildings can be found in Torabian et al (2019).

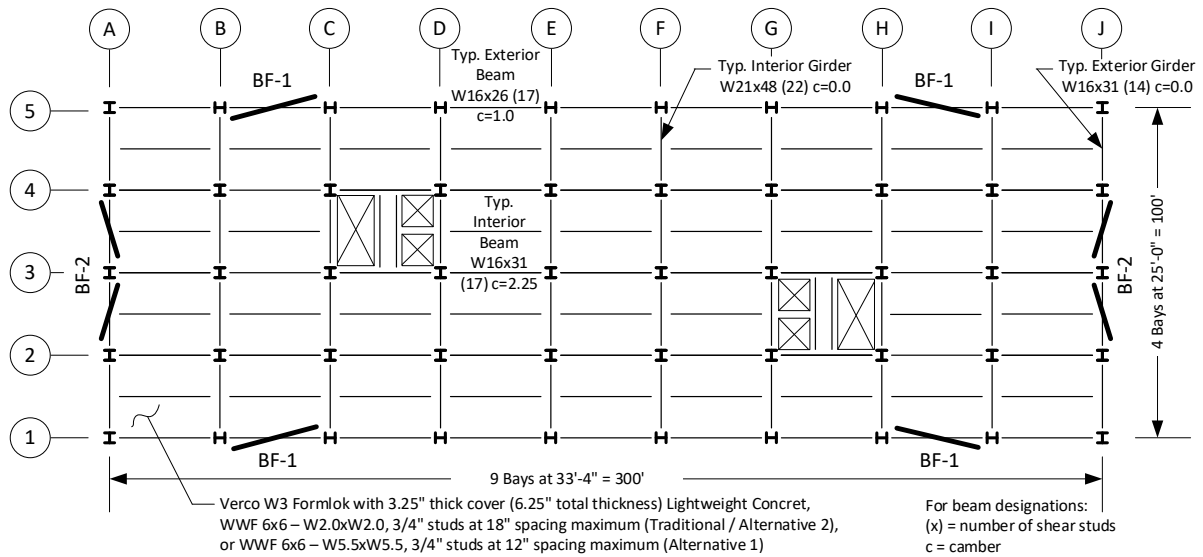


Figure 5-1 Typical Floor Framing Plan

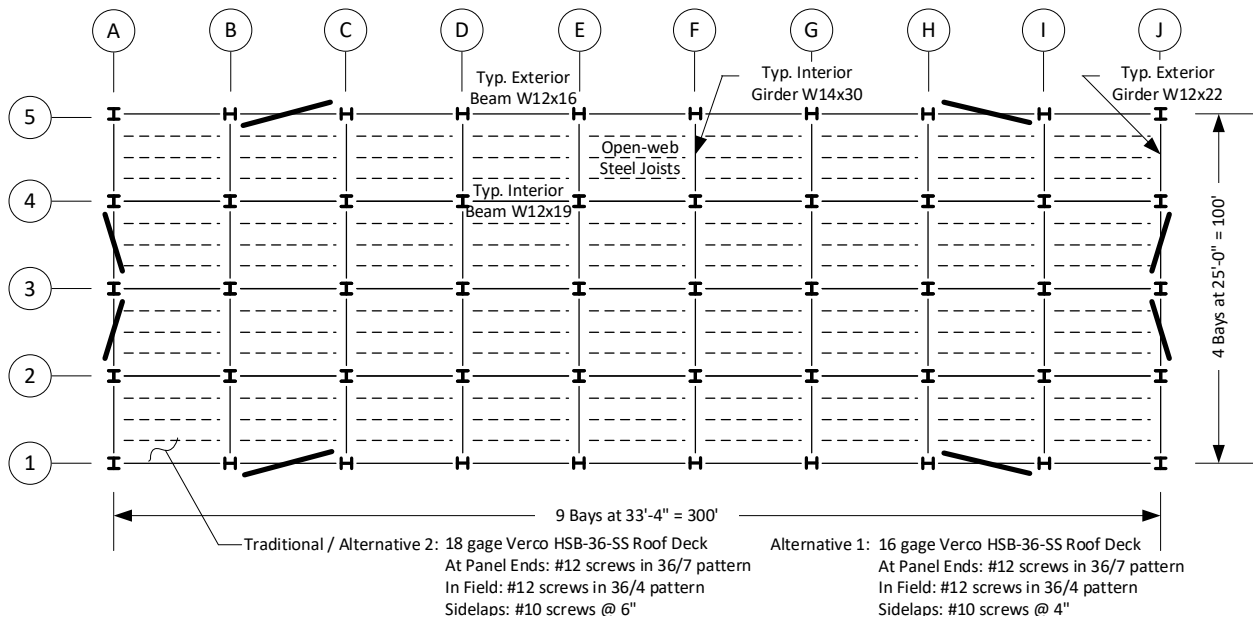


Figure 5-2 Typical Roof Framing Plan

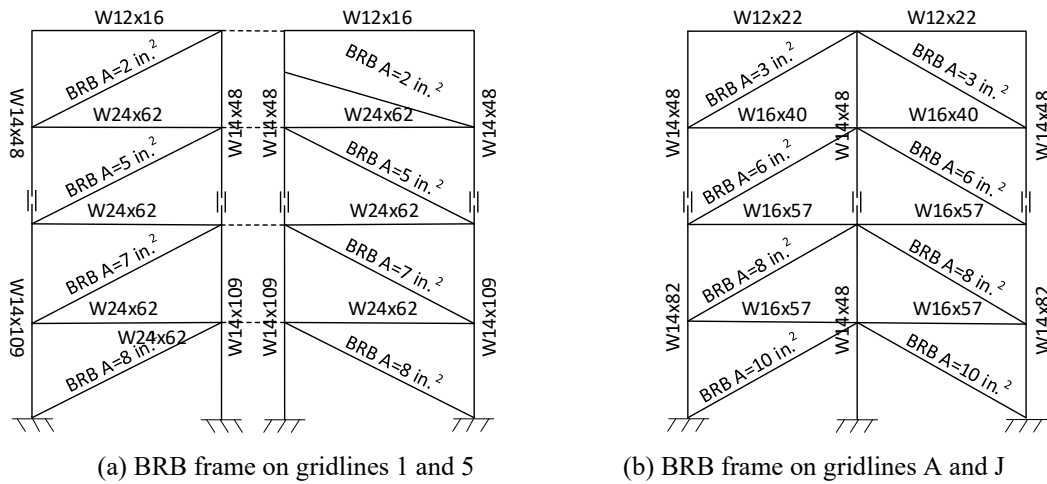


Figure 5-3 Elevation view of 4-story building braced frames

Table 5-4 Diaphragm Design Shear per Unit width at Diaphragm Edge along Short Dimension of Building

Archetype Building	Level	Diaphragm Design Forces (kip/ft)		
		Traditional*	Alternative 1	Alternative 2*
1-story (bare steel deck roof)	Roof	1.31	2.10	1.31
1-story (composite concrete on steel deck roof)	Roof	2.71	4.27	2.71
4-story	Roof	1.31	2.10	1.31
	2-4	2.62	4.25	2.62
	3	2.62	4.57	2.62
	2	2.62	4.89	2.62
8-story	Roof	1.31	2.10	1.31
	8	2.62	4.19	2.62
	7	2.62	4.25	2.62
	6	2.62	4.42	2.62
	5	2.62	4.58	2.62
	4	2.62	4.74	2.62
	3	2.62	4.90	2.62
	2	2.62	5.06	2.62
12-story	Roof	1.31	2.10	1.31
	12	2.62	4.19	2.62
	11	2.62	4.19	2.62
	10	2.62	4.26	2.62
	9	2.62	4.36	2.62
	8	2.62	4.47	2.62
	7	2.62	4.58	2.62
	6	2.62	4.69	2.62
	5	2.62	4.80	2.62
	4	2.62	4.91	2.62
	3	2.62	5.01	2.62
	2	2.62	5.12	2.62

*: All diaphragm design forces for the Traditional and Alternative 2 design were controlled by the minimum, $0.2S_{DS}$ multiplied by the diaphragm seismic weight.

5.3. Development of Computational Models

Nonlinear 3D computational models were created using the *OpenSees* software (Mazzoni et al, 2006), a structural analysis program widely used for earthquake engineering simulations. Figure 5-4 shows a schematic view of the 1, 4, 8, and 12-story archetype building models used in this study. Details of the modeling scheme is provided in this section.

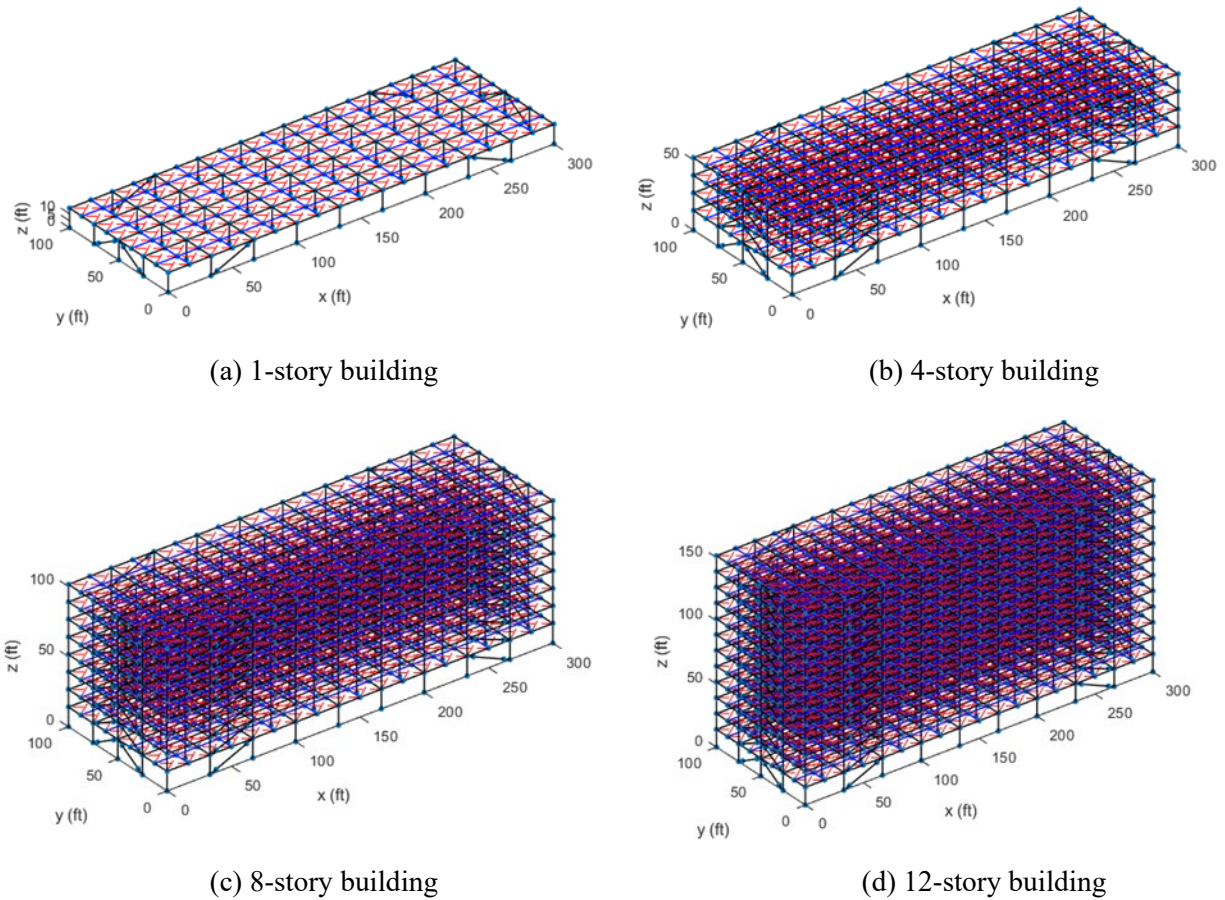


Figure 5-4 3D OpenSees models of archetype buildings

5.3.1. Modeling of Diaphragms

Truss elements were used to simulate the in-plane diaphragm behavior in the archetype buildings. The load-deformation behavior of a diaphragm is typically obtained through cantilever diaphragm tests in which a steel deck diaphragm with or without concrete fill is supported with one edge fixed and the parallel edge subjected to a shear loading (Figure 5-5a). Using the force-displacement data from these types of tests, computational models with diagonal nonlinear truss

elements of unit cross-section area (Figure 5-5b) were calibrated to capture the behavior of the diaphragm tests. All connections were modeled as pinned, and the perimeter framing beams were modeled as nonlinear beam-column elements with kinematic hardening material and with the same size of cross sections as the test. Figure 5-6 shows the meshing of diaphragms in the computational models of the archetype buildings. The dimension of the diaphragm unit in the mesh is 200 in.×150 in., which is similar in scale to the test specimens used for calibration.

The cantilever diaphragm test database established by O'Brien et al (2017) was utilized as a tool to help select specimens for diaphragm model calibration. For the roof diaphragm, the specimen labeled as Test 33 by Martin (2002) with 20-gage P3615 1.5 in. B-deck was selected to satisfy the force demand for the archetype building roof diaphragm with Traditional / Alternative 2 design procedures (herein denoted as SP1). For the floor diaphragm, Specimen 3/6.25-4-L-NF-DT tested by Avellaneda-Ramirez et al (2021) was used, which consisted of 3 in. deck, with lightweight concrete fill and 6.25 in. total thickness (herein denoted as SP2). The dimensions of the test specimens (240 in.×144 in. for SP1 and 180 in.×144 in. for SP2) are close to those of the diaphragm units in the mesh of the building models.

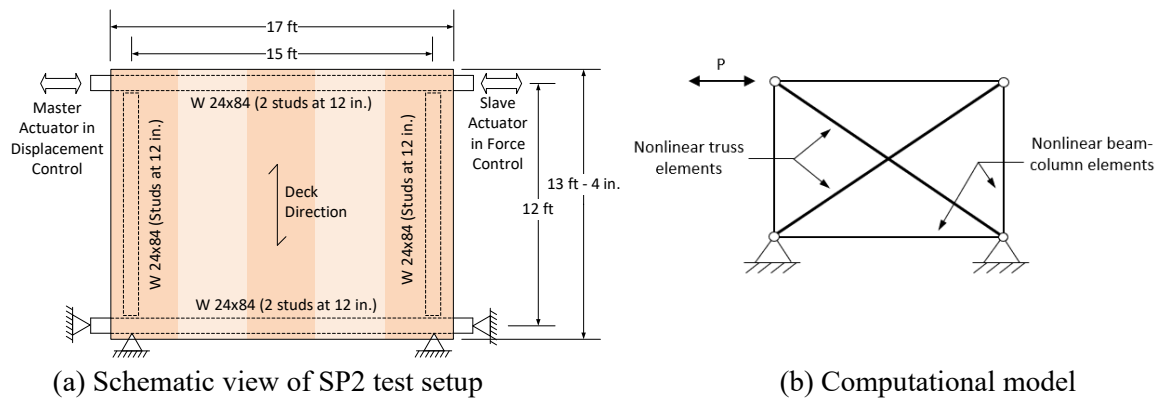


Figure 5-5 Test setup and computational model of cantilever diaphragm test

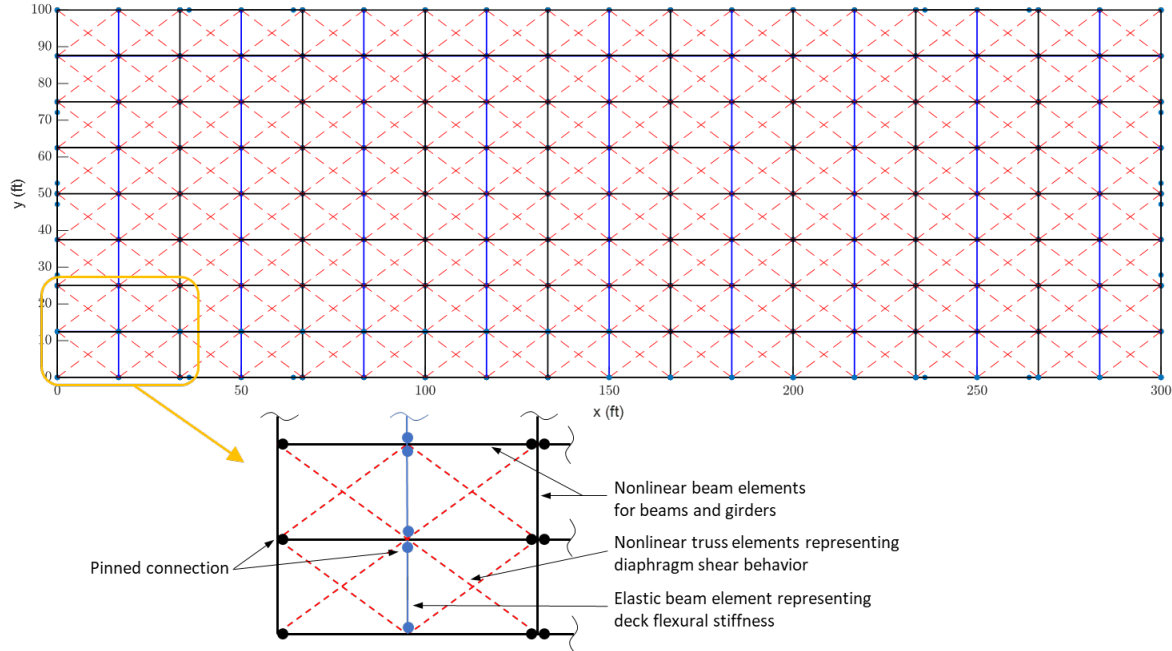


Figure 5-6 Diaphragm meshing in computational models of archetype buildings

As is shown in Figure 5-7, the Pinching4 material model in OpenSees was used for the truss elements. This model is capable of capturing the hysteretic pinching, cyclic strength degradation, and cyclic stiffness degradation behavior of diaphragms. Material parameters for the Pinching4 model, including backbone stresses and strains and cyclic strength and stiffness degradation parameters, were calibrated through a six-step optimization algorithm to achieve an optimal match between hysteretic response from the simulation and test that minimizes the objective functions:

- 1) The experimental stress-strain backbone curve was first obtained from the cyclic test data, and was simplified to a curve with multiple linear segments as defined by Pinching4 model, where the third characteristic point was obtained at the peak load of the backbone, and the first, second, and fourth points were obtained by interpolation at 40%, 80%, and 40% (for SP1) or 30% (for SP2), respectively, of the peak load on the backbone. The initial stress-strain backbone was obtained by scaling the backbone of the cyclic cantilever test data with a factor equal to 1.3, which was selected from multiple runs of the optimization algorithm with the different scale factors for the initial stress-strain backbone such that the sum of the errors for peak forces, reloading stiffness, unloading stiffness, and cumulative energy dissipation of the hysteretic loops, considering different weights for each type of error, was the minimum.

- 2) The strength degradation parameters considering displacement and energy history are optimized to achieve a minimum error for the peak forces of the hysteretic loops.
- 3) The reloading stiffness degradation parameters considering displacement and energy history are optimized to achieve a minimum error for the reloading stiffness of the hysteretic loops.
- 4) The unloading stiffness degradation parameters considering displacement and energy history are optimized to achieve a minimum error for the unloading stiffness of the hysteretic loops.
- 5) The parameters for reloading / unloading are optimized to achieve a minimum error for the cumulative energy dissipation in the hysteretic loops.
- 6) All the Pinching4 parameters are optimized together to achieve a minimum value for an objective function defined as the sum of the errors for peak forces, reloading stiffness, unloading stiffness, and cumulative energy dissipation of the hysteretic loops, considering different weights for each type of error.

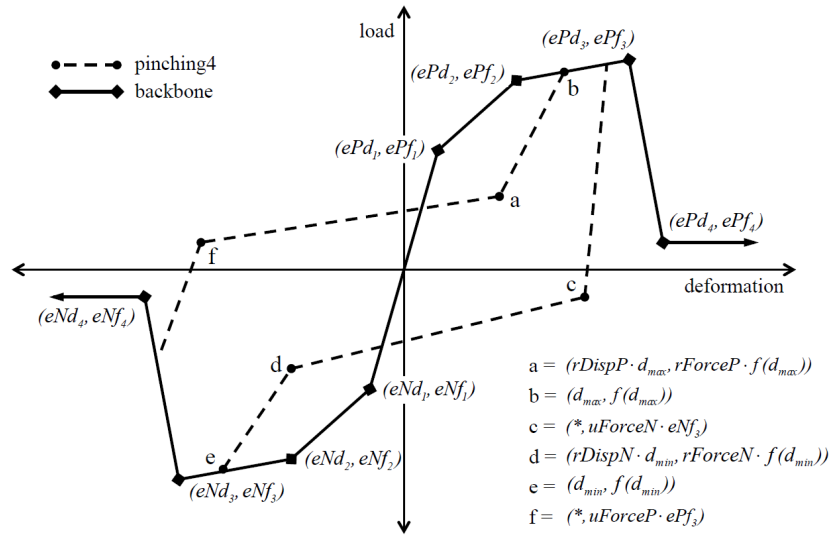


Figure 5-7 Pinching4 material model [from (Ding, 2015)]

Table 5-5 shows the resulting values of the Pinching4 material model parameters for the two selected diaphragm specimens. It should be noted that the dimensions of the archetype building diaphragm units do not coincide with those of the test specimens, and therefore the backbone parameters were modified using the strategy described in Appendix A so that the diaphragm shear

strength per unit length is consistently represented. A comparison of the hysteretic response from the calibrated diaphragm simulation and that from the experiment is shown in Figure 5-8.

Table 5-5 Calibrated Pinching4 Material Model Parameters

Test	Backbone				Pinching			Strength Degradation					Stiffness Degradation					Energy Dissipation
	ε_1, σ_1 (ksi)	ε_2, σ_2 (ksi)	ε_3, σ_3 (ksi)	ε_4, σ_4 (ksi)	$r_{\Delta^+}, r_{\Delta^-}$	r_{F^+}, r_{F^-}	u_{F^+}, u_{F^-}	gF_1	gF_2	gF_3	gF_4	gF_{lim}	gK_1, gD_1	gK_2, gD_2	gK_3, gD_3	gK_4, gD_4	gK_{lim}, gD_{lim}	gE
SP1	0.0008, 22.18	0.0017, 28.90	0.0033, 30.69	0.0053, 23.97	0.20, 0.35	0.20, 0.35	0.10, 0.12	0	0.35	0	0.70	0.90	0, 0	0, 0.50	0, 0	0, 0.75	0, 0.90	4.31
SP2	0.0005, 63.46	0.0006, 76.41	0.0014, 107.40	0.0143, 48.33	-0.06, -0.06	0.12, 0.12	0.11, 0.11	0	0.83	0.0	0.46	0.33	1.09, 0.14	0.76, 0.47	0.32, 0.12	0.75, 0.10	1.04, 0.61	4.29

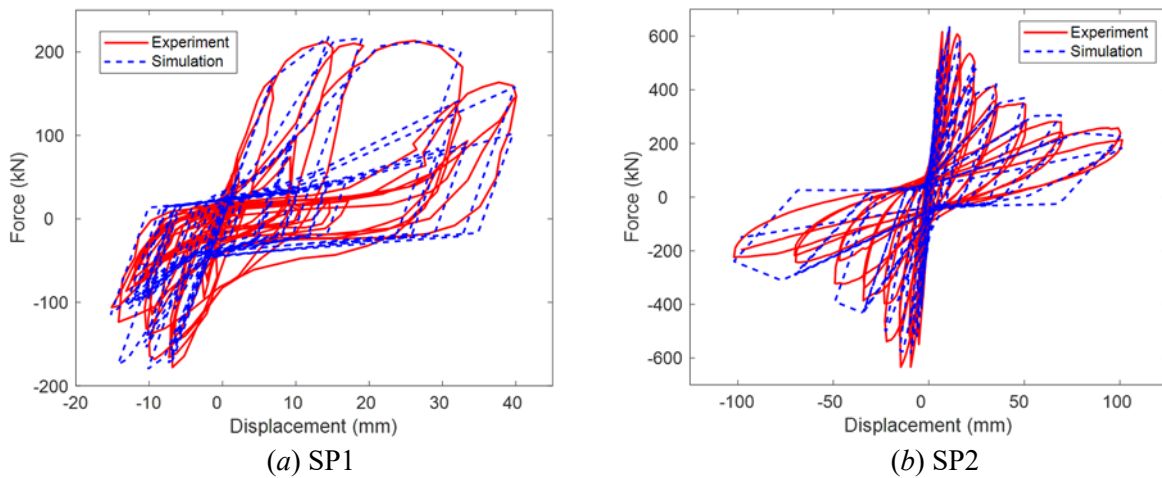


Figure 5-8 Hysteretic response of diaphragm from experiment and simulation

Table 5-6 provides the diaphragm demands and designs using ASD for the archetype buildings, where v is the shear demand per unit width of the diaphragm (as given in Table 5-4 in detail), Ω is the safety factor for ASD ($\Omega = 1.5$), and v_a is the allowable strength of the diaphragm given by the manufacturers based on the resulting design as described in the notes of Table 5-6. For the models of the same archetype building with different diaphragm designs that are not a perfect match with past testing, the same Pinching4 model parameters were used except that the backbone stresses were scaled so that the peak strength exactly equals the expected nominal strength of the diaphragm from design. In this case, no overstrength of the diaphragm is considered. The expected

nominal strength is calculated with prediction equations to the best knowledge of the authors. For bare steel deck diaphragm, DDM04 (Luttrell et al., 2015) and AISI 310-16 (AISI, 2016) are used to calculate the nominal strength. For concrete on steel deck diaphragm, the nominal strength is determined as the lesser of: the strength associated with concrete slab diagonal tension cracking limit state calculated with the proposed equations (for AISI S310 2022 edition) in O'Brien et al 2017, in addition to the contribution of reinforcing steel which is calculated with ACI 318-14 (ACI, 2014); and the strength associated with the perimeter fastener limit state calculated per AISC 360-16 (AISC, 2016). The expected nominal strength and scale factors are given in Table 5-7, where v_{pred} is the expected nominal strength of the diaphragm design, and v_{exp} is the peak strength from hysteretic curve.

Table 5-6 Diaphragm Demands and Design

Archetype Building			v (kip/ft)	v/Ω (kip/ft)	Diaphragm Design	v_a (kip/ft)
1-story (bare steel deck roof)	Traditional / Alternative 2		1.31	0.87	Bare Deck 1*	1.04
	Alternative 1		2.10	1.40	Bare Deck 2*	1.73
1-story (comp. deck roof)	Traditional / Alternative 2		2.67	1.78	Composite 1*	1.81
	Alternative 1		4.27	2.85	Composite 2*	3.70
4-story	Traditional / Alternative 2	Roof	1.31	0.87	Bare Deck 1	1.04
		Levels 1-3	2.62	1.75	Composite 1	1.81
	Alternative 1	Roof	2.10	1.40	Bare Deck 2	1.73
		Levels 1-3	4.25-4.89	3.26	Composite 2	3.70
8-story	Traditional / Alternative 2	Roof	1.31	0.87	Bare Deck 1	1.04
		Levels 1-7	2.62	1.75	Composite 1	1.81
	Alternative 1	Roof	2.10	1.40	Bare Deck 2	1.73
		Levels 1-7	4.19-5.06	3.37	Composite 2	3.70
12-story	Traditional / Alternative 2	Roof	1.31	0.87	Bare Deck 1	1.04
		Levels 1-11	2.62	1.75	Composite 1	1.81
	Alternative 1	Roof	2.10	1.40	Bare Deck 2	1.73
		Levels 1-11	4.19-5.12	3.41	Composite 2	3.70

*: Details of the diaphragm design are given as follows.

Bare Deck 1: 18 gage HSB®-36-SS steel deck, 36/7/4 #12 screw pattern at supports, #10@6" sidelap attachments.

Bare Deck 2: 16 gage HSB®-36-SS steel deck, 36/7/4 #12 screw pattern at supports, #10@4" sidelap attachments.

Composite 1: Vercor W3 Formlok with 3.25" thick lightweight concrete cover (6.25" total thickness, $f'_c = 3$ ksi), WWF 6×6 – W2.0×W2.0, 3/4" studs at 18" spacing maximum

Composite 2: Vercor W3 Formlok with 3.25" thick lightweight concrete cover (6.25" total thickness, $f'_c = 3$ ksi), WWF 6×6 – W5.5×W5.5, 3/4" studs at 12" spacing maximum

Table 5-7 Diaphragm Nominal Strength and Backbone Stress Scale Factors

Archetype Building			v_{pred} (kip/ft)	Limit State of v_{pred}	v_{exp} (kip/ft)	Limit State of v_{exp}	Scale Factor
1-story (bare steel deck roof)	Trad. /	Alt. 2	2.18	connection	2.41	sidelap fastener	0.906
		Alt. 1	3.50	connection	2.41	sidelap fastener	1.452
1-story (comp. deck roof)	Trad. /	Alt. 2	11.40	connection	9.55	sidelap fastener	1.194
		Alt. 1	16.79	connection	9.55	sidelap fastener	1.758
4-story	Trad. /	Roof	2.18	connection	2.41	sidelap fastener	0.906
	Alt. 2	Levels 1-3	11.40	perim. fastener	9.55	diag. tension crack.	1.194
	Alt. 1	Roof	3.50	connection	2.41	sidelap fastener	1.452
		Levels 1-3	16.79	diag. tension crack.	9.55	diag. tension crack.	1.758
8-story	Trad. /	Roof	2.18	connection	2.41	sidelap fastener	0.906
	Alt. 2	Levels 1-7	11.40	perim. fastener	9.55	diag. tension crack.	1.194
	Alt. 1	Roof	3.50	connection	2.41	sidelap fastener	1.452
		Levels 1-7	16.79	diag. tension crack.	9.55	diag. tension crack.	1.758
12-story	Trad. /	Roof	2.18	connection	2.41	sidelap fastener	0.906
	Alt. 2	Levels 1-11	11.40	perim. fastener	9.55	diag. tension crack.	1.194
	Alt. 1	Roof	3.50	connection	2.41	sidelap fastener	1.452
		Levels 1-11	16.79	diag. tension crack.	9.55	diag. tension crack.	1.758

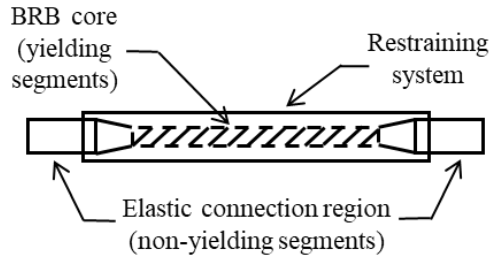
The limit states that control the nominal strength calculation and the experimental strength are also provided in Table 5-7. While it would be ideal to use test specimens that match the predicted limit states, test data was not available for some of the diaphragm configurations and limit states considered herein at the time this study was conducted. Therefore, the test specimens selected were used to represent some of the diaphragm designs even though their limit states do not match exactly. This was deemed acceptable for concrete on steel deck diaphragms whose limit states differ considerable, as it can be shown with other experimental test data (Eatherton et al., 2020) that the hysteretic behavior of a test specimen with the limit state of v_{pred} is similar to that of the specimen selected herein. Also, it can be observed that the allowable strength of the diaphragms v_a in Table 5-6 is substantially smaller than the predicted strength v_{pred} given in Table 5-7, which is due to the ASD safety factor and conservation in current diaphragm design.

5.3.2. Modeling of Buckling-Restrained Braces

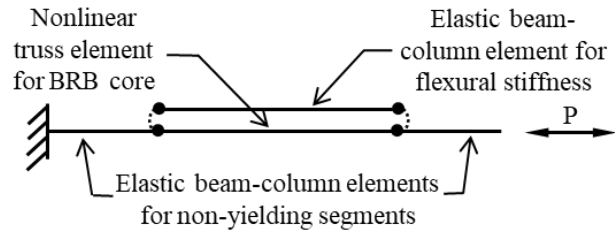
As shown in Figure 5-9, the BRB core (restrained yielding segment) is represented by a nonlinear truss element with *Steel4* material model in *OpenSees*, the non-yielding segments on both ends are modeled with elastic beam-column elements (with cross-section area equal to 10 times that of the BRB core), and another elastic beam-column element with negligible cross-section area and large bending stiffness is also used to connect the non-yielding segments to fix the rotational degrees of freedom and prevent instability of the truss element.

The material model for the BRB core was calibrated to match the behavior of specimens tested by Newell et al (2006). All specimens from the test (Specimens 1G, 2G, 3G, and 4G) were examined, and the maximum cross-section area of the BRBs in the archetype building is 14 in², which is closest to Specimens 1G and 2G (12 in²). However, Specimen 2G had increased compressive strength during large displacement cycles due to the bearing of core plate against the confining HSS. As a representative option, the calibrated *Steel4* material parameters for Specimen 1G were used in the archetype building modeling and the values are given in Table 5-8. Figure 5-10 shows the hysteretic curves of the calibrated model as compared to test results from Specimens 1G and 3G. For the building models, the same parameters and configuration of the BRB model were used, except that the cross section area of the core brace was changed to match the BRB design of the buildings. Figure 5-11 shows a schematic view of BRB modeling in the 4-story archetype building.

The fatigue material model *uniaxialMaterial Fatigue* in *OpenSees* was also calibrated to capture BRB fracture. The only specimen from Newell et al. (2006) that fractured was Specimen 3, and therefore its test data was used in the calibration. Two parameters were calibrated: ϵ_0 , the strain at which one cycle will cause failure, and m , the slope of Coffin-Manson curve (Coffin, 1954; Manson, 1954) in log-log space. The value of ϵ_0 was assumed to be 0.2 based on an appropriate elongation at fracture of an ASTM A36 plate subjected to monotonic tension test per ASTM standards (ASTM, 2019). The value of m was set equal to -0.5976 to produce fracture at a point in the simulation close to that in the test of Specimen 3G.

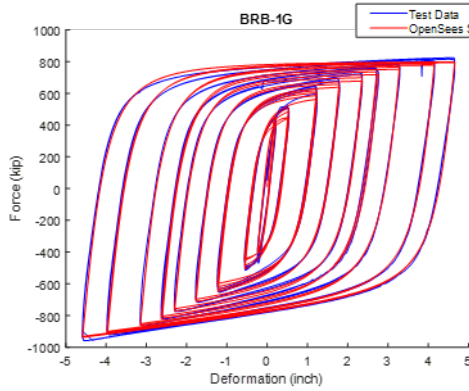


(a) Schematic BRB assembly

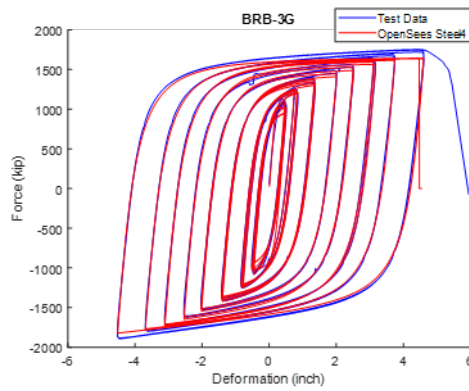


(b) Computational model

Figure 5-9 Configuration of a typical BRB and computational model

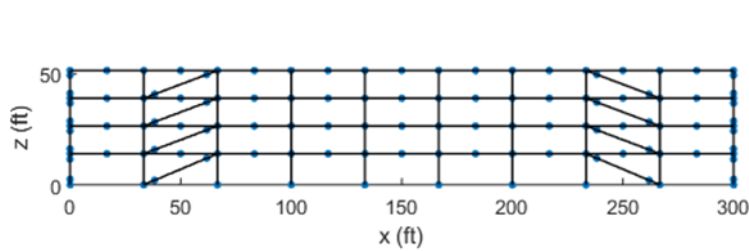


(a) Specimen 1G

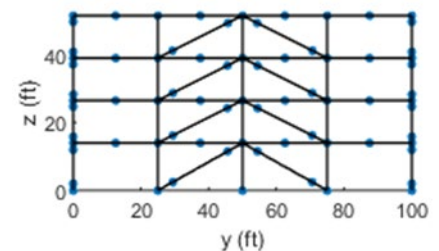


(b) Specimen 3G

Figure 5-10 Calibration of BRB Steel4 models for Specimens 1G and 3G



(a) x-z view



(b) y-z view

Figure 5-11 Schematic view of BRB modeling in 4-story archetype building

Table 5-8 Calibrated Steel4 Parameters

b_k	R_0	r_1	r_2	b_i	b_l	ρ_i	R_i	l_{yp}	f_u	R_u
0	20.9837	0.9122	0.1209	0.0306	0	0.7262	1.3134	18.2022	70.3000	620.6286
b_{kc}	R_{0c}	r_{1c}	r_{2c}	b_{ic}	b_{lc}	ρ_{ic}	R_{ic}	l_{ypc}	f_{uc}	R_{uc}
0.0121	18.9116	0.9133	0.1232	0.0020	0	0.9061	2.9727	37.3548	108.4701	583.5268

5.3.3. Other Modeling Details

Additional details of the models are discussed in this section.

5.3.3.1. Boundary Conditions and Joint Fixity

All columns were pinned at the base and continuous over the building height. All the beam-to-column and beam-to-beam joints were pinned except for the beam-to-column joints of the BRB frames which were made rigid for all degrees of freedom. The reason for making these connections rigid is that in practice these connections have substantial gusset plates, welds and/or bolts that make them effectively act as a moment connection.

5.3.3.2. Gravity Loads and Masses

As recommended by FEMA P695 (FEMA, 2009) the gravity loads included a combination of dead loads and live loads ($1.05D+0.25L$). Masses were determined from the dead loads and lumped at the column nodes on each floor. Masses at typical node locations are given in Table 5-9.

Table 5-9 Masses at Typical Node Locations

Level	Masses at Different Locations (kip-sec ² /in.)			
	Corner	Left/Right Edge	Top/Bottom Edge	Interior
Roof	0.046	0.059	0.067	0.070
Typical Floor	0.077	0.110	0.121	0.155
2 nd Floor	0.079	0.112	0.123	0.155

5.3.3.3. Material and Geometric Nonlinearity

Both material and geometric nonlinearity were considered in the analysis. In addition to the aforementioned nonlinear material models used for diaphragms and BRB's, the columns and beams were represented by nonlinear forced-based beam-column elements with fiber-section formulation and linear kinematic hardening material with a yield stress equal to 60 ksi, elastic modulus equal to 29000 ksi, shear modulus equal to 11500 ksi, and hardening modulus equal to 450 ksi. The fiber section is formulated with eight Gauss-Lobatto integration points along the element, and the number of fibers along the web depth, web thickness, flange width, and flange thickness of the cross section is 16, 2, 16, and 4, respectively. Geometric nonlinearity was

considered by including the gravity loads and using the P-Delta coordinate transformation algorithm in *OpenSees* for the columns.

5.3.3.4. Damping

For nonlinear response history analyses, Rayleigh damping with a critical damping ratio equal to 2% for the 1st and 4th modes was used for the archetype building models.

5.3.3.5. Encouraging Convergence

An algorithm with multiple steps was developed to encourage convergence in the nonlinear response history analyses and is described as follows. Starting from the first trial for convergence at each time step, if convergence fails, then the algorithm will move to the next trial step. A flow chart of the convergence algorithm is also shown in Figure 5-12.

- 1) Use a convergence criterion based on the unbalanced energy (*EnergyIncr*) with tolerance equal to 1e-12 kip-in.
- 2) Try all available algorithms for solving system equations (*Newton*, *ModifiedNewton*, *NewtonLineSearch*, *Broyden*, and *KrylovNewton*).
- 3) Reduce the applied displacement increment for pushover analysis or the time step for response history analysis by a factor of 10.
- 4) Reduce the applied displacement increment for pushover analysis or the time step for response history analysis by a factor of 100.
- 5) Temporarily relax convergence criterion with the tolerance amplified by a factor of 10.
- 6) Temporarily relax convergence criterion with the tolerance amplified by a factor of 100.
- 7) Change the convergence criterion to the one based on the norm of unbalanced forces (*NormUnbalance*) with an initial value of tolerance equal to 1e-5 (unit in kip and kip-in.).
- 8) Go through Steps 2 to 6 again.
- 9) Change the convergence criterion to the one based on the norm of displacement increment (*NormDispIncr*) with an initial value of tolerance equal to 1e-6 (unit in in. and rad.).
- 10) Go through Steps 2 to 6 again.
- 11) For response history analysis, increase the Rayleigh damping ratio of the whole structure to 5% and then 10% to facilitate the convergence of a certain time step.

- 12) If all these attempts do not work, the simulation is considered to have experienced convergence failure and the analysis is terminated.

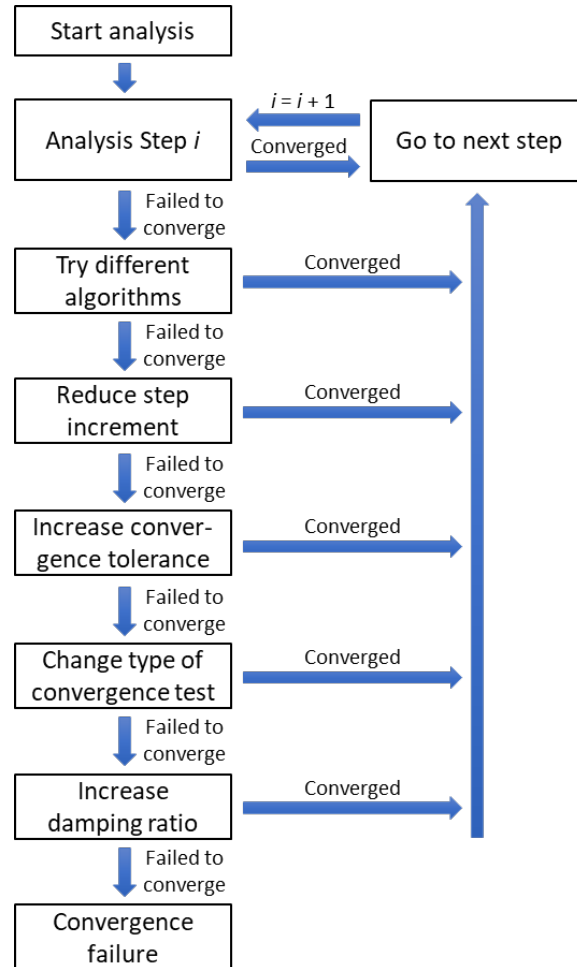


Figure 5-12 Flow chart of the algorithm for convergence tests

5.3.4. Processing of Analysis Results

A wide range of structural response quantities were obtained from the analyses, such as nodal displacements, element deformations, element forces, and reactions. These results have been post-processed to calculate other local deformation variables including story drift ratio and diaphragm shear angle (i.e. shear strain), which are described in this section.

5.3.4.1. Story Drift Ratio Calculation

For pushover analysis, roof drift ratio is defined as the applied displacement at the top of the building divided by the building height. For response history analysis, story drift ratio (SDR) at

any time in the record is determined for the x and y directions at each story, which is defined as the x and y relative displacements of any two nodes on the adjacent floors with the same x and y coordinates, divided by the story height. The resultant story drift ratio at any time in the record is calculated by taking the square root of the sum of the squares (SRSS) of the story drift ratios in the x and y directions at that time. The peak story drift ratio is then determined by the largest value of the resultant story drift ratio at any time during the motion and at any location of the building.

5.3.4.2. Diaphragm Shear Angle Calculation

Diaphragm shear angle (shear strain) is calculated at the center of each diaphragm unit, and is given by the following equation:

$$\gamma = \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \quad (5-1)$$

where u_x and u_y are the displacement at the center of diaphragm unit along x and y direction, respectively. u_x and u_y are obtained using piecewise finite element approximation:

$$u_x = \sum_{i=1}^4 N_i(x, y) u_{x,i} \quad (5-2)$$

$$u_y = \sum_{i=1}^4 N_i(x, y) u_{y,i} \quad (5-3)$$

where $N_i(x, y)$ are the shape functions given as follows:

$$N_1(x, y) = \frac{(x - x_2)(y - y_2)}{A} \quad (5-4)$$

$$N_2(x, y) = -\frac{(x - x_1)(y - y_2)}{A} \quad (5-5)$$

$$N_3(x, y) = \frac{(x - x_1)(y - y_1)}{A} \quad (5-6)$$

$$N_4(x, y) = -\frac{(x - x_2)(y - y_1)}{A} \quad (5-7)$$

$u_{x,i}$ and $u_{y,i}$ are the displacement along x and y direction, respectively, of the four nodes on the diaphragm unit, whose coordinates are given by: Node 1 (x_1, y_1), Node 2 (x_2, y_1), Node 3 (x_2, y_2), Node 4 (x_1, y_2). A is the area of the diaphragm unit.

5.3.5. Type of Analyses and Related Issues

For each of the archetype buildings considered in this study, modal analysis, nonlinear static pushover analysis, and nonlinear response history analyses were conducted to investigate the behavior of the buildings with different diaphragm design procedures. Some additional details of the analyses are provided in this section.

5.3.5.1. Modal Analysis

Modal analysis was performed for the archetype buildings in *OpenSees* to obtain their natural periods and mode shapes. Results were compared to structural models in a commercial structural analysis software, *SAP2000*, as discussed in the next chapter.

5.3.5.2. Nonlinear Static Pushover Analysis

Pushover analysis was conducted to study the static behavior of the archetype buildings. A displacement-controlled load pattern was applied to the structure in the short direction (long diaphragm span direction), where the displacement of the center node on the roof in the short direction controlled the solution. Per FEMA P695, vertical distribution of the lateral force at each node was assigned proportional to the product of the tributary mass and the fundamental mode shape coordinate at the node: $F_x \propto m_x \phi_{1,x}$, where F_x is the relative magnitude of force applied at node x , m_x is the mass associated with node x , and $\phi_{1,x}$ is the fundamental mode shape coordinate at node x . A view of the lateral force distribution on the 4-story archetype building is shown in Figure 5-13, in which the arrow length denotes the relative magnitude of the applied force.

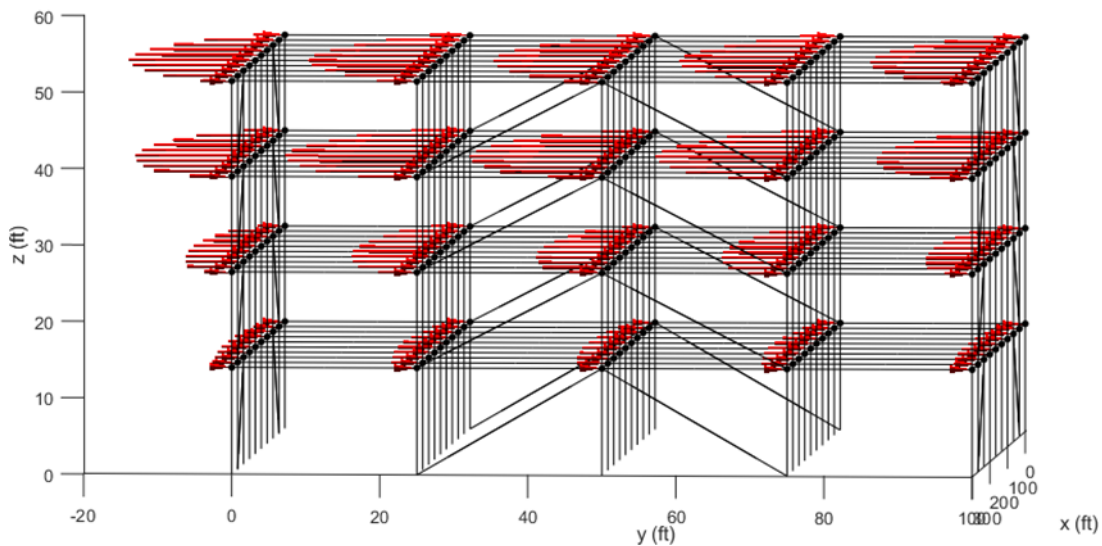


Figure 5-13 Lateral force distribution on 4-story archetype building for pushover analysis

5.3.5.3. Nonlinear Response History Analysis

To evaluate the seismic performance of the archetype buildings with different diaphragm design procedures, nonlinear response history analysis was performed with the archetype models subjected to the suite of FEMA P695 far-field earthquake motions. This section provides some details for the scaling of ground motion records to desired hazard levels and the criteria adopted to define building collapse.

5.3.5.3.1. Ground Motion Scaling

A total of 22 pairs of FEMA P695 far-field earthquake ground motions (44 records) were used in this study, which were applied in orthogonal directions of the building in the nonlinear response history analysis (two possible orientations of each pair resulted in 44 total sets of analysis for each archetype building model). Information of the 22 ground motion pairs is given in Table 5-10.

Table 5-10 Far-Field Ground Motions Used for Nonlinear Response History Analysis

ID No.	Earthquake			Recording Station	
	Magnitude	Year	Name	Name	Owner
1	6.7	1994	Northridge	Beverly Hills - Mulhol	USC
2	6.7	1994	Northridge	Canyon Country - WLC	USC
3	7.1	1999	Duzke, Turkey	Bolu	ERD
4	7.1	1999	Hector, Mine	Hector	SCSN
5	6.5	1979	Imperial Valley	Delta	UNAMUCSD
6	6.5	1979	Imperial Valley	El Centro Array #11	USGS
7	6.9	1995	Kobe, Japan	Nishi-Akashi	CUE
8	6.9	1995	Kobe, Japan	Shin-Osaka	CUE
9	7.5	1999	Kocaeli, Turkey	Duzce	ERD
10	7.5	1999	Kocaeli, Turkey	Arcelik	KOERI
11	7.3	1992	Landers	Yermo Fire Station	CDMG
12	7.3	1992	Landers	Coolwater	SCE
13	6.9	1989	Loma Prieta	Capitola	CDMG
14	6.9	1989	Loma Prieta	Gilroy Array #3	CDMG
15	7.4	1990	Mnajil, Iran	Abbar	BHRC
16	6.5	1987	Superstition Hills	El Centro Imp. Co.	CDMG
17	6.5	1987	Superstition Hills	Poe Road (temp)	USGS
18	7.0	1992	Cape Mendocino	Rio Dell Overpass	CDMG
19	7.6	1999	Chi-Chi, Taiwan	CHY101	CWB
20	7.6	1999	Chi-Chi, Taiwan	TCU045	CWB
21	6.6	1971	San Fernando	LA – Hollywood Star	CDMG
22	6.5	1976	Friuli, Italy	Tolmezzo	-

Three scale levels were considered for nonlinear response history analysis (NRHA): 1) design earthquake (DE), 2) maximum considered earthquake (MCE), and 3) a scale level based on adjusted collapse marginal ratio ($ACMR_{10\%}$, see FEMA P695). The third scale level was considered to evaluate the conformance of the archetype buildings with the acceptance criteria in FEMA P695, i.e., less than 50% of ground motions causing collapse implies conformance with the acceptance criteria.

In this study, Seismic Design Category (SDC) D_{max} from FEMA P695 was considered. The design spectral acceleration parameters, $S_{DS} = 1.0$, $S_{D1} = 0.6$, were used to create the target design earthquake (DE) spectrum. The maximum considered earthquake (MCE) spectrum was obtained using 1.5 times the S_{DS} and S_{D1} values. The third scale level ($ACMR_{10\%}$) is related to median collapse for acceptability according to FEMA P695.

The 44 ground motion records were scaled accordingly to each desired level in the nonlinear response history analysis. For DE and MCE, the ground motions were scaled such that the median spectrum matches the design spectrum at the fundamental period of the building (see Figure 5-14). To be consistent with FEMA P695 methodology, the value of the fundamental period for each archetype building was obtained by the product of the coefficient for upper limit on calculated period (C_u) and the approximate fundamental period (T_a) as defined in ASCE 7-16 Section 12.8.2, which is provided in Table 5-3. The scale factor for the third scale level ($ACMR_{10\%}$) was obtained with the method as described in Appendix F.3 of FEMA P695: first an acceptable value of adjusted collapse margin ratio ($ACMR_{10\%}$) was obtained with assumed total system collapse uncertainty; then the period-based ductility (μ_T) was obtained from the pushover analysis; and finally the spectral shape factor (SSF) and the collapse margin ratio (CMR) was obtained. The scale factor based on $ACMR_{10\%}$ was then obtained by multiplying the collapse marginal ratio by the scale factor for MCE. An example is given below based on 4-story building with traditional diaphragm design. The values for the other buildings are provided in Table 5-11.

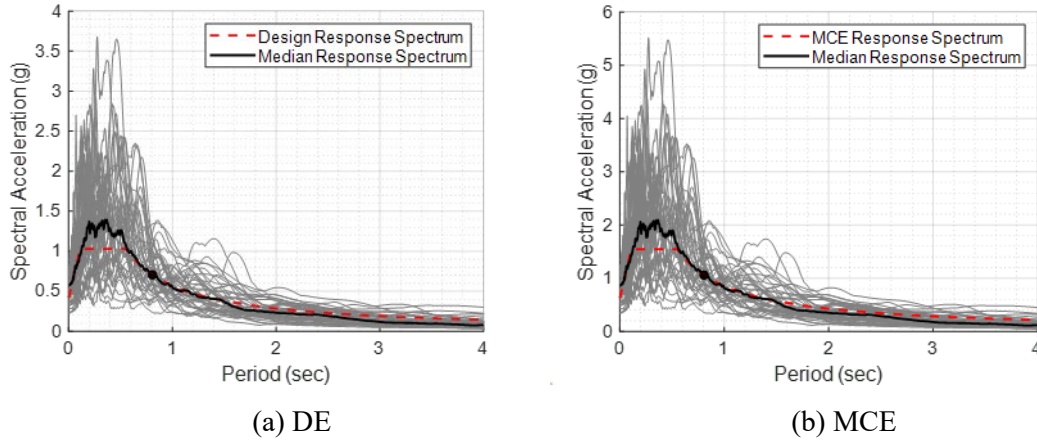


Figure 5-14 Example ground motion scaling for DE and MCE (4-story building)

Example calculation of $ACMR_{10\%}$ scale factor for 4-story archetype buildings with traditional diaphragm design:

1. Period-based ductility, μ_T , is obtained from the pushover analysis. Values of the coefficient C_0 , maximum base shear V_{max} , building weight W , fundamental period T (equal to $C_u T_a$), fundamental period obtained from modal analysis T_1 , effective yield roof drift displacement $\delta_{y,eff}$, ultimate roof drift displacement δ_u , and period-based ductility μ_T , are given as follow (see FEMA P695 for details):
 - a. $C_0 = 2.03$
 - b. $V_{max} = 1335$ kip
 - c. $W = 8930$ kip
 - d. $T = 0.81$ sec
 - e. $T_1 = 1.17$ sec
 - f. $\delta_{y,eff} = 4.06$ in.
 - g. $\delta_u = 43.29$ in.
 - h. $\mu_T = 10.65$
2. Assumed total system collapse uncertainty, $\beta_{TOT} = 0.525$, based on the following
 - a. Total system collapse uncertainty is calculated based on Equation 7-5 per FEMA P695:
 - b. $\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}$

where β_{TOT} = total system collapse uncertainty, β_{RTR} = record-to-record collapse uncertainty, β_{DR} = design requirements-related collapse uncertainty, β_{TD} = test data-related collapse uncertainty, β_{MDL} = modeling-related collapse uncertainty

- c. Assuming the quality ratings for design requirements, test data, and modeling are all Good, we have (Section 7.3.4): $\beta_{DR} = 0.20$, $\beta_{TD} = 0.20$, $\beta_{MDL} = 0.20$
 - d. β_{RTR} is a function of period-based ductility μ_T (Equation 7-2): $\beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.40$, But for $\mu_T \geq 3$, we have $\beta_{RTR} = 0.40$.
 - e. $\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} = \sqrt{0.40^2 + 0.20^2 + 0.20^2 + 0.20^2} = 0.525$ (rounded to the nearest 0.025). This value can also be obtained directly from Table 7-2b of FEMA P695.
3. Find acceptable level of ACMR: ACMR for 10%
 - a. Using Table 7-3 with $\beta_{TOT} = 0.525$, and 10% collapse probability
 - b. $ACMR_{10\%} = 1.96$
 4. Spectral shape factor, SSF
 - a. Table 7-1b of FEMA P695 is used to get SSF
 - b. Based on period, $T = C_u T_a = 0.81$ sec for BRBF building and period based ductility, $\mu_T = 10.65$
 - c. $SSF = 1.41$
 5. Find scale factor as scale factor for MCE multiplied by CMR
 - a. The scale factor is obtained using Equation G-1 of FEMA P695:

$$SF_{ACMR_{10\%}} = \frac{ACMR_{10\%}}{C_{3D} SSF} \left(\frac{S_{MT}}{S_{NRT}} \right)$$

$$SF_{ACMR_{10\%}} = \frac{ACMR_{10\%}}{C_{3D} SSF} SF_{MCE}$$

$$SF_{ACMR_{10\%}} = \frac{1.96}{1.2(1.41)} 2.5$$

$$SF_{ACMR_{10\%}} = 2.90$$

Table 5-11 Ground Motion Scaling for all Buildings

Building	$C_u T_a$ (sec)	T_1 (sec)	μ_T	β_{TOT}	$ACMR_{10\%}$	SSF	DE Scale Factor, SF_{DE}	MCE Scale Factor, SF_{MCE}	$ACMR_{10\%}$ Scale Factor, $SF_{ACRM_{10\%}}$
1-story (bare steel deck roof)	0.30	1.00	7.01	0.525	1.96	1.31	1.29	1.94	2.43
1-story (comp. deck roof)	0.30	0.81	28.18	0.525	1.96	1.33	1.29	1.94	2.39
4-story	0.81	1.17	10.66	0.525	1.96	1.41	1.67	2.50	2.90
8-story	1.34	2.06	5.21	0.525	1.96	1.43	1.67	2.50	2.86
12-story	1.81	2.77	3.51	0.525	1.96	1.36	1.99	2.98	3.58

5.3.5.3.2. Criteria for Collapse Definition

To perform statistical analysis on building collapse using the nonlinear response history analysis results, it is necessary to determine whether a ground motion caused building collapse based on selected criteria, including non-simulated collapse. For collapse definition in the response history analyses, the three criteria listed in the following were considered, and if any of them was satisfied, the building was considered as collapsed. It should be noted that some limit states such as BRB fracture were explicitly captured in the models and therefore not included in these criteria.

- 1) Peak resultant story drift ratio (as defined in Section 3.4.1) exceeds 10%. This limit is consistent with the evaluation of two-dimensional BRBF collapse performance by NIST (Kircher et al., 2010).
- 2) Maximum diaphragm shear angle exceeds 4%. This limit is determined based on the evaluation of the cantilever diaphragm test and connector test database, in which the majority of the specimens failed at an average shear angle equal to 4%.
- 3) Convergence failure occurs in the analysis. There are potentially many reasons for convergence failure during the analysis, and one of them is that large displacements cause local or global instability. For those runs of analysis that fail to converge, criteria 1) and 2) are first checked. If neither of these two criteria is met, the time history of story drift at the location where the maximum story drift occurs is examined on a case-by-case basis. Examples for determining the occurrence of building collapse in an individual analysis are provided as follows.

- i) If the building collapses under the same pair of ground motions with a smaller scale factor, then the building is considered collapsing and is included in the calculation of collapse ratio of all runs (with the reasoning that smaller magnitude of ground motions typically cause less damage to the building). Alternative, a run may be considered a collapse if the building undergoes substantial amount of inelastic deformation at the early stage of analysis (e.g. before the peak ground acceleration is applied). Figure 5-15 shows an example time history of maximum story drift for the runs of analysis with two different scale levels of ground motions. The analysis fails to converge for ACMR_{10%}-level ground motions. However, because the building is considered to collapse for the analysis with the same pair of ground motions at MCE level (the story drift ratio exceeds 10%), it is also considered to collapse for the ACMR_{10%} level since the ground motions are scaled to a higher hazard level.

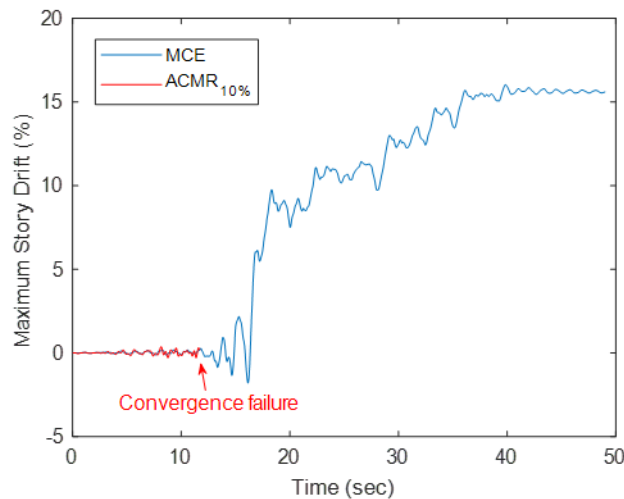


Figure 5-15 Example time history of maximum story drift for analysis with convergence failure considered as building collapse (4-story Trad. / Alt.2, Ground Motion Set 21)

- ii) If the building does not collapse under the same pair of ground motions with a larger scale factor, then the building is considered non-collapsing and is included in the calculation of collapse ratio of all runs. The reasoning is that smaller magnitude of ground motions typically cause less damage to the building.
- iii) If it cannot be determined whether the building collapses or not, the run is excluded from the calculation of collapse ratio of all runs. This happens if neither i) nor ii) is satisfied. In

this case, the analysis is considered incomplete, and is deemed inappropriate to be included in the calculation of collapse ratio. An example is shown in Figure 5-16.

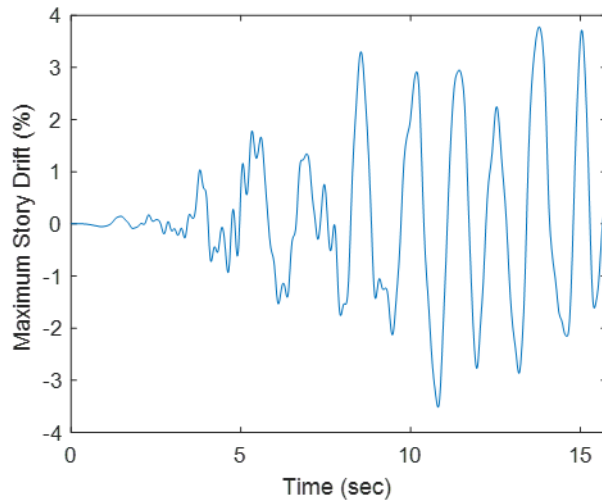


Figure 5-16 Example time history of maximum story drift for analysis with convergence failure excluded from collapse ratio calculation (8-story Trad. / Alt.2, Ground Motion Set 26)

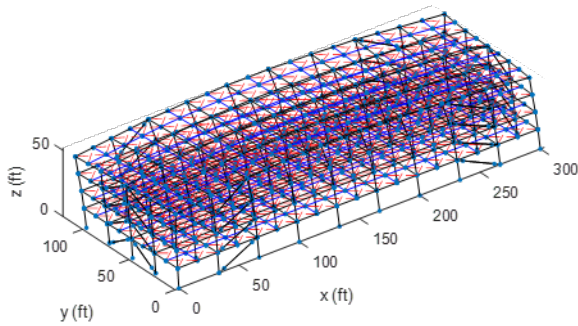
5.4. Results and Discussion

5.4.1. Eigenvalue Analysis

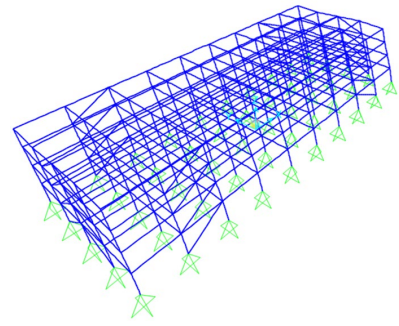
Eigenvalue analysis was performed for the archetype buildings to obtain their natural periods and mode shapes. To study the effect of the rigid diaphragm assumption on modal properties of the building structure, linear elastic models were also created using the commercial structural analysis program *SAP2000* for the building framing members using rigid diaphragm constraints. Table 5-12 provides the 1st and 2nd periods of the archetype buildings obtained from eigenvalue analysis of the models in *OpenSees* that uses an elastic diaphragm and *SAP2000* that uses a rigid diaphragm. The 1-story archetype building with bare steel deck roof has more flexible diaphragm, so the fundamental period is most affected by the rigid diaphragm assumption in SAP model. Other archetype buildings have concrete on steel deck typical floor diaphragms (more rigid) so the periods are less affected. Figure 5-17 shows the mode shape for the 1st mode of the four-story archetype models. It can be observed that diaphragm deflections can have a substantial effect on building natural period (up to 48% larger than rigid) and on the mode shape shown in Figure 5-17a which shows potential for a “whipping effect” at the roof due to roof diaphragm flexibility. Different mode shapes of the 4-story archetype building are also shown in Figure 5-18.

Table 5-12 Natural Periods of Archetype Models in OpenSees and SAP2000

Building Model	Long Dimension Mode 1 (sec)			Short Dimension Mode 1 (sec)		
	Elastic Diaphragm (OpenSees)	Rigid Diaphragm (SAP)	Difference	Elastic Diaphragm (OpenSees)	Rigid Diaphragm (SAP)	Difference
1-story (bare steel deck roof)	0.61	0.46	25%	1.00	0.52	48%
1-story (concrete on steel deck roof)	0.81	0.53	35%	0.61	0.41	33%
4-story	1.17	0.94	20%	1.17	0.76	35%
8-story	2.61	1.79	31%	1.84	1.40	24%
12-story	2.77	2.41	13%	2.38	1.91	20%

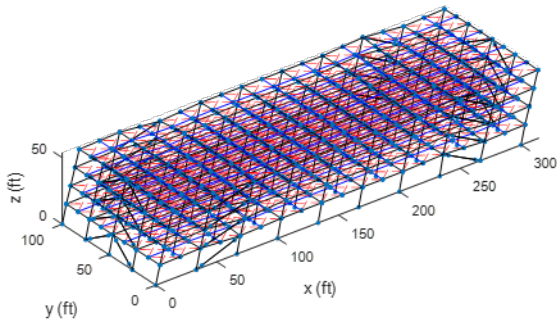


(a) OpenSees model

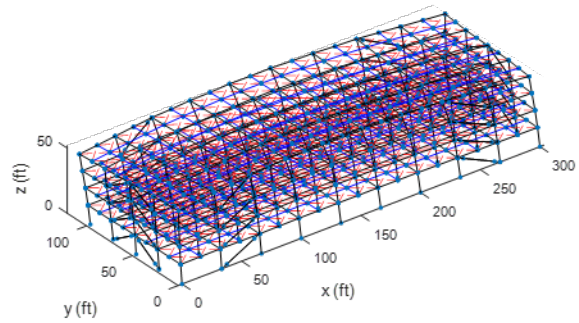


(b) SAP2000 model

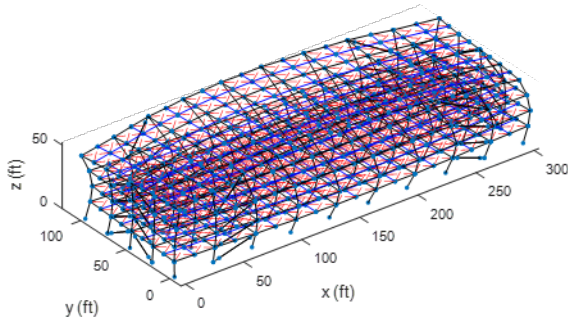
Figure 5-17 Mode shapes for the 1st mode of four-story archetype models



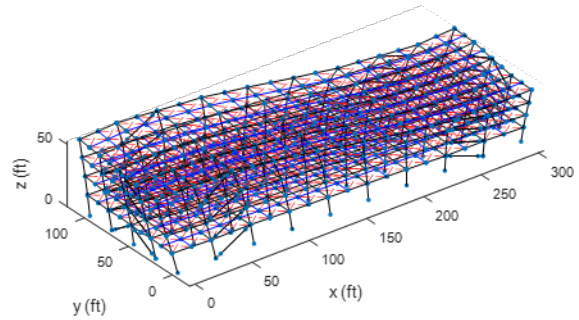
(a) Long dimension Mode 1 ($T = 1.17$ sec)



(b) Short dimension Mode 1 ($T = 1.17$ sec)



(c) Short dimension Mode 2 ($T = 0.74$ sec)



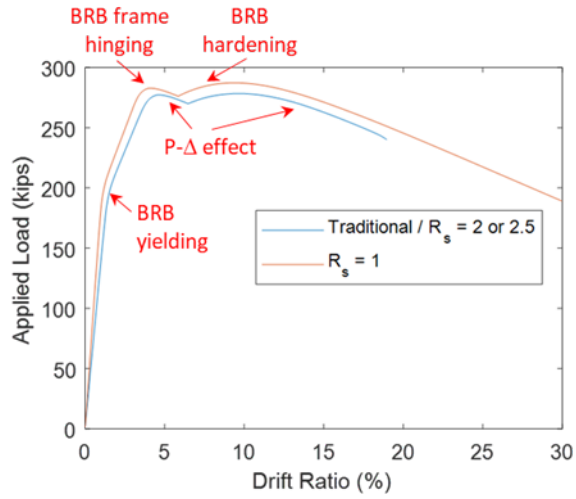
(d) Torsional mode ($T = 0.61$ sec)

Figure 5-18 Mode shapes of four-story archetype models

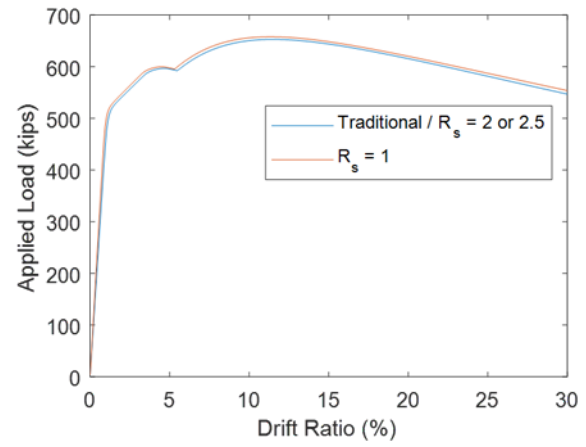
5.4.2. Nonlinear Static Pushover Analysis

Pushover analysis was conducted to study the static behavior of the archetype buildings. Figure 5-19 shows the pushover curves of the archetype buildings with different diaphragm designs. The drift ratio was calculated as the applied displacement at the center of the roof divided by the building height. It can be observed that the different diaphragm designs had little effect on the pushover behavior because the pushover analyses were dominated by BRB inelasticity. The load pattern was based on the first mode shape and for these buildings resulted in BRB yielding instead of diaphragm inelasticity. The first point of nonlinearity on the pushover curves is associated with yielding of the BRB cores, followed by a hardening segment with reduced slope, which is related to the stiffness provided by the BRB frames before hinging occurs at the beam-to-column connections. This is more pronounced for the 1-story and 4-story buildings. Once the beam-to-column connections develop plastic hinges, softening response occurs due to P- Δ effect (note that the analysis for 4-story building with Traditional / Alternative 2 diaphragm design failed to converge before secondary softening occurred). Secondary hardening is not observed for the 8 and 12-story models where P- Δ effect controls over the BRB material hardening after the peak load.

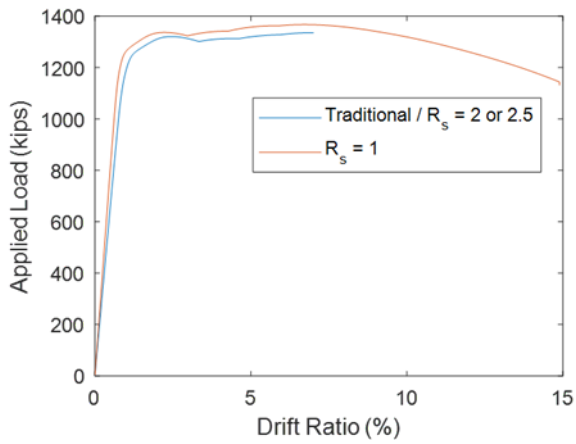
The deformed shapes of the building models at the end of the pushover analysis are shown in Figure 5-20. As the behavior of the buildings were dominated by the BRB inelasticity, diaphragm deformation was not observable in the deformed shape, while the story drift of the buildings primarily concentrated in the first several stories from the ground level.



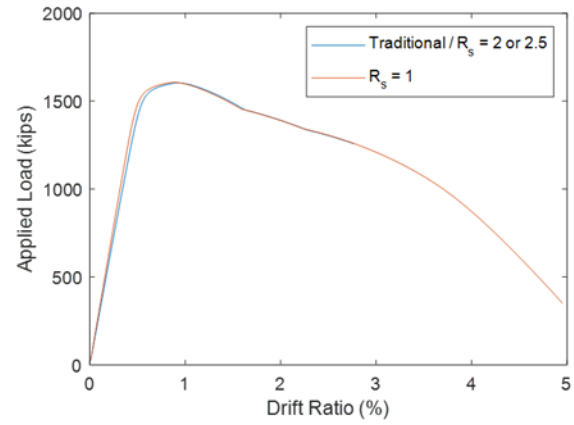
(a) 1-story with bare steel deck roof



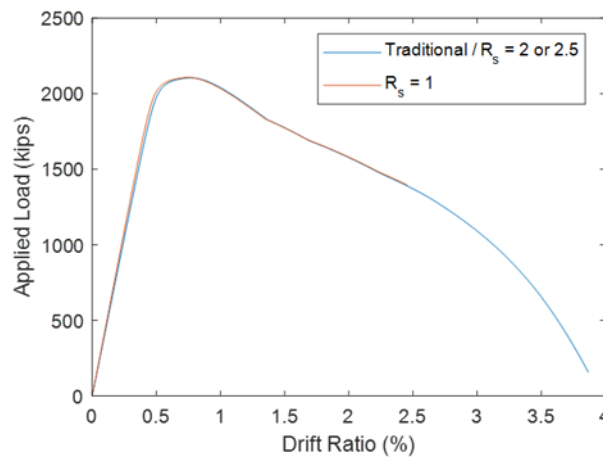
(b) 1-story with composite concrete on steel deck roof



(c) 4-story

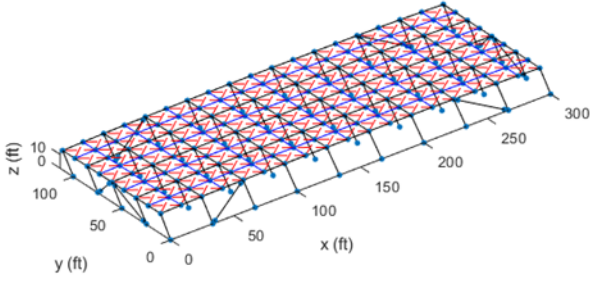


(d) 8-story

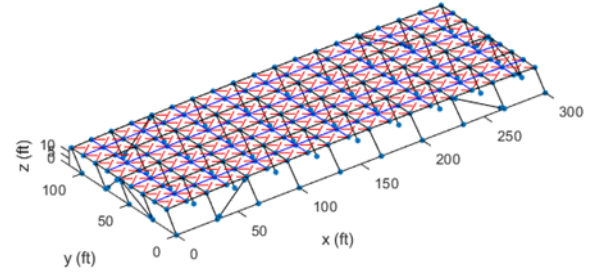


(e) 12-story

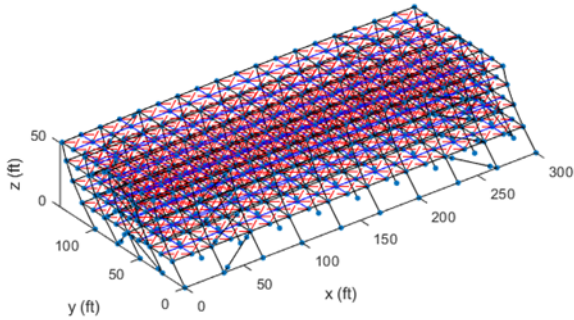
Figure 5-19 Pushover curves of archetype buildings with different diaphragm design procedures



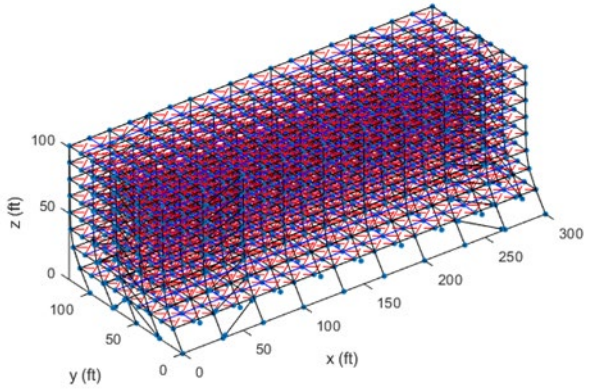
(a) 1-story (bare steel deck roof)



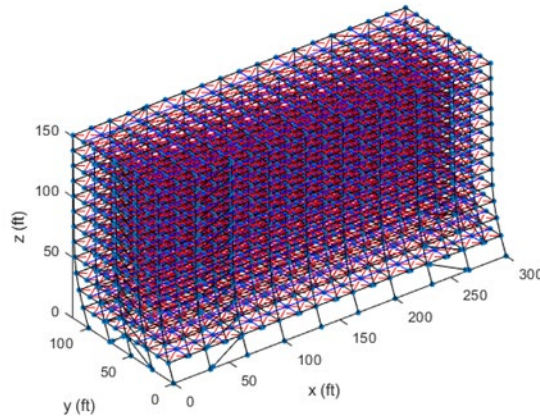
(b) 1-story (composite concrete on steel deck roof)



(c) 4-story



(d) 8-story



(e) 12-story

Figure 5-20 Deformed shapes of archetype buildings with Trad. or Alt. 2 diaphragm design procedures (deformation amplification factor: 5)

5.4.3. Nonlinear Response History Analysis

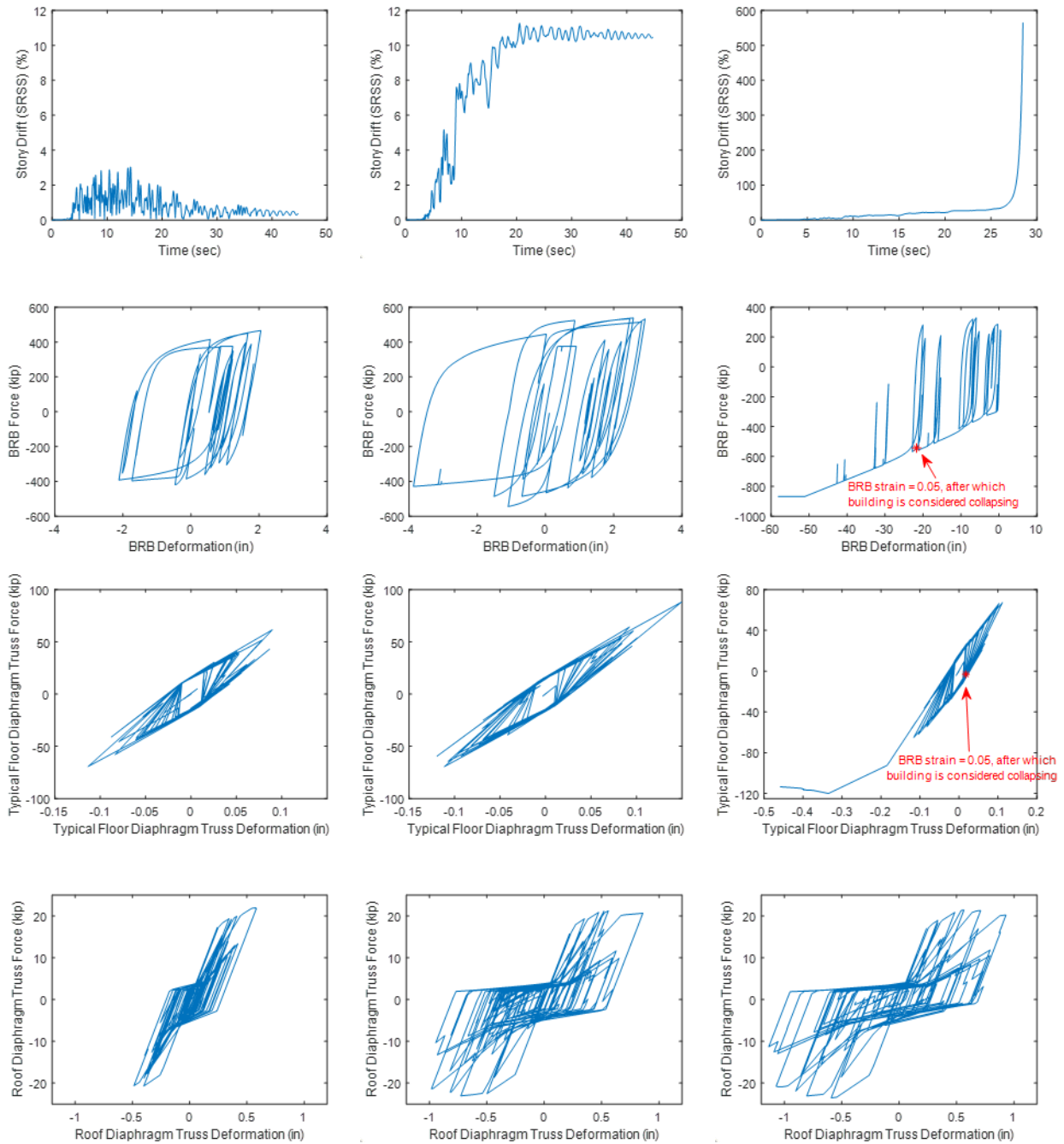
To evaluate the seismic performance of the archetype buildings with different diaphragm design procedures, nonlinear response history analysis was performed with the archetype models

subjected to the suite of FEMA P695 far-field earthquake motions scaled to different hazard levels. Results of the analysis is presented in this section.

5.4.3.1. Detailed Investigation of 4-story Building Behavior Subjected to One Ground Motion Pair

This section provides a detailed investigation of building behavior in the nonlinear response history analysis using a single building height subjected to one ground motion pair. The 4-story archetype building model with different diaphragm designs subjected to the ground motion with ID No. 7 in Table 5-10 at different earthquake hazard levels was selected.

Figure 5-21 shows response history results including peak story drift (at the location where the maximum story drift ratio occurred), BRB hysteresis (at the location where the maximum BRB force occurred), and diaphragm truss hysteresis (at the location where the maximum diaphragm shear angle occurred) of the buildings with Traditional / Alternative 2 diaphragm design. While the peak story drift of the building subjected to DE-level ground motion is less than 3%, the MCE-level ground motion produces peak story drift larger than the 10% limit for collapse definition, and under $ACMR_{10\%}$ -level ground motion the building experiences ever increasing story drifts, which indicates building collapse. The BRB's and diaphragms both undergo inelastic deformation at all three hazard levels. For BRB's, the hysteresis curves show that energy is dissipated by the BRB inelastic deformation, and at the $ACMR_{10\%}$ level, excessive BRB deformation occurs and causes the building to collapse. Floor diaphragms remain relatively elastic compared to the roof diaphragms under the DE and MCE-level ground motions, whereas at the $ACMR_{10\%}$ level, the floor diaphragms are affected by the large story drift due to the excessive deformation of the BRB and also undergo large deformation.



(a) DE

(b) MCE

(c) ACRM_{10%}

Figure 5-21 Time history response of 4-story building with the Traditional / Alternative 2 diaphragm design under three levels of ground motions (from top to bottom: peak story drift, base story BRB hysteresis, floor diaphragm truss hysteresis, roof diaphragm truss hysteresis)

Figure 5-22 shows the deformed shapes of the building under the three levels of the ground motion (plotted at the moment in the time history when peak story drift occurs). The deformed shapes further illustrate the cause of building collapse at the MCE and $ACMR_{10\%}$ -level ground motions which is failure of BRB's, particularly at the first story where story drifts concentrate. In addition, unlike the first-mode based pushover analysis in which inelasticity focuses in the BRB's, the participation of higher modes in the response history analysis leads to diaphragm inelasticity. The total story drifts include inelastic deformations in the vertical LFRS and the diaphragm such that the two compound each other (i.e. interact) to exacerbate the P- Δ effect which eventually leads to the collapse of the buildings.

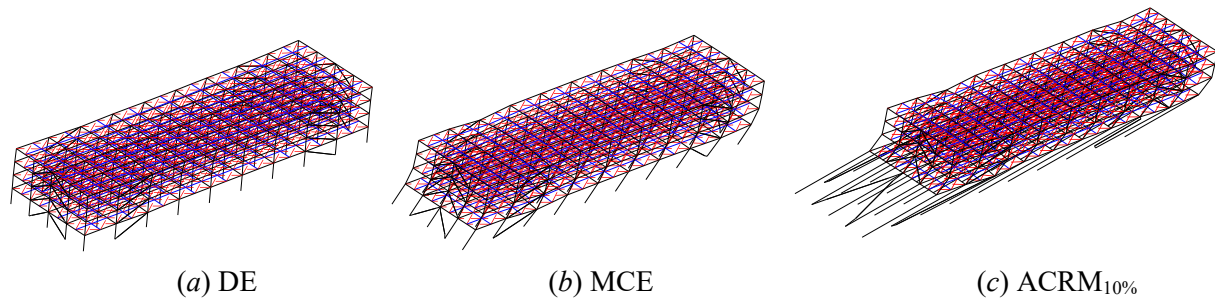


Figure 5-22 Deformed shapes of 4-story archetype building with the Traditional / Alternative 2 diaphragm design under three levels of ground motions (deformation amplification factor: 10)

Figure 5-23 shows the time history of the maximum total story drift (at any location of the building including diaphragm deformation) and the maximum story drift at the BRB frames plotted separately for the x (along the longer dimension of the building) and y (along the shorter dimension of the building) directions of the archetype building with Traditional / Alternative 2 diaphragm design subjected to the ground motion at MCE level. It can be observed that the building experiences larger story drift in the x direction than in the y direction, possibly because the magnitude of ground motion accelerations is larger in the x direction than in the y direction. Also, the total story drift in the x direction is very close to the story drift at the BRB frames throughout the time history, indicating negligible in-plane diaphragm deformation in this direction, which is due to the large in-plane stiffness of the diaphragm along the longer dimension of the building. However, the in-plane stiffness of the diaphragm in the y direction is much smaller, resulting in significantly larger in-plane diaphragm deformations and thus the total story drift is substantially larger than the story drift at the BRB frames in this direction. This is worth some attention as in

conventional structural analysis where diaphragms are assumed infinitely rigid or elastic in plane with zero or small deformation, the story drift of the building could be underestimated.

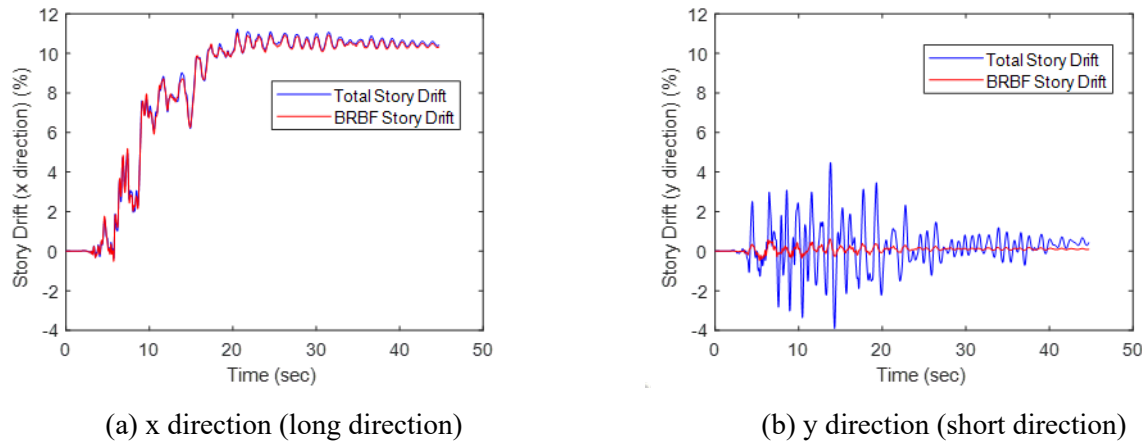


Figure 5-23 Time history of peak story drift in x and y directions of 4-story building with Traditional / Alternative 2 diaphragm design under MCE-level ground motion: total story drift vs. BRBF story drift

Figure 5-24 shows the time history of the maximum total base shear (including the shear in the columns at the base story) and the maximum base shear at the BRB frames plotted separately for the x and y directions of the 4-story archetype building with Traditional / Alternative 2 diaphragm design subjected to the ground motion at DE and MCE levels. The peak values of these base shears are provided in Table 5-13. It can be observed that although the scale factor for MCE ground motion accelerations is 1.5 times larger than that for DE ground motions, the peak base shear is an average of 1.2 times larger for MCE compared to DE because the BRBF strength limits the force that can transfer through the vertical LFRS. It is also noted that the peak total base shear in the y direction is close to the peak base shear at the BRB frames, while in the x direction these two quantities are approximately 40% different, with the peak total base shear being smaller than the peak base shear at the BRB frames. The main reason is that the P- Δ effect causes shear at the base of gravity columns that acts in the direction opposite to the BRBF base shear. From Figure 5-23 it is shown that the story drift in the x direction is much larger than that in the y direction, and therefore the base shears are more affected by the P- Δ effect, leading to larger difference between the total base shear and BRB frame base shear as shown in Figure 5-24 and Table 5-13.

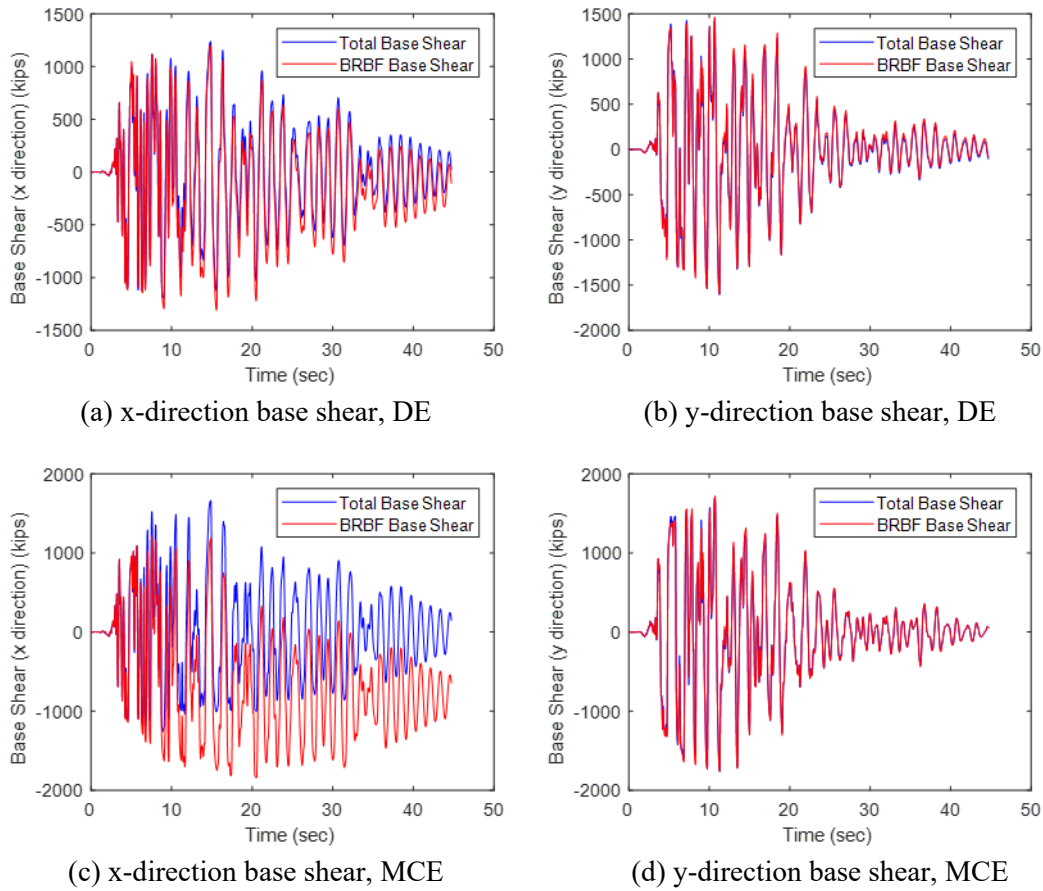


Figure 5-24 Example time history of base shear in x and y directions of 4-story building with Traditional / Alternative 2 design under DE and MCE-level ground motions: total base shear vs. BRBF base shear

Table 5-13 Base Shear of 4-story Archetype Building with Traditional / Alternative 2 Diaphragm Design under DE and MCE-level Ground Motions

Ground motion scale	Peak total base shear in x direction (kip)	Peak BRBF base shear in x direction (kip)	Peak total base shear in y direction (kip)	Peak BRBF base shear in y direction (kip)
DE	1238	1309	1607	1600
MCE	1664	1816	1762	1760

Figure 5-25 shows the total base shear vs. story drift (of the location where peak story drift occurs) hysteretic curves of the building under the MCE-level ground motion. It is noted that the peak base shear typically does not occur at the same time as the peak story drift, which are both selected for further investigation.

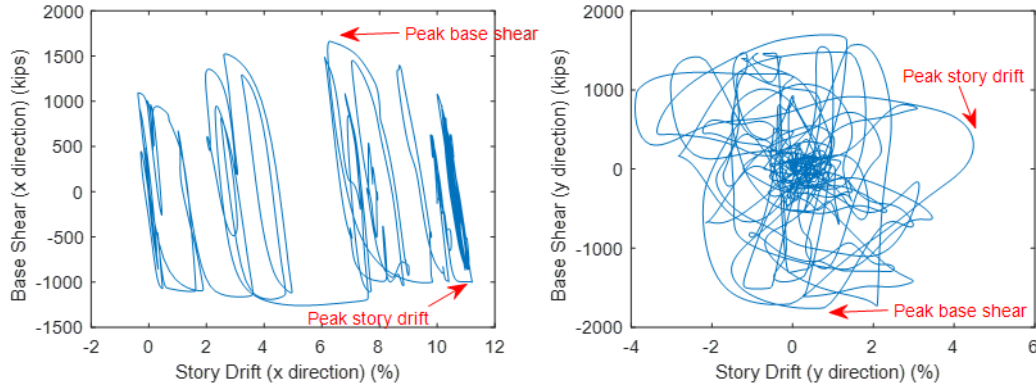


Figure 5-25 Base shear vs. story drift hysteretic curves of 4-story building with Traditional / Alternative 2 diaphragm design under MCE-level ground motion

Figure 5-26 shows the contour of the normalized shear angles of the diaphragm units and the normalized strain of the BRB's plotted at different moments in the time history, i.e., at peak story drift and peak base shear in the x and y directions. The y-direction displacement appears small in the contour plotted at the peak story drift because the story drifts in the y direction are much smaller than those in the x direction, as shown in Figure 5-23. The diaphragm shear angles (γ_{diaph}) are normalized by γ_1 , which is the shear angle reached when the diagonal trusses of the diaphragm unit undergo an axial strain equal to ϵ_1 of the Pinching4 parameters given in Table 5-5, i.e., the elastic regime. The normalized strain demand of each BRB is obtained by dividing the BRB strain (ϵ_{BRB}) by ϵ_y , which is the yield strain of the BRB given by $\epsilon_y = F_y/E$ where F_y is the yield stress of the BRB and E is the elastic modulus of steel. The normalization is done such that the contours provide a visualization of the inelasticity distribution for the horizontal and vertical systems. It can be observed from Figure 5-26 that the diaphragm deformation is relatively small at the moment when the peak story drift or the peak base shear in the x direction is reached, but almost all the BRB's parallel to the x-z plane have yielded. However, at the moment when the peak story drift or peak base shear in the y direction occurs, there is significant inelastic deformation in the roof diaphragm and it is concentrated at its two edges where the shear demand is largest.

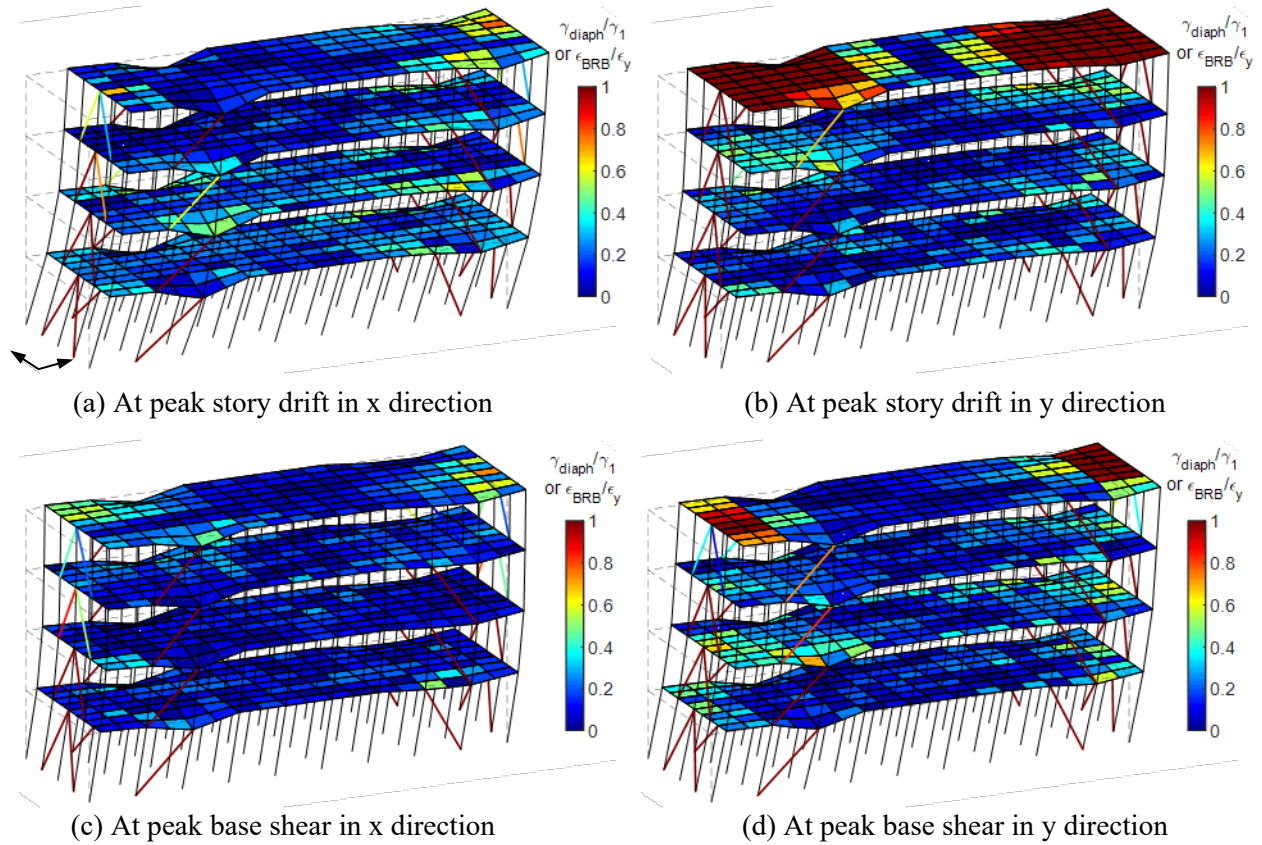


Figure 5-26 Contour of normalized diaphragm shear angle and normalized BRB strain of 4-story building with Traditional / Alternative 2 diaphragm designs under MCE-level ground motion

To illustrate the deformation demands for buildings with different diaphragm designs, the contour of normalized diaphragm shear angle demand and BRB strain demand are plotted in Figure 5-27. The diaphragm shear angles and BRB strains are normalized as described previously for Figure 5-26, but in this plot, the maximum deformation demands at any time during the record are used. It can be observed from Figure 5-27 that in each of the four cases, all the BRB's experienced inelastic deformation. As expected, the diaphragm shear angle demand of the building with Traditional / Alternative 2 diaphragm design was larger than that of the building with Alternative 1 design. Inelastic deformation occurred in the bare steel deck roof diaphragm in each of the four cases, while the composite concrete on steel deck floor diaphragms stayed mostly elastic except that at the MCE level, the floor diaphragms with Traditional / Alternative 2 design exhibited some inelastic deformation demand.

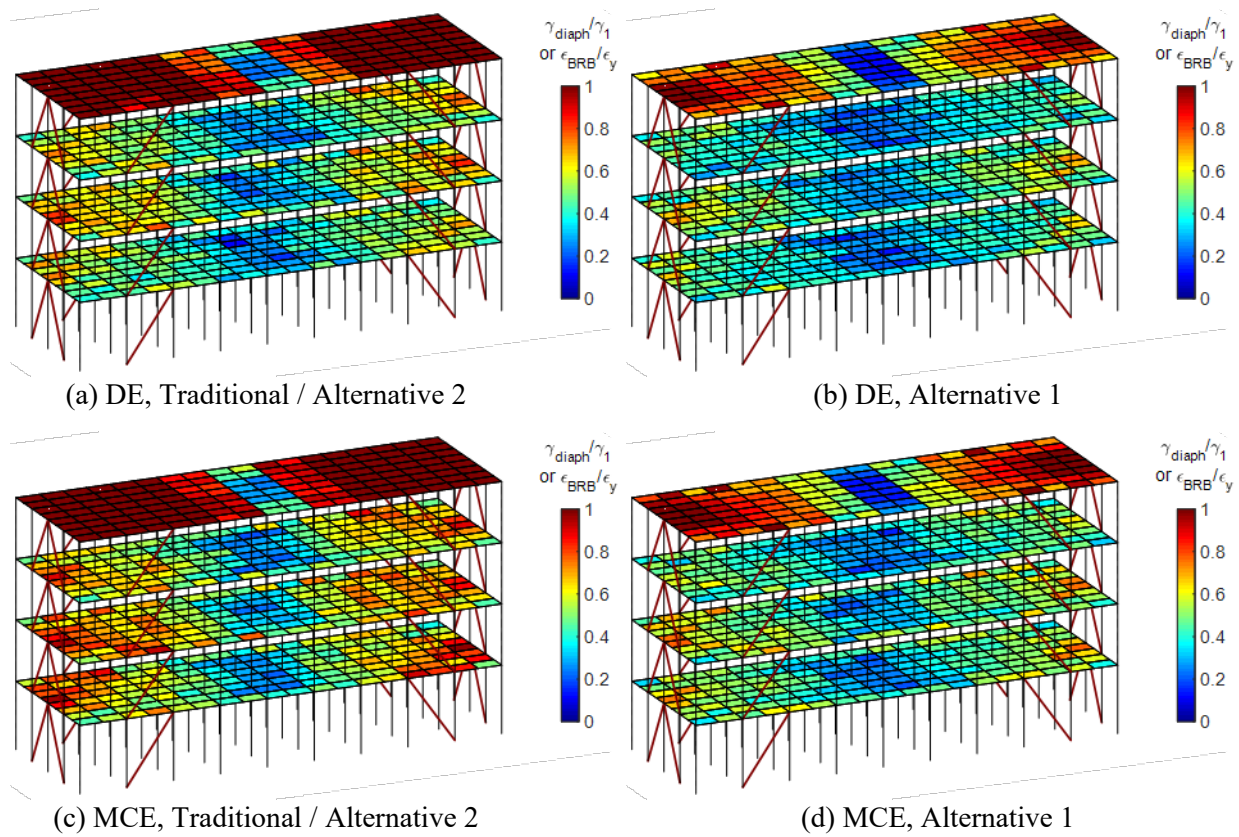


Figure 5-27 Contour of normalized diaphragm shear angle demand and normalized BRB strain demand of 4-story building with different diaphragm designs under DE and MCE-level ground motions

5.4.3.2. Statistical Results and Discussion of All Archetype Buildings

After the results of the nonlinear response history analysis were collected, statistical analysis was performed to investigate the overall seismic behavior and performance of the archetype buildings. Results are provided and discussed in this section.

5.4.3.2.1. Story Drift

Figure 5-28 shows an example of the distribution of median peak story drifts at each story along the building height for the 12-story archetype buildings with Traditional / Alternative 2 diaphragm design. The medians of peak story drifts across the 44 runs of analyses was found for each story in the x and y directions for the BRBF frame and the total BRBF plus diaphragm deflection, and for the resultant story drift. One can tell how many cases of the ground motions cause building collapse based on the story drift criterion by counting the number of curves hitting the 10% story drift limit.

It is noted from Figure 5-28 that the median resultant story drift is larger than the median story drift in the x or y direction alone, especially at the first story where the difference can range approximately from 50% to 100%. When examining x and y story drifts separately, results are similar to planar frame analysis with results close to what can be expected based on the story drift limit per ASCE 7, which is 2% at DE and 3% at MCE. However, the resultant story drift is larger leading to median story drift as large as 3% at DE and 6% at MCE. This indicates that analysis of 2D frames can substantially underestimate peak story drifts.

Since the P- Δ effect is controlled by the story drift in any direction, the resultant story drift is a better estimate of story drift contributing to the P- Δ effect than the x or y-direction story drift considered alone which is typically used in conventional frame analysis. This deserves some attention as there is concentrated story drift at the base story where the gravity load is the largest and thus the P- Δ effects are also the worst. Above the first story along the building height, the story drifts are more uniformly distributed with a smaller magnitude in the intermediate stories, while the story drifts near the roof become larger due to the participation of higher modes. For the BRB frames, the story drifts at the BRB frames in the y direction are typically smaller than those in the x direction. This can be explained by the fact that on one hand, the in-plane stiffness of the diaphragms in the x direction is much larger than in the y direction, forcing more inelasticity to occur in the BRB frames, and on the other hand, the more flexible diaphragms in the y direction can dissipate more energy through inelastic deformation, which reduces the story drifts of the BRB frames in this direction. It can also be observed that because of the different in-plane stiffness of the diaphragms in different directions, the peak total story drifts considering diaphragm deformation are up to 80% larger than the peak story drifts at the BRB frames in the y direction, while in the x direction these two are very close to each other (up to 11% difference), indicating much diaphragm deformation in the y direction and little in the x direction. For the same reason, the higher mode effect is more pronounced and causes a much larger difference between the total story drifts and the BRB frame story drifts near the roof.

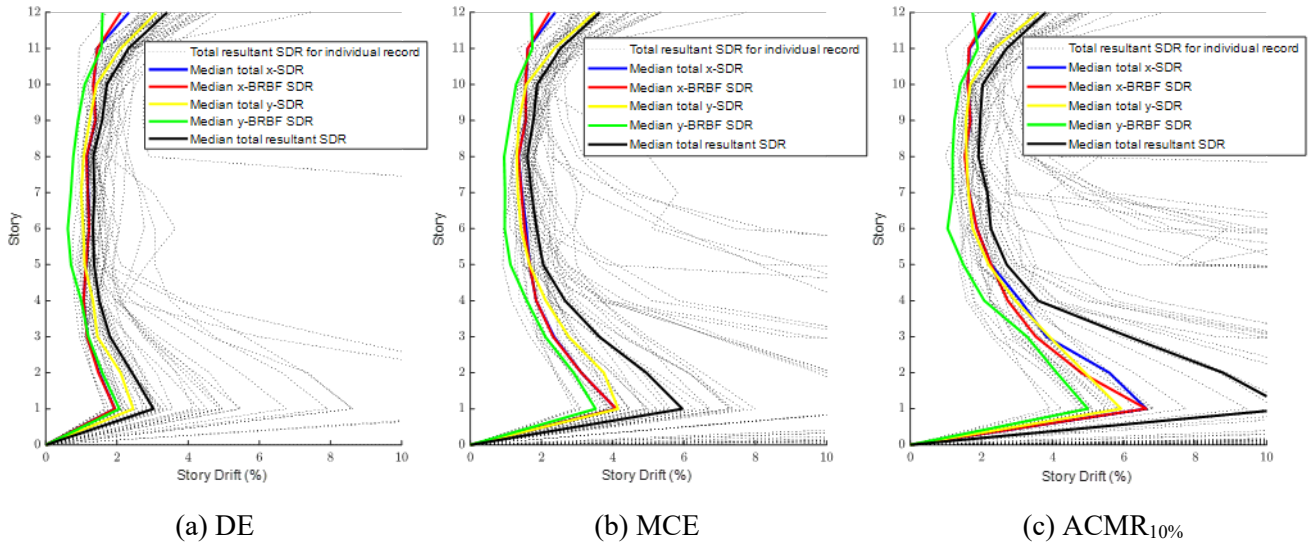


Figure 5-28 Distribution of median peak story drifts at each story along building height of 12-story archetype buildings with Trad. or Alt. 2 design under three levels of ground motions

Figure 5-29, Figure 5-30 and Figure 5-31 show the distribution of median peak resultant story drift, median peak story drift in the x direction, and the median peak story drift in the y direction along the building height, respectively, for all the archetype buildings under the three levels of ground motions. Values for these quantities are provided in Table A-5 of Appendix A. A similar pattern is observed for the distribution of peak story drift for all the buildings, with larger story drift at the first story, more uniform and smaller story drift at the intermediate stories, and larger story drift near the roof. Due to the 3D effect of the analysis, the median peak resultant story drifts range from 3% for buildings under DE-level ground motions, to approximately 10% for buildings under ACMR_{10%}-level ground motions. If peak story drifts are considered for x or y direction separately, they are reasonably smaller and comparable to results from conventional 2D frame analysis (Chen, 2010; Özkılıç et al., 2018), ranging from slightly larger than 2% for buildings under DE-level ground motions to approximately 6% for buildings under ACMR_{10%}-level ground motions. It can also be observed that different diaphragm design procedures for the archetype buildings do not affect the median peak story drifts much (with an average difference equal to 6%), mainly because the story drifts of the buildings are dominated by the BRB behavior. Another observation is that the median peak story drift at the first story remains similar for all the archetype buildings except for the 12-story building under ACMR_{10%}-level ground motions.

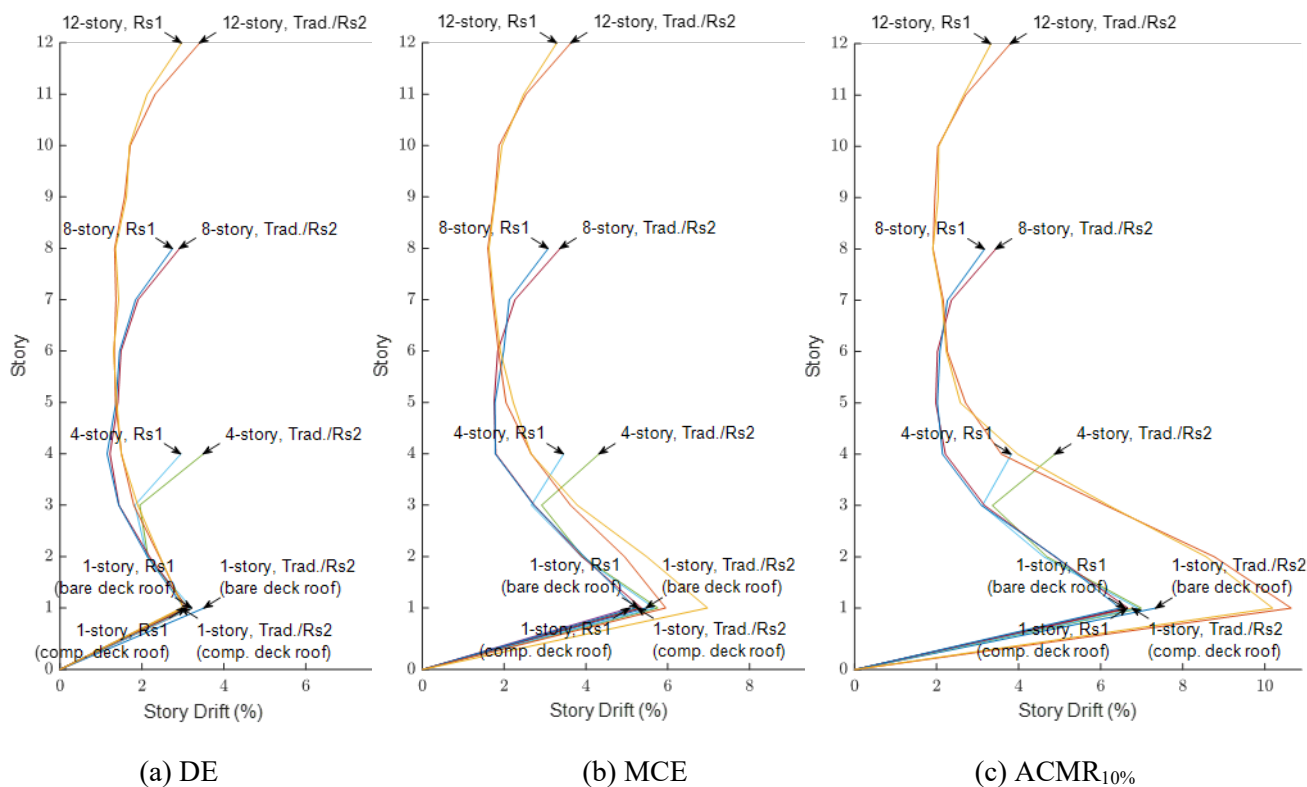


Figure 5-29 Distribution of median peak resultant story drift along building height

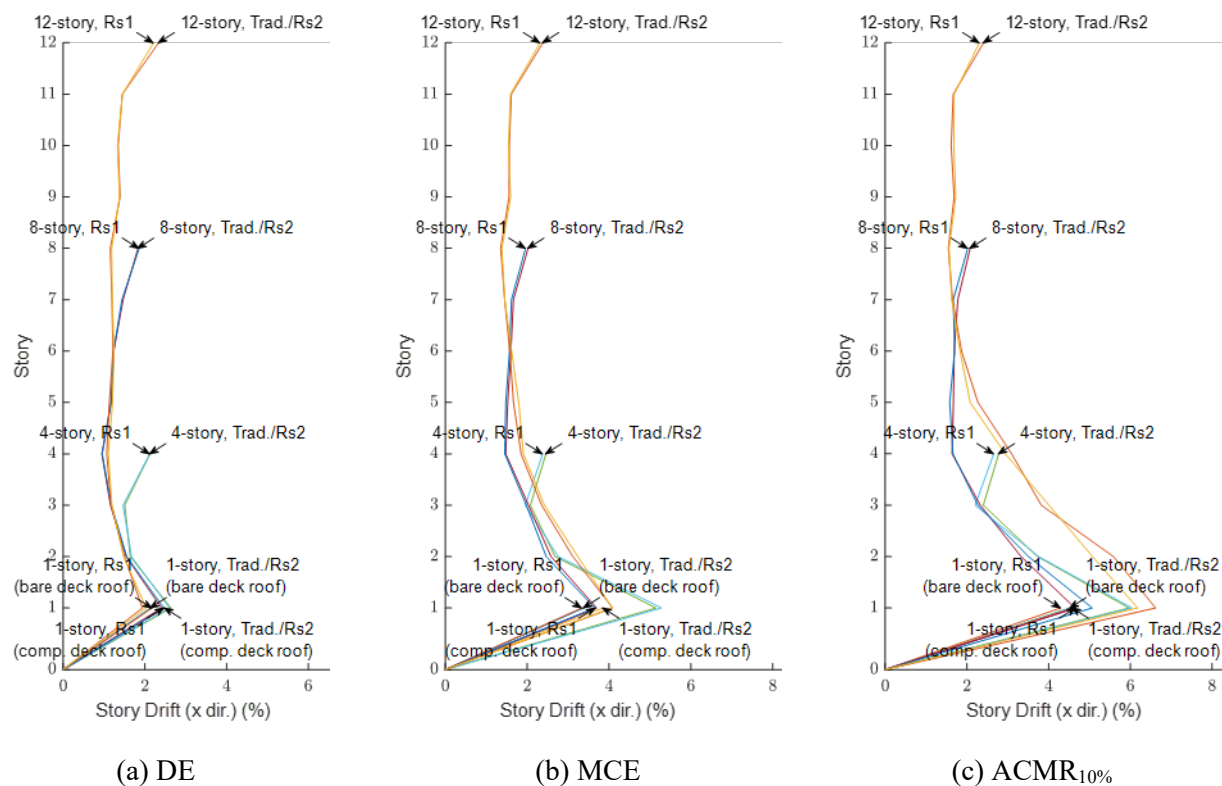


Figure 5-30 Distribution of median peak story drift in x direction along building height

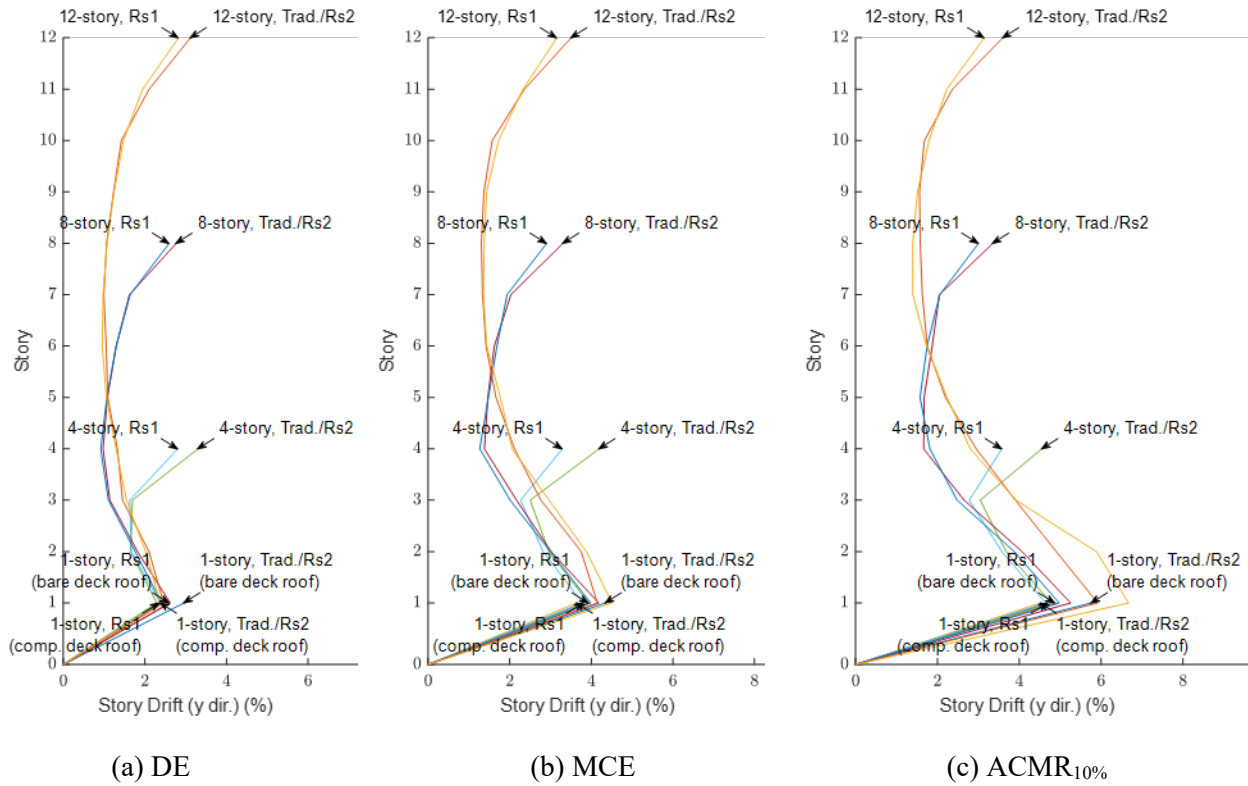


Figure 5-31 Distribution of median peak story drift in y direction along building height

5.4.3.2.2. Elastic Diaphragm Shear

To evaluate the accuracy of elastic diaphragm forces from alternative diaphragm design procedure of ASCE 7-16 Section 12.10.3, the diaphragm shear was obtained from the analyses with Alternative 1 diaphragm design procedure (where $R_s = 1.0$ implies diaphragms should remain elastic). Specifically, the medians of the diaphragm shear demand at the edges, F_{peak} , as calculated by the maximum value of the sum of diaphragm shear along the two edges (x or y direction) in the records, were obtained from the analysis results and provided in Table 5-14 for each story of the 4-story archetype building. Values for other archetype buildings are given in Table A-6 of Appendix A. These values can be viewed as the median peak inertial forces of the diaphragms. As is shown by the contour of diaphragm deformation demand in Figure 5-27, the diaphragms of the 4-story building with Alternative 1 diaphragm design remained almost entirely elastic under the DE-level ground motions, and therefore the diaphragm shear demands should be comparable to the elastic design shear for diaphragms F_{px} calculated using the alternative diaphragm design procedures. It is observed that ratios of the elastic diaphragm shear demand obtained from the analysis to the design shear given by the Alternative 1 diaphragm design procedures are relatively

close to 1.0, indicating a reasonable accuracy of the prediction of elastic diaphragm shear demand with the design approach. This can be further validated by the average value of $(F_{peak})_{DE}/F_{px}$ equal to 0.84 for all the archetype buildings. These ratios are shown in Figure 5-32, and one can also observe that the ratio for the roof diaphragm is the largest among the building stories and in some cases is slightly greater than 1.0 (e.g., 1.06 and 1.03 for the roof of 4-story and 12-story building, respectively). It is therefore concluded that the alternative diaphragm design procedure in ASCE 7-16 produced elastic diaphragm design forces that were somewhat conservative on average, but slightly unconservative at the roof for these archetype buildings with flexible roof diaphragms.

Table 5-14 Medians of Diaphragm Shear Demand for 4-story Archetype Buildings and Comparison to Design Shear

Diaphragm Design	Story	Median of F_{peak} at DE (kip)			F_{px} (kip)	$\frac{(F_{peak})_{DE}}{F_{px}}$
		x	y	x or y		
Alt. 1	1	746	655	816	977	0.83
	2	662	773	796	914	0.87
	3	613	629	727	850	0.86
	4	352	441	442	419	1.06

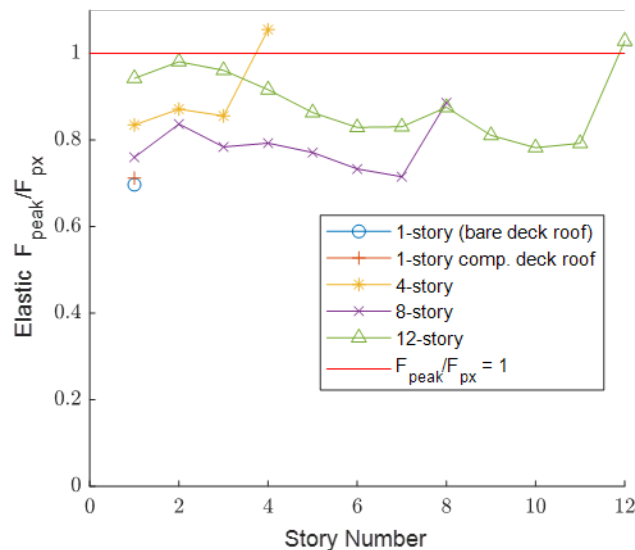


Figure 5-32 Diaphragm shear demand of archetype buildings with Alternative 1 diaphragm design normalized by diaphragm design shear

5.4.3.2.3. Collapse Ratio

To investigate the seismic performance objective related to collapse prevention, collapse ratios, i.e., percentage of ground motions causing collapse, were calculated for each set of nonlinear response history analyses. The criteria for defining collapse was given in Section 5.3.5.3.2. It should be noted that based on collapse criteria item 3) in Section 5.3.5.3.2, 4 out of 440 runs at DE and 1 out of 440 runs at ACMR_{10%} with convergence failure in the analysis were excluded from the collapse ratio calculation. The number of analysis runs that are included in each collapse ratio are given in Table 5-15, which also provides the collapse ratios for all the archetype buildings.

Figure 5-33 shows the breakdown of collapse ratios based on each of the three criteria as defined in Section 3.5.3.2, namely, the peak resultant story drift ratio exceeding 10%, the maximum diaphragm shear angle exceeding 0.04 rad., and convergence failure occurring in the analysis. The resulting collapse ratio based on the union of the three collapse criteria is also shown in Figure 5-33, where the horizontal axis shows the 10 archetype buildings investigated in this study, and the vertical axis is the percentage of ground motions causing collapse. Observations from analysis of the results include the following:

- 1) As the number of stories increases, collapse ratio of the archetype buildings tends to become larger. This is more pronounced when comparing the 1-story buildings to other multistory buildings subjected to the DE and MCE-level ground motions: under DE-level ground motions, none of the 1-story buildings collapsed, while 6% of the ground motions caused collapse of the multistory buildings; and under MCE-level ground motions, 4% of the ground motions caused collapse of the 1-story building, while 25% of ground motions caused collapse of the multistory buildings. This observation is contrary to findings from some studies where 2D frame analysis was performed and low-rise buildings were deemed more vulnerable. For example, in Kircher et al. (2010), 2-story BRBFs had ACMR values close to 2.0, which was smaller than other BRBFs (with 3 to 16 stories) with ACMR values all larger than 3.0, indicating a smaller collapse margin for short buildings. In another study by Zaruma and Fahnstock (2018), the ACMR value increased from 1.87 for the 4-story baseline BRBF prototype to 2.08 for the 15-story. However, there also exist some studies which yield a similar conclusion that shorter buildings outperform taller buildings in terms of collapse prevention. In Veismoradi et al. (2016), the target spectral acceleration at the

5%-damped first mode period, $S_a(T_1, 5\%)$, that caused global dynamic instability of the structure decreased from 2.19g for the 3-story BRBF to 1.30g for the 10-story BRBF, indicating a higher capacity to prevent collapse for shorter buildings. Similarly in Khorami et al. (2017), the median value of $S_a(T_1, 5\%)$ associated with the collapse prevention limit state decreased from 2.08g for 3-story BRBF to 0.93g for 10-story BRBF. Further study is warranted to investigate the difference in these results.

- 2) In terms of collapse ratio, buildings with Alternative 1 diaphragm design perform slightly better than buildings with Traditional / Alternative 2 design, with an average difference of collapse ratios equal to 1.4%, 2.7%, and 2.5% at the DE, MCE, and ACMR_{10%} hazard levels, respectively. The small difference of collapse ratios implies that different diaphragm designs have small impact on the building performance in terms of collapse. This is because building collapse is more associated with excessive BRB deformation (and fracture) than diaphragm design. Based on the breakdown of collapse ratios, 94% of the building collapses have story drift limit exceeded with large BRB deformation and potential BRB fracture, while only 54% of the building collapses have diaphragm shear angle exceeded.
- 3) $R_s = 1.0$ design can be less conservative. From Figure 5-33 and Table 5-15, it is shown that for the 1-story building with concrete on steel deck roof under ACMR_{10%}-level ground motions and the 12-story building subjected to MCE-level ground motions, the collapse ratios for buildings with Traditional / Alternative 2 diaphragm design are smaller than those for buildings with Alternative 1 diaphragm design. This is likely because the Alternative 1 design produces stronger and stiffer diaphragms, which forces more inelasticity to occur in the BRB's under the specific scales of ground motions, resulting in larger story drift and more building collapses.
- 4) Examining Table 5-15 shows that strengthening the diaphragm produces the most benefit in terms of reducing collapses for the medium height buildings such as 4-story buildings. The 1-story buildings have sufficiently small collapse ratio, and for 8-story and 12-story buildings, the diaphragm design does not have a huge effect on reducing the number of collapses (average of 1% reduction in collapses). However, for 4-story buildings, strengthening the diaphragm reduced the collapse ratio by an average of 5.3% for the three hazard levels.

- 5) Based on the FEMA P695 methodology, one approach to evaluating acceptability is if less than 50% of the ground motions at the $ACMR_{10\%}$ level cause building collapse. Another approach is presented in Section 4.3.2.4. As shown in Figure 5-33c, all 1, 4, and 8-story buildings pass the acceptance criteria (with the number of collapses less than 50%), while the collapse ratios for the 12-story buildings exceeds the limit (58.1% for $R_s = 2$ or 2.5 and 54.5% for $R_s = 1$). It is also observed under this level of ground motions, the collapse ratios of the buildings with Traditional / Alternative 2 diaphragm design are close to those with Alternative 1 design (2.5% difference on average). Even for the 12-story buildings with collapse ratios slightly larger than 50%, there is only one additional analysis with building collapse for the Traditional / Alternative 2 design compared to the Alternative 1 design, which increases the collapse ratio by 3.6%. This also demonstrates that the collapses of the 12-story building are more associated with the BRBFs and not the diaphragms. Therefore, it is concluded that the alternative diaphragm design procedure with proposed R_s values ($R_s = 2$ for composite deck diaphragm and $R_s = 2.5$ for bare deck diaphragm) did not have a significant adverse effect on seismic performance of the archetype buildings compared to $R_s = 1.0$, and thus these R_s values may be reasonable for use in design of these types of structures.
- 6) However, the number of collapses for 4, 8, and 12-story buildings associated with DE and MCE hazard levels was larger than expected. The FEMA P695 methodology suggests that the probability of collapse due to MCE ground motions be limited to 10% for each performance group and 20% for individual archetypes. The average collapse ratios for multistory buildings subjected to DE and MCE-ground motions are 6% and 25%, respectively, and the 12-story building has more than 30% of ground motions causing collapse at MCE. These collapse probabilities are larger than those from some other studies with frame analysis. For example, the probabilities of collapse at MCE in Atlayan and Charney (2014) were below 3.5%. However, in the study by Zaruma and Fahnestock (2018), the results were more comparable, with the numbers of collapses for the baseline prototypes subjected to MCE being 9/44 (20.5%) for 4-story BRBF, 12/44 (27.3%) for 9-story BRBF, and 8/44 (18.2%) for 15-story BRBF. It is hypothesized that the excessive number of collapses at DE and MCE in this study are due to the use of 3D models, and to a lesser extent the consideration of diaphragm deformations. As mentioned in Section

4.3.2.1, median peak resultant story drifts are 50% to 100% larger at the critical first story compared to the story drifts in the x or y direction alone, which significantly exacerbates the P- Δ effect. Also, pushover curves show softening (negative slope) at relatively small roof drift ratio, especially for tall buildings.

- 7) The performance of archetype buildings investigated in this study in terms of collapse is in general worse compared to results from conventional 2D frame analyses. It can be deduced that the ACMR values for the 1, 4, and 8-story buildings in this study are larger than the acceptable ACMR_{10%} values equal to 1.96 as given in Table 5-11, since less than half of the ground motions caused collapse of these buildings, while the ACMR values for the 12-story with collapse probability slightly larger than 50% is slightly smaller than the acceptable ACMR_{10%} equal to 1.96. In contrast, all archetype BRB frames evaluated in Chen (2010) with number of stories ranging from 2 to 16 passed the evaluation criteria per FEMA P695, with the ACMR values ranging from 2.31 to 4.12. In addition, the case studies in Atlayan and Charney (2014) also showed satisfactory performance of 5-story archetype BRB frames using the ACMR acceptance criteria, where the ACMR values ranged from 2.63 to 3.23. However, in Zaruma and Fahnstock (2018), ACMR values for the baseline prototypes investigated were smaller which were 1.87 for 4-story BRBF (failed), 1.95 for 9-story BRBF (failed), 2.08 for 15-story BRBF (pass). It should be noted that these comparisons may not be ideal because of the different design conditions (e.g., seismic design category) and different criteria for the definition of collapse cases (e.g., story drift limit), etc. Further investigation is desired to examine the 3D effect on the evaluation of seismic performance of buildings.

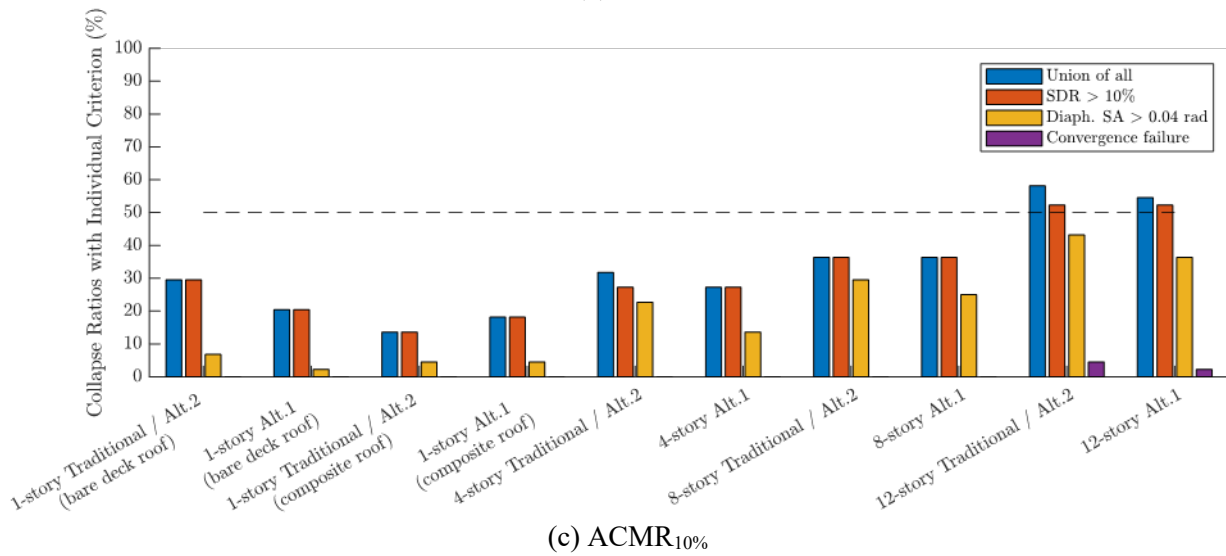
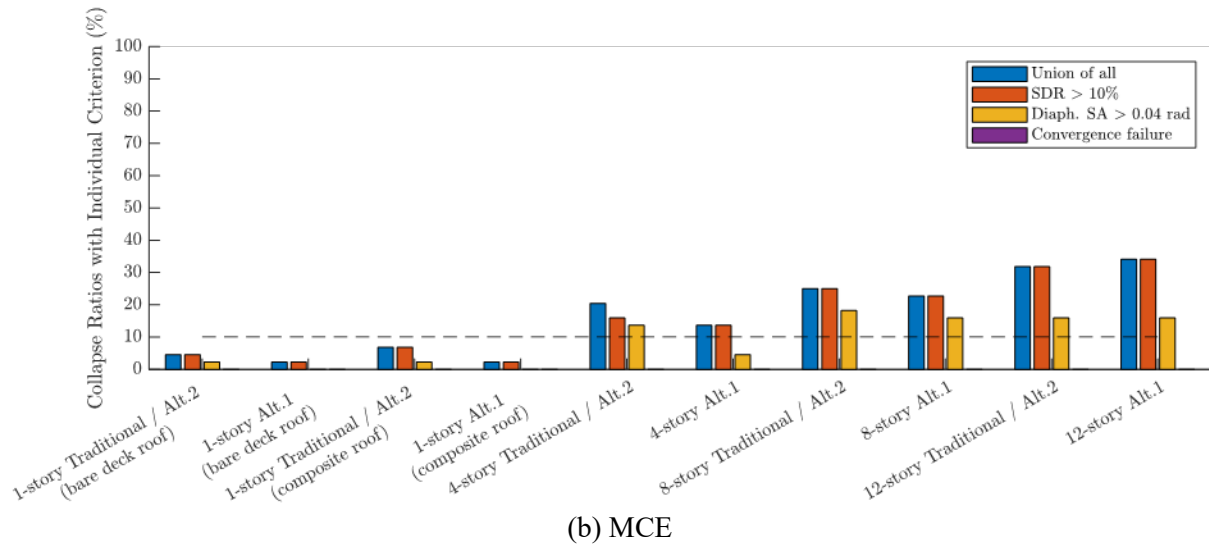
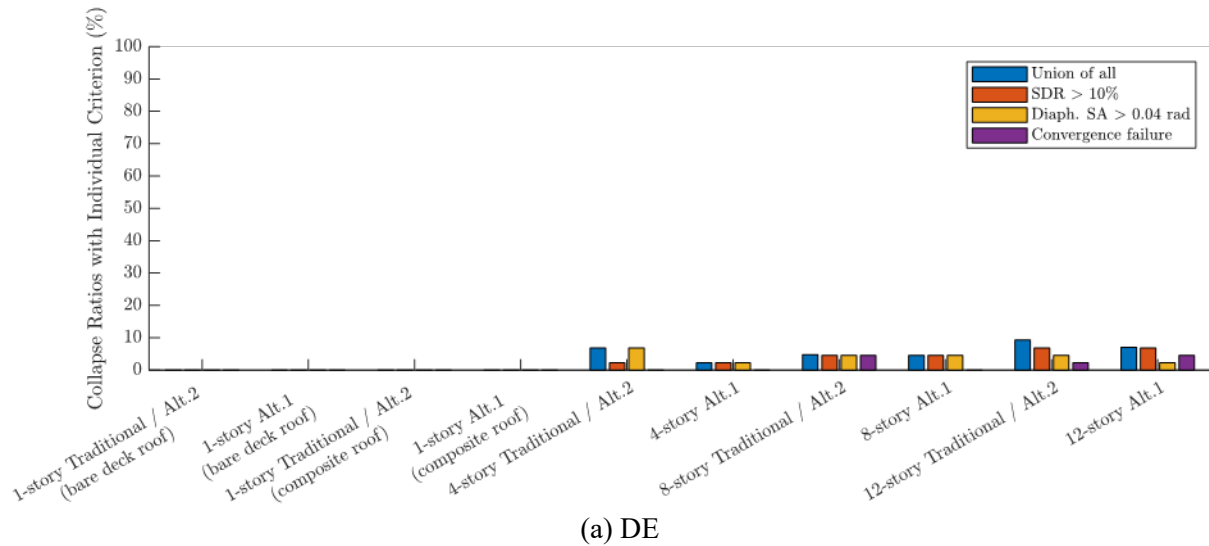


Figure 5-33 Collapse ratio breakdown for buildings under three levels of ground motions

Table 5-15 Collapse ratios for buildings under three levels of ground motions

Archetype Building		DE			MCE			ACMR _{10%}		
		N_{run}^1	P_c^2 (%)	ΔP_c^3 (%)	N_{run}	P_c (%)	ΔP_c (%)	N_{run}	P_c (%)	ΔP_c^3 (%)
1-story (bare steel deck roof)	Trad. /Alt. 2	44	0.0	0.0	44	4.5	2.3	44	29.5	9.1
	Alt. 1	44	0.0		44	2.3		44	20.5	
1-story (comp. deck roof)	Trad. /Alt. 2	44	0.0	0.0	44	6.8	4.5	44	13.6	-4.5
	Alt. 1	44	0.0		44	2.3		44	18.2	
4-story	Trad. /Alt. 2	44	6.8	4.5	44	20.5	6.8	44	31.8	4.5
	Alt. 1	44	2.3		44	13.6		44	27.3	
8-story	Trad. /Alt. 2	42	4.8	0.2	44	25.0	2.3	44	36.4	0.0
	Alt. 1	44	4.5		44	22.7		44	36.4	
12-story	Trad. /Alt. 2	43	9.3	2.3	44	31.8	-2.3	43	58.1	3.6
	Alt. 1	43	7.0		44	34.1		44	54.5	
Average		44	3.5	1.4	44	16.4	2.7	44	32.6	2.5

¹: number of runs considered for collapse ratio calculation

²: collapse ratio (probability of collapse)

³: difference of the collapse ratios between archetype buildings with Traditional / Alternative 2 diaphragm design and archetype buildings with Alternative 1 diaphragm design

5.4.3.2.4. Adjusted Collapse Margin Ratios

FEMA P695 also provides a methodology to evaluate the seismic collapse performance of the buildings based on the adjusted collapse margin ratio. Collapse margin ratios are estimated in this work based on a lognormal cumulative distribution function that is fit to the collapse data obtained at three hazard levels. The computed adjusted collapse margin ratio (*ACMR*) is then compared to the acceptable value of *ACMR*, which for individual index archetypes within a performance group is *ACMR*_{20%}, and for the average *ACMR* for each performance group is *ACMR*_{10%}. The subscripts of the *ACMR* acceptable values, *ACMR*_{10%} and *ACMR*_{20%}, denote the acceptable collapse probability due to MCE ground motions, taken as 10% and 20%, respectively.

The collapse margin ratio (CMR) was first obtained, which is defined in FEMA P695 as the ratio of the median spectral acceleration of the collapse level ground motions, $\hat{S}_{CT}$, to the spectral acceleration of the MCE ground motions, S_{MT} , at the fundamental period of the structure, T . Table

5-16 summarizes the fundamental period and spectral acceleration of the MCE (S_{MT}) and DE (S_{DT}) ground motions for the archetype buildings. The median spectral acceleration of the collapse level ground motions, $\hat{S}_{CT}$, was estimated as the spectral acceleration corresponding to a collapse probability equal to 50% using the lognormal cumulative distribution function (CDF), which was fitted with the values of collapse probabilities of the archetypes at the three hazard levels using the *fminunc* function in MATLAB that minimizes the sum of the square of the errors in the predicted collapse probabilities from the fitted CDF. An example of the fitted lognormal CDF plots for the 4-story archetype with different diaphragm designs is given in Figure 5-34. The fitted lognormal CDF plots for other archetype buildings are provided in Appendix A.

Table 5-16 Spectral acceleration of MCE and DE ground motions

Design Parameters	1-story ^a	1-story ^b	4-story	8-story	12-story
T	0.304	0.304	0.807	1.343	1.814
S_{MT}	1.545	1.545	1.058	0.636	0.471
S_{DT}	1.030	1.030	0.705	0.424	0.314

^a: bare deck roof; ^b: concrete-filled steel deck roof

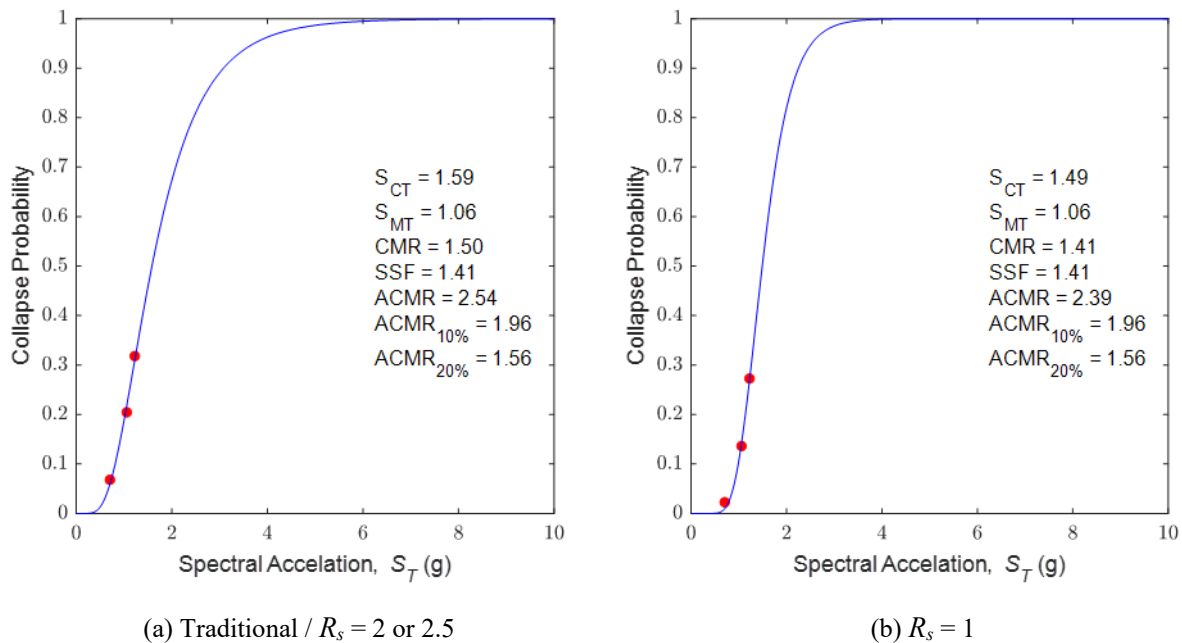


Figure 5-34 Example lognormal cumulative distribution plots for 4-story archetype with different diaphragm designs

The values of the CMR given by the ratio of the estimated $\hat{S}_{CT}$ to S_{MT} was multiplied by the spectral shape factor (SSF) to obtain the $ACMR$, which was then compared to the acceptable values of $ACMR$. Table 5-17 summarizes the evaluation of the seismic collapse performance of the archetype buildings using the FEMA P695 procedure. It can be observed that all the individual archetypes have an $ACMR$ value greater than $ACMR_{20\%}$ and therefore meet the evaluation criterion. In addition, the two groups of archetype buildings with different diaphragm designs satisfy the requirement of FEMA P695 since the average of the $ACMR$ values within both groups is greater than $ACMR_{10\%}$. This means that the traditional diaphragm design, diaphragm design with $R_s = 2.0$ for concrete-filled steel deck and $R_s = 2.5$ for bare deck, and the $R_s = 1.0$ diaphragm design all satisfy the FEMA P695 requirements.

It is observed that the average $ACMR$ value for the archetype buildings with traditional / $R_s = 2$ or 2.5 diaphragm design is larger than that of the archetype buildings with $R_s = 1$ diaphragm design, indicating that the design approach with stronger diaphragms does not lead to a larger safety margin against collapse for this type of structural system. Building models with stronger diaphragms may cause higher probability of collapse in these BRBF buildings because the collapse mode is typically related to excessive deformations / fracture of the BRBs and stronger diaphragms will force more deformation demand into the BRBF.

Table 5-17 Summary of collapse performance evaluation of archetype buildings using FEMA P695 procedure

Archetype Building	FEMA P695 Evaluation													
	Traditional / $R_s = 2$ or 2.5							$R_s = 1$						
	S_{CT}	S_{MT}	CMR	ACMR	$ACMR_{20\%}$	$ACMR_{10\%}$	Pass/Fail	S_{CT}	S_{MT}	CMR	ACMR	$ACMR_{20\%}$	$ACMR_{10\%}$	Pass/Fail
1-story ^a	2.15	1.55	1.39	2.18	1.56	1.96	Pass	2.19	1.55	1.42	2.26	1.56	1.96	Pass
1-story ^b	3.17	1.55	2.05	3.27	1.56	1.96	Pass	2.25	1.55	1.46	2.33	1.56	1.96	Pass
4-story	1.59	1.06	1.50	2.54	1.56	1.96	Pass	1.49	1.06	1.41	2.39	1.56	1.96	Pass
8-story	1.07	0.64	1.68	2.40	1.56	1.96	Pass	0.82	0.64	1.28	2.24	1.56	1.96	Pass
12-story	0.54	0.47	1.14	1.86	1.56	1.96	Pass	0.54	0.47	1.15	1.89	1.56	1.96	Pass
Average			1.55	2.45	1.56	1.96	Pass			1.34	2.22	1.56	1.96	Pass

a: bare deck roof; b: concrete-filled steel deck roof

5.5. Evaluation of 3D Effects on Seismic Collapse of Buildings

Two-dimensional (2D) frame analysis has been used extensively to predict the seismic response (e.g., peak story drifts and collapse) of buildings subjected to earthquake excitation (e.g., Farahi and Mofdi, 2013; Elkady and Lignos 2014; Malakoutian et al., 2015). In this type of analysis, the vertical LFRS in the building are modeled assuming the horizontal diaphragm system is rigid without dissipating seismic energy and that the vertical LFRS responds independent of the rest of the building. In an actual building, the diaphragm system deforms contributing to lateral drifts, and interacts dynamically with the vertical LFRS, as was observed from the analysis results presented in Section 5.4. Furthermore, inelasticity and failure of the diaphragm can have substantial effect on the seismic behavior and collapse of the overall building. It might be expected, therefore, that 2D frame analysis would result in smaller estimates of seismic collapse potential than 3D analysis. For this reason, FEMA P695 specifies that the collapse margin ratio obtained from 3D analyses should be amplified by a factor of 1.2, irrespective of diaphragm modeling approach, to account for larger collapse potential found in 3D analyses compared to 2D analyses.

To understand the effects of 3D analysis, different diaphragm modeling assumptions, and unidirectional vs. bidirectional seismic excitation on the seismic collapse response of buildings, another computational study was conducted with models varying in complexity from those equivalent to a nonlinear 2D frame analysis to a full 3D building analysis with nonlinear behavior in both the vertical LFRS and the diaphragm system. Nonlinear response history analyses were performed with the same far field ground motion set from FEMA P695 scaled to different hazard levels as described in Section 5.3.5.3.1 and the resulting performance in terms of drifts and collapse were evaluated.

5.5.1. Details of Building Models

The eight-story archetype building with Traditional / Alternative 2 diaphragm design from Section 5.2 was adopted for use in this study. Typical elevations of the BRB frames and a visualization of the 3D building model are shown in Figure 5-35. The modeling scheme developed in Section 5.3 was used for this study.

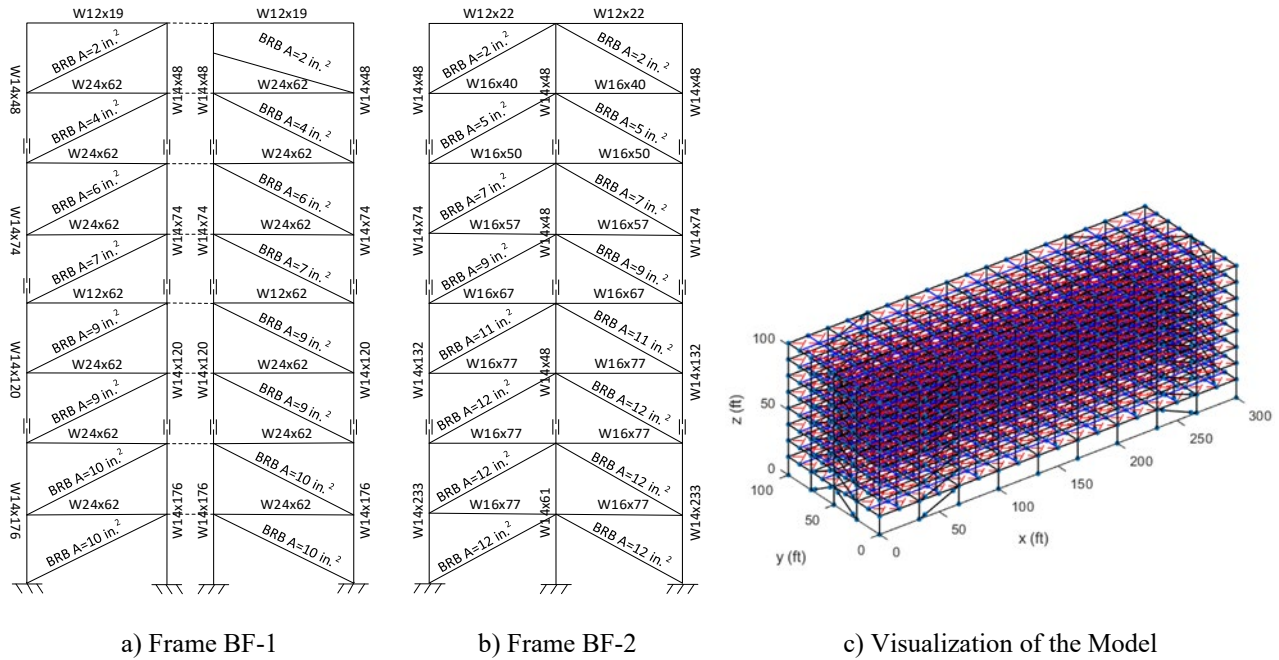


Figure 5-35 Typical BRBF elevations of 8-story archetype building

A total of nine building models were considered in this study, including every combination of three diaphragm modeling assumptions (i.e., rigid, elastic, and inelastic diaphragms), and three earthquake loading patterns (i.e., bidirectional loading, unidirectional loading in the longitudinal (x) direction, and unidirectional loading in the transvers (y) direction). For simplicity, they are herein referred to as: “RigidBi”, “RigidUniX”, “RigidUniY”, “ElasticBi”, “ElasticUniX”, “ElasticUniY”, “InelasticBi”, “InelasticUniX” and “InelasticUniY”. The prefix describes the approach to diaphragm modeling and the suffix refers to the ground motion direction. The models with the elastic and rigid diaphragm assumption are identical to the models with inelastic diaphragms except that elastic material was used for the diaphragm truss elements instead of the Pinching4 material model. The stiffness of the elastic material for models with elastic diaphragms is equal to the initial stiffness of the Pinching4 material model. This stiffness is amplified by a factor of 1000 for the elastic material of rigid diaphragms.

5.5.2. Analysis Results

The results of the response history analyses are presented in the following order: 1) evaluating median peak story drifts for all models, 2) examining response to one ground motion to help

understand behavior, 3) investigating collapse results, and 4) discussion of the effect of bidirectional vs. unidirectional ground motions.

Figure 5-36 shows the distribution of median peak story drift ratio (SDR) in the y direction (short dimension of the building) along the building height for the building models subjected to bidirectional earthquake loading at all three hazard levels. Two different story drifts are plotted, i.e., the median peak story drift for the BRBFs and the median peak total story drift of the building which includes the deformation of the BRBFs and the diaphragms. Only one line is plotted for the models with rigid diaphragms because the two are identical.

For the DE and MCE hazard levels, Figure 5-36a and Figure 5-36b show that the peak total story drifts generally get larger as the diaphragm model goes from rigid (RD) to elastic (ED) to inelastic (ID). For lower hazard levels, explicitly modeling diaphragm deformations will increase the total story drifts because the diaphragm is adding flexibility. The added flexibility is demonstrated by the fact that the BRBF story drifts at the DE and MCE levels are quite similar regardless of the diaphragm modeling approach, and that diaphragm deformations act as merely superimposing additional deflections leading to larger total story drift. This is particularly true for smaller motions where the LFRS and diaphragm remain in the elastic range or experience moderate inelasticity. It is also observed that the story drift at the top story is most affected by the diaphragm modeling assumption because the bare steel deck roof diaphragm is considerably more flexible than the concrete-filled steel deck floor diaphragms.

At the ACMR_{10%} hazard level, however, the behavior is substantially different as the building models approach collapse. Figure 5-36c shows that the building models with rigid (RD) and elastic (ED) diaphragms experience median peak story drifts that are approximately two times larger than the model with inelastic diaphragms (ID). To understand why the trend at ACMR_{10%} hazard level is opposite that at the lower hazard levels, it is necessary to examine all the possible reasons that changing the diaphragm model affects peak drifts:

- 1) Diaphragm deflections add flexibility to the structure which increase total story drift for ED and ID.
- 2) The mode shapes of the building in higher modes has large diaphragm deformation component (see Section 5.4.1) that could increase upper story drifts for ED and ID.

- 3) The periods for models with ED and ID are larger than RD (see Section 5.4.1) and thus the associated spectral accelerations and peak drifts would be expected to decrease for ED and ID.
- 4) Inelastic action of the diaphragm would dissipate seismic energy such that peak drifts are decreased for ID.
- 5) Inelastic deformations in the diaphragm reduce the deformation demands in the BRBF which can greatly decrease peak drifts in ID, especially as the BRBs near fracture.

For the DE and MCE hazard levels, reasons (1) and (2) lead to as much as 20% increase in story drifts for the models with elastic and inelastic diaphragms as compared to the rigid diaphragm models. At the ACMR_{10%} hazard level, reason (5) creates substantially smaller story drifts in the ID models as demonstrated by the following discussion of a single ground motion run.

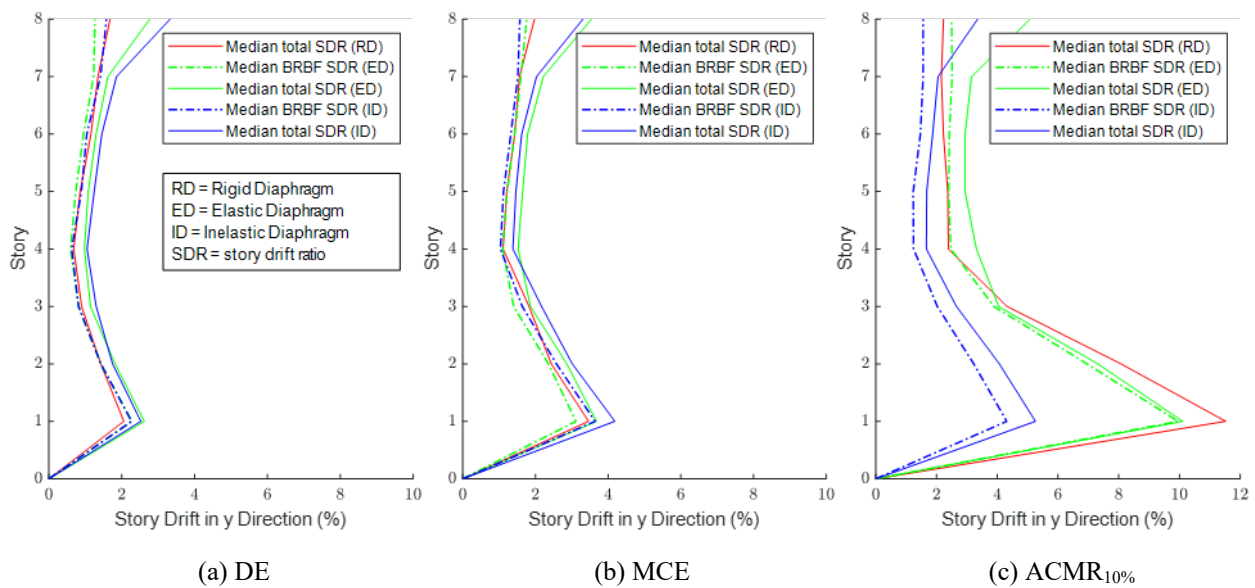


Figure 5-36 Distribution of median peak story drift in y direction along building height with different diaphragm modeling assumption and bidirectional earthquake loading

The responses of the rigid diaphragm (RD) and inelastic diaphragm (ID) building models subjected to bidirectional ground motion from the Nishi-Akashi recording station during the 1995 Kobe Earthquake scaled to ACMR_{10%} are shown in Figure 5-37. Figure 5-37a shows the resultant story drift (i.e., square root of the sum of the squares of the story drift ratios in the x and y directions) for the first story and Figure 5-37b shows the BRB core strain for one of the BRBs in the first story along the x-axis. During the first 10 seconds of the motion, the two building models experienced

similar story drift, but the BRB core strain was larger for the RD case. In a context where earthquake motions are considered as applying a displacement demand on the structure, this result can be interpreted as follows: making the diaphragm rigid, forces more deformation demand into the BRBF. Between 10 seconds and 25 seconds, the BRB undergoes a ratcheting behavior as the story drift goes from approximately 5% to more than 10%. The P- Δ effect contributes to this ratcheting behavior as does the low hardening slope of the BRBs (Kiggins and Uang 2006). After 25 seconds, the damage index associated with the Fatigue fracture law used in the model reaches a value of 1.0 which causes the BRB strength to go to zero and story drift to increase toward simulated collapse.

Viewed another way, the first 8 seconds of the ground motion may represent building behavior during ground motions such as the DE and MCE hazard level where the inelastic diaphragm model experiences large story drift. The latter part of the ground motion represents building behavior at ACMR_{10%} and near collapse where the rigid diaphragm model forces larger deformation demands into the BRBF which leads to BRB fracture and collapse. At the ACMR_{10%} hazard level, the number of ground motions resulting in BRB fracture was found to be 8, 20, and 21 for the building models with inelastic, elastic, and rigid diaphragms, respectively.

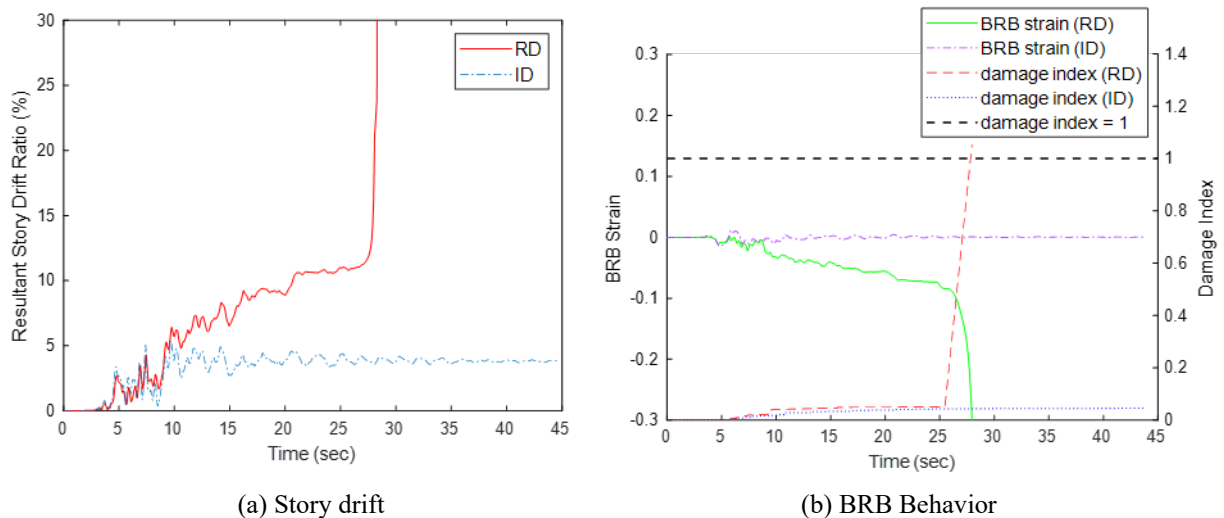


Figure 5-37 Building response with rigid and inelastic diaphragms subjected to ACMR_{10%} hazard level and bidirectional ground motions for the Nishi-Akashi motion of 1995 Kobe, Japan earthquake

To investigate the seismic collapse performance of the buildings with different diaphragm modeling assumptions and earthquake loading patterns, the number of ground motions causing

collapse was tabulated for each hazard level. The same criteria as described in Section 5.3.5.3.2 were considered for the definition of building collapse.

Table 5-18 provides the percent of ground motions causing collapse for building models subjected to bidirectional ground motions. For the DE and MCE hazard levels, it is observed that the number of ground motions causing collapse was similar regardless of how the diaphragm was modelled. However, at the ACMR_{10%} hazard level, the model with rigid diaphragm had almost twice as many ground motions causing collapse at the model with inelastic diaphragm. Since simulated collapse in these building models is sensitive to large BRB core strains which cause ratcheting in the story drift response and BRB fracture, models that forced more deformation demand into the BRBF (i.e., rigid diaphragm models), result in substantially more collapses.

Table 5-18 Proportion of ground motions causing collapse for models subjected to bidirectional motions.

Diaphragm Rigidity	Hazard Level		
	DE	MCE	ACMR _{10%}
Rigid	4.5%	25.0%	64.3%
Elastic	4.5%	20.5%	54.8%
Inelastic	4.5%	25.0%	36.4%

The effect of bidirectional ground motions compared to unidirectional ground motions is also evaluated. Figure 5-38 shows the median peak story drifts of the building models with rigid diaphragms subjected to unidirectional and bidirectional earthquake loading at the MCE hazard level. It can be observed from Figure 5-38a that the median peak x-direction story drift in the first story BRBF increases by 11% when the loading is changed from single ground motions applied in the x-direction only to bidirectional ground motion pairs. Similarly, Figure 5-38b shows a 16% increase in first story BRBF drift demand with bidirectional ground motions. This shows that the application of bidirectional ground motion pairs leads to larger in-plane drift demands in the braced frames than unidirectional ground motions, especially at stories subjected to large drift where P-Delta effects are more significant. Figure 5-38c shows the expected result that the peak resultant drifts (i.e., along the diagonal), are larger with the application of ground motion pairs. This result, however does not directly relate to the story drift demands on the BRBF in the X and Y direction like Figure 5-38a and Figure 5-38b.

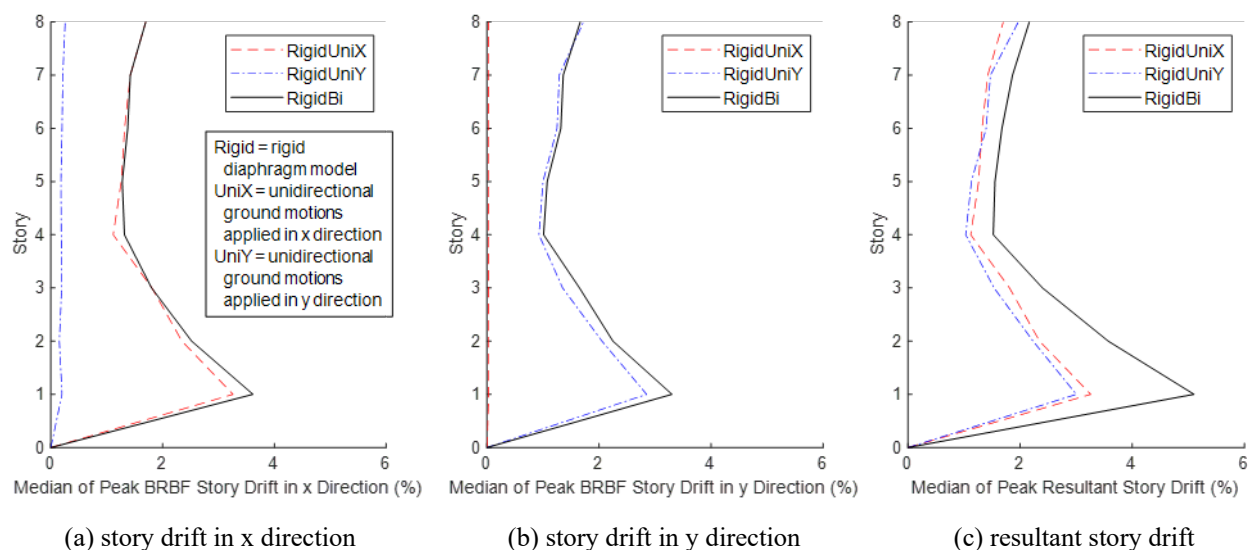


Figure 5-38 Distribution of median peak story drifts along building height for building models with rigid diaphragms considering unidirectional and bidirectional earthquake loading at MCE hazard level

Table 5-19 provides the ratio of ground motions causing building collapse for all building models analyzed and separated for unidirectional and bidirectional earthquake loading. The likelihood of collapse was significantly larger with bidirectional ground motions for both the rigid diaphragm and elastic diaphragm models. This indicates that the probability of building collapse can be underestimated using 2D building models subjected to unidirectional earthquake loading. FEMA P695 accounts for this difference by applying a factor of 1.2 to the collapse margin ratio obtained from 3D building models subjected to bidirectional ground motions. The ratios of the number of ground motions causing collapse using unidirectional X or Y direction motions to the number of ground motions causing collapse when subjected to bidirectional motions are given in the last two columns of Table 6 for the $ACMR_{10\%}$ hazard level. These results are not tabulated for other hazard levels because they represent the tail of the collapse distribution and thus are not as easy to quantify with 44 motions. Table 6 shows that the number of ground motions that cause collapse for the rigid and elastic diaphragm building models was between 1.3 and 1.8 times larger when bidirectional ground motions are used. While this ratio on number of ground motions causing collapse is not directly comparable to the 1.2 factor on collapse margin ratio in FEMA P695, it suggests that the factor may warrant additional investigation. The models with inelastic diaphragms had a different result in that collapse probabilities were similar regardless of whether the ground motions were applied in one direction or bidirectional.

Table 5-19 Proportion of ground motions causing collapse for all models

Diaphragm Model	Hazard Level	Earthquake Loading			Ratio	
		X-Direction	Y-Direction	Bi-Directional	Bi / X	Bi / Y
Rigid	DE	2.3%	0.0%	4.5%	-	-
	MCE	13.6%	5.0%	25.0%	-	-
	ACMR _{10%}	48.8%	36.4%	64.3%	1.32	1.77
Elastic	DE	2.3%	0.0%	4.5%	-	-
	MCE	15.9%	6.8%	20.5%	-	-
	ACMR _{10%}	40.5%	30.0%	54.8%	1.35	1.83
Inelastic	DE	4.5%	2.4%	4.5%	-	-
	MCE	16.7%	8.1%	25.0%	-	-
	ACMR _{10%}	40.9%	35.7%	36.4%	0.89	1.02

5.6. Conclusions

A series of 1, 4, 8, and 12-story archetype buildings were designed to the current U.S. building code with three different diaphragm designs: a traditional design that uses conventional diaphragm design forces, an alternative design that uses the seismic demand calculated assuming some diaphragm ductility (values proposed for future editions of the building code of $R_s = 2$ for concrete on steel deck floor diaphragms and 2.5 for bare steel deck roof diaphragms) which ended up the same as the traditional design, and an alternative design with diaphragm demands assuming no diaphragm ductility ($R_s = 1.0$). Using material models calibrated against test data for diaphragms and BRB, 3D computational models with material and geometric nonlinearity were created. These models were used to conduct modal analyses to study their modal properties, nonlinear pushover analyses to investigate their static behavior, and nonlinear response history analyses to evaluate their seismic performance.

It was found that design office models with a rigid diaphragm assumption can significantly underpredict the natural period (up to 48% underpredicted for some models) and miss some key features of the mode shape. The different diaphragm designs had little effect on the pushover behavior because the pushover analyses used a first mode shape based load pattern and were dominated by BRB inelasticity.

Conversely, response history analyses showed significant inelasticity occurred in the diaphragms as higher modes affected the diaphragm demands. There was also an interaction between diaphragm inelasticity and BRB inelasticity as the two compounded each other to exacerbate the second order effects and cause collapse. Large story drift concentrates at the first story of the building where P- Δ effects are the worst. For the intermediate stories, the peak story drifts are smaller and more uniformly distributed along the building height, while the peak story drifts near the roof become larger due to the “whipping effect” of the building. In addition, because of the 3D effect and diaphragm deformation, the peak resultant story drifts can be twice as large as the story drifts along either orthogonal direction of the buildings. The total story drift considering diaphragm deformation can be significantly larger than the story drift at the BRB frames (up to 80% difference), especially when the diaphragms have smaller in-plane stiffness, which can result in even larger P- Δ effect. It also indicates that conventional 2D or 3D frame analysis with rigid diaphragm assumption can well underestimate the story drifts of the building.

The diaphragms of the archetype buildings remained almost entirely elastic under DE-level ground motions. The diaphragm shear demands for archetype building with alternative $R_s = 1.0$ diaphragm design were compared to the elastic diaphragm design shear from ASCE 7 alternative diaphragm design procedure. It was found that ratios of the diaphragm shear demand obtained from the analysis to the design shear given by the alternative diaphragm design procedures in ASCE 7 have an average value of 0.84, indicating a reasonably accurate but slightly conservative prediction of elastic diaphragm shear demand with the design approach.

The performance of the archetype buildings in terms of collapse was evaluated based on the collapse ratio from the results of nonlinear response history analysis. As the number of stories increases, collapse ratio of the archetype buildings tends to become larger. This is more pronounced when comparing the 1-story buildings to other multistory buildings under the DE and MCE-level ground motions, which is contrary to some other studies with 2D frame analysis that have shown low-rise buildings to be more vulnerable. In general, the number of collapses associated with alternative $R_s = 1.0$ diaphragm design is very close (with an average difference of 2.5% for collapse ratios) to that with traditional or alternative design with $R_s = 2.0$ for concrete on metal deck diaphragms and 2.5 for bare deck diaphragms, and it is expected that these collapses are more associated with 3D effects than diaphragm design. This is further supported by observing

that the difference in median story drifts was negligible between the alternative $R_s = 1.0$ and alternative $R_s = 2.0$ or 2.5 diaphragm design.

The adjusted collapse margin ratio (ACMR) of all buildings with different diaphragm design procedures was found to satisfy the FEMA P695 criteria with all individual archetypes satisfying the ACMR_{20%} limit and the average for each performance group satisfying the ACMR_{10%} limit. Therefore, it is concluded that the alternative diaphragm design procedure with proposed R_s values ($R_s = 2$ for composite deck diaphragms and $R_s = 2.5$ for bare deck diaphragms) is reasonable for use in the design of these types of structures.

It is noted that due to the 3D effect in the analysis with the consideration of diaphragm nonlinearity in this study, there are more collapses than expected for multistory buildings under the DE and MCE-level ground motions, with the average collapse ratios equal to 6% and 25%, respectively. Conventional seismic performance evaluation of steel braced frame buildings primarily focuses on 2D frame analysis where the inelasticity in the diaphragms and its interaction with the inelasticity in the vertical lateral force resisting system are neglected. To investigate the effect of diaphragm modeling assumptions and earthquake loading patterns in 3D collapse modeling of building structures, a series of 8-story building models were developed and analyzed with varying diaphragm rigidity (rigid, elastic, or inelastic) and earthquake loading pattern (bidirectional or unidirectional).

For the DE and MCE hazard levels, the response history analyses showed that even though the number of ground motions causing building collapse was similar regardless of how the diaphragm was modelled, diaphragm deformations contributed to as much as 20% larger drifts in the models with elastic and inelastic diaphragms as compared to the rigid diaphragms. However, at the ACMR_{10%} hazard level, inelastic deformations in the diaphragm reduce the deformation demands in the BRBF as the BRBs near fracture, which results in substantially smaller story drifts and up to 43% fewer collapses in buildings with inelastic diaphragms than those with rigid or elastic diaphragms.

For rigid diaphragm building models, it was found that the in-plane BRBF story drift demands were between 11% and 16% larger when the building was subjected to bidirectional ground motion pairs as opposed to single unidirectional ground motions. The number of ground motions that

caused collapse of the rigid diaphragm and elastic diaphragm buildings was also notably larger (between 30% and 80% larger) when ground motion pairs, scaled to the $ACMR_{10\%}$ hazard level, were applied compared to unidirectional ground motions. This discrepancy in collapses indicates that the probability of building collapse can be underestimated using 2D building models subjected to unidirectional earthquake loading and the magnitude of the discrepancy implies that the FEMA P695 3D factor of 1.2 on collapse margin ratio may warrant additional investigation.

CHAPTER 6. EXPERIMENTAL STUDY ON STANDING SEAM ROOF

6.1. Introduction

6.1.1. Overview

The SSR system is widely used in metal buildings due to several advantages over some conventional through-fastened metal deck roof systems. It eliminates the need to create holes in the roof panels for deck attachment (e.g. welds, power actuated fasteners, or screws through metal roof deck typical in steel-framed buildings), and as a result can improve water tightness of the roof system and prolong its life cycle. The clip fasteners are hidden in the standing seams and are not exposed to outside environmental conditions which protects them against accelerated corrosion and failure. This feature also reduces the cost of regular maintenance. In addition, the SSR system has superior performance when subjected to thermal variations. The flexibility of the clip fasteners and the standing seam allow the thermal expansion and contraction of the roof panels in both transverse and longitudinal directions. The reflection of sun's ray by the metal roof decreases the heat transfer and improves the energy efficiency of the cooling system, while optional thermal blocks and blanket insulation provide further energy efficiency. The favorable architectural appearance of the SSR system is also one of the reasons why it has become the first choice for many buildings.

Despite multiple advantages of the SSR system, it also presents some challenges. It's usually considered more expensive than some other alternatives, partially due to its labor-intensive installation process including the spacing and fastening of clips on the purlins, aligning and snapping the legs of roof panels next to each other, and roll forming (i.e. crimping) the standing seam with tools and machines. Also, the standing seam roof provides only limited stiffness or strength as a diaphragm because clips and seams allow movement.

6.1.2. Motivation

There are different types of bracing at the roof level that restrain lateral movement of the entire building and restrain lateral movement of individual members. The in-plane bracing in the roof plane of a metal building is provided by tension rod or cable bracing with X braces attached to the

web of the rafter near the top flange. Another type of bracing involves top and bottom flange bracing of the rafter to prevent lateral torsional buckling of the rafter. This type of bracing is typically provided by the purlins at the top flange and diagonal flange braces that extend from the purlin to the bottom flange.

The flange brace and purlin create frame action that acts as torsional bracing for the rafter. Additionally, the SSR system provides some restraint against rafter lateral motion because it resists relative longitudinal motion of purlins. That is, for purlins not located at the point where roof X braces intersect the rafter, the SSR system restrains longitudinal motion of the purlin relative to adjacent purlins. This concept is demonstrated in Figure 6-1 as the lateral force associated with rafter lateral torsional buckling is applied to the purlins. A simplified free-body diagram of the SSR system with lateral bracing load is shown in Figure 6-2. It is noted that this loading configuration is highly idealized as a single bracing load and simple supports. In the context of AISC 360-16 (AISC 2016), the locations where the roof X braces connect to the rafter may be considered point bracing for global stability, while the SSR system provides point bracing at the interior purlin locations for local instability. Also, there may be loads imposed at more than one lateral brace, and these loads may not be going the same direction.

AISC 360-16 (AISC 2016) Appendix 6 provides equations for calculating the required stiffness and strength for beam point bracing. However, the in-plane resistance of the SSR panels and their ability to brace the rafter have not been previously studied. It is therefore necessary to conduct experimental tests to investigate the in-plane bending stiffness and strength of SSR assemblies.

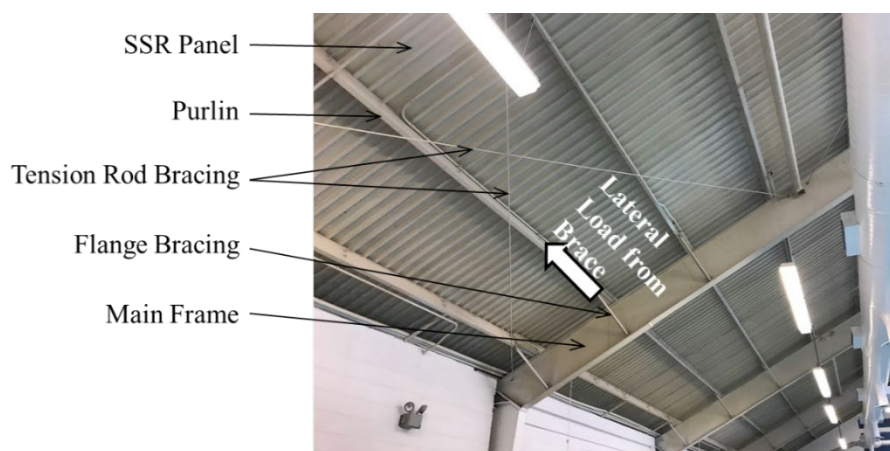


Figure 6-1 Lateral bracing and load transfer of main frame

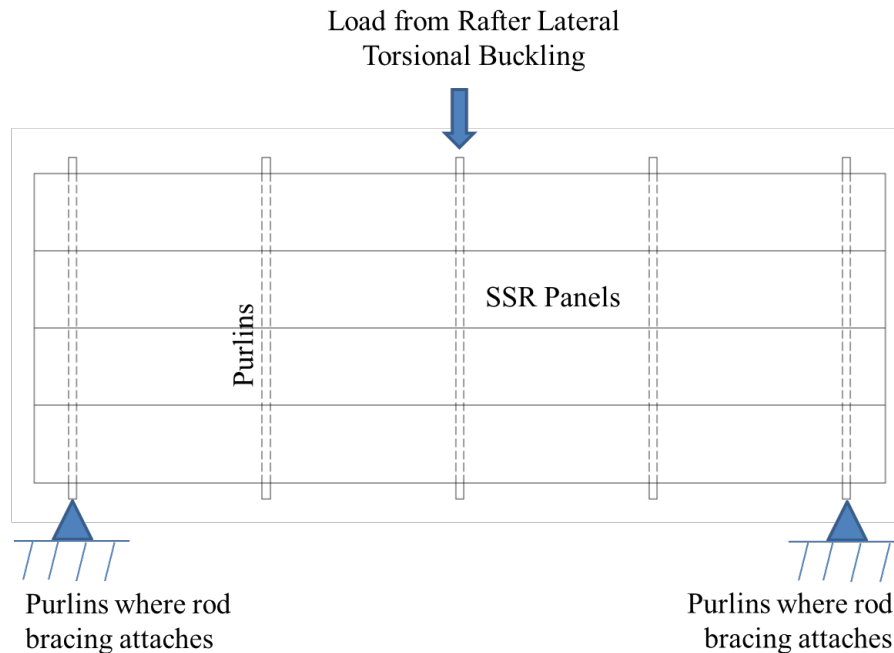


Figure 6-2 Lateral load from rafter lateral torsional buckling acting on purlins and SSR panels

6.1.3. Scope of Work

Eleven full-scale in-plane bending tests on SSR systems were conducted at the Thomas M. Murray Structures Lab of Virginia Tech. Each specimen was a 22 ft long SSR unit with panels and purlins that represent the portion of a roof between points where roof X bracing attaches to the rafter. The objective of this study is to investigate the in-plane strength and stiffness of the SSR system for restraining the purlins against longitudinal movement. Each specimen had a unique configuration to study the effect of different parameters including SSR panel profiles, SSR panel width, type of clip and screw fasteners, clip standoff, and use of thermal blocks and blanket insulation. Results from the tests were analyzed and compared to the bracing requirements in AISC 360-16 for an example rafter.

6.2. Testing Program

6.2.1. Test Matrix

There were 11 specimens with different configurations as given in Table 6-1 and typical specimen layout shown in Figure 6-3. The specimens had 4 SSR panels and 5 or 9 purlins

depending on the purlin spacing. The length of the panels was 22 ft, and the length of the purlins was either 6.5 ft for the specimens with 16 in. wide panels or 9 ft for the specimens with 24 in. wide panels. The parameters considered in developing the test matrix include the following:

- 1) Panel profile. The profiles of the SSR panels are categorized into two main types: trapezoidal rib profile which has sloped legs adjacent to the seam, and vertical rib profile which has vertical legs adjacent to the seam. The different panel profiles are shown in Figure 6-4.
- 2) Manufacturer. Material for constructing the specimens, including SSR panels, purlins, clips and screw fasteners, were provided by different manufacturers which are referred to as TS-324, MSC, VS-216, and MVP.
- 3) Panel gauge. All the specimens used SSR panels with 24-gauge thickness.
- 4) Panel width. The panels were either 24 in. or 16 in. wide.
- 5) Clip type. Fixed clips are snapped to the male leg of panels and mounted onto the purlin flange. They do not have a slider and are fixed in place. Floating clips are two-piece clips with the body of clip snapped and seamed to the legs of panels, the base installed to the purlin flange, and these two pieces being connected by a mechanism to allow sliding of the main body as a unit. Sliding tab clips have a rigid main body with a sliding tab that fits into the seam and slides along a slotted hole on the main body. Figure 6-5 shows the different clips used to construct the specimens.
- 6) Clip standoff. To allow for insulation to be sandwiched between the purlin and the panels, the height of the clip may be extended to increase the distance between the top of the purlin and the underside of panels. This distance is referred to as standoff and was varied in the tests from 0 to 1.5 in.
- 7) Insulation. Two specimens were constructed with blanket insulation, one of which also included thermal blocks.
- 8) Purlin spacing. Only one specimen used purlins spaced at 2.5 ft and the rest all had a typical purlin spacing of 5 ft.

Table 6-1 SSR Test Matrix

Specimen	Panel Profile	Manu- facturer	Panel Gauge	Panel Width (in.)	Clip Type	Standoff (in)	Insulation	Purlin Spacing (ft)
1	Trapezoidal	TS-324	24	24	Fixed	0	No	5
2	Trapezoidal	TS-324	24	24	Floating	0.5	No	5
3	Trapezoidal	MSC	24	24	Sliding tab	0.4	No	5
4	Vertical	VS-216	24	16	Fixed	0	No	5
5	Vertical	VS-216	24	16	Floating	3/8	No	5
6	Vertical	MVP	24	16	Floating	0.4	No	5
7	Trapezoidal	TS-324	24	24	High Fixed	0.5	No	5
8	Trapezoidal	TS-324	24	24	High Floating	1.5	No	5
9	Trapezoidal	TS-324	24	24	High Floating	1.5	9 in. insulation 1 in. thermal block	5
10	Trapezoidal	MSC	24	24	Sliding tab	0.4	6 in. insulation no thermal block	5
11	Trapezoidal	TS-324	24	24	Fixed	0	No	2.5

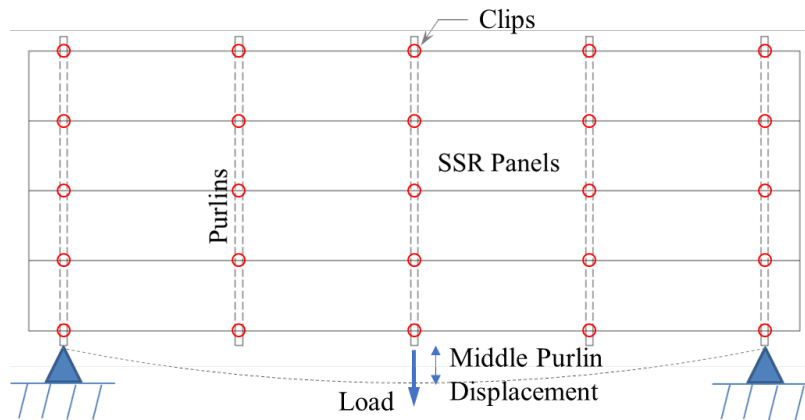


Figure 6-3 Typical specimen layout

SS2-4A5CD5F4-
24GA TS-324 Panel
Galv- Clear Acrylic



Specimens 1, 2, 7, 8, 9, 11

MSC (trapezoidal)-
24"- Roof Panel



Specimens 3, 10

SS14A5A7975--24
GA VS-216 Panel
Galv-Acrylic Coated



Specimens 4, 5

MVP (vertical) panel-
16" CT1188919RV264



Specimen 6

Figure 6-4 SSR panels used in the experimental program

Fixed - FC461 0
standoff



Specimens 1, 11
(with TS-324)

Floating - MPS 602
0.5" standoff



Specimen 2
(with TS-324)

Sliding tab - STC
0.5" standoff



Specimens 3, 10
(with MSC)

Fixed - FC10200
0 standoff



Specimen 4
(with VS-216)

Floating - MC 1203
3/8" standoff



Specimen 5
(with VS-216)

Floating - MVP short clip
0.4" standoff



Specimen 6
(with MVP)

Fixed - FC462
0.5" standoff



Specimen 7
(with TS-324)

High Floating - MPS 603
1.5" standoff



Specimens 8, 9
(with TS-324)

Figure 6-5 Clips used in the experimental program

6.2.2. Specimen Construction

Figure 6-6 shows the process of constructing the specimens. Two C-channels with a length of 22 ft were temporarily used to space and position the purlins, which were mounted at the bottom flange of both ends to the channels with self-drilling screws (Figure 6-6a). Thermal blocks and blanket insulation (if any) were placed on the purlins (Figure 6-6e). Clips were then installed onto the top flange with self-tapping screws starting from one end of each purlin (Figure 6-6b). The SSR panels were aligned such that the leg of the panel could be attached to the clips (Figure 6-6c). Once the previous panel was in place, clips were fastened on the other edge of the panel and this process was repeated until all the panels and clips were installed (Figure 6-6d, 6f).

A final step of construction before testing each of these specimens was to seam the panels as shown in Figure 6-7. A professional roof seamer assisted with the seaming process. A hand seamer was used to create a starting seam at one end (Figure 6-7a, 7b) followed by running a self-propelling roll forming machine to seam the entire legs of the adjacent panels (Figure 6-7c, 7d). Depending on the panel profiles, a single lock (e.g. Specimen 6) or a double lock (e.g. Specimen 4) standing seam was made. Table 6-2 shows the average and standard deviation of seam width measured with a digital caliper at six arbitrary locations (on clip or off clip) of each specimen.



Figure 6-6 Specimen construction process

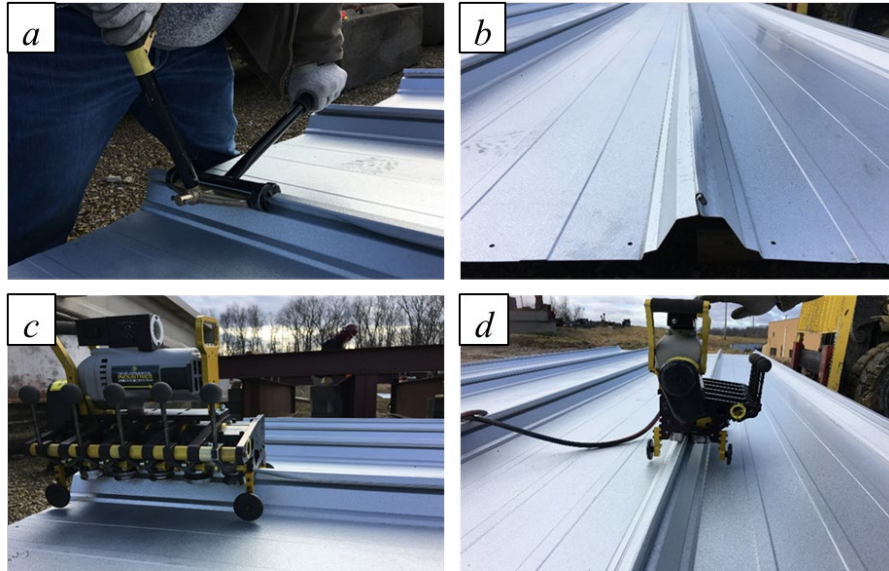


Figure 6-7 Pictures showing the seaming process in sequential order

Table 6-2 Measured Seam Width

Specimen	On clip		Off clip	
	avg (in.)	std (in.)	avg (in.)	std (in.)
1	0.500	0.007	0.459	0.006
2	0.484	0.002	0.444	0.002
3	0.370	0.014	0.321	0.004
4	0.471	0.009	0.454	0.004
5	0.478	0.007	0.451	0.005
6	0.304	0.012	0.338	0.005
7	0.501	0.017	0.488	0.016
8	0.506	0.017	0.492	0.024
9	0.498	0.030	0.491	0.018
10	0.370	0.012	0.323	0.005
11	0.498	0.018	0.447	0.023

6.2.3. Test Setup

A reaction frame was built to test the SSR specimens as shown in Figure 6-8. It consisted of five 36-inch tall built-up beams spaced at 5 ft with two 12-inch tall built up beams attached to the

top flanges at both ends. Fixed supports for the exterior purlins were made with angles (see Figure 6-9) that were bolted to the reaction beams and the purlins to simulate the point bracing location (i.e. hard point) at the ends of the roof X bracing. For interior purlins, angle supports with slots to facilitate the sliding of purlins were attached to the interior reaction beams (see Figure 6-10). A hydraulic jack with a capacity of 25 kips was connected to the web of the reaction beam and the other end of the jack was attached to a load cell and then two 60 in.×6 in.×¼ in. plates which were used to sandwich and bolt to the web of the middle purlin. The orientation of the clips was kept consistent for all the specimens with the base of the clips facing the hydraulic jack. A view of the reaction frame is also shown in Figure 6-11.

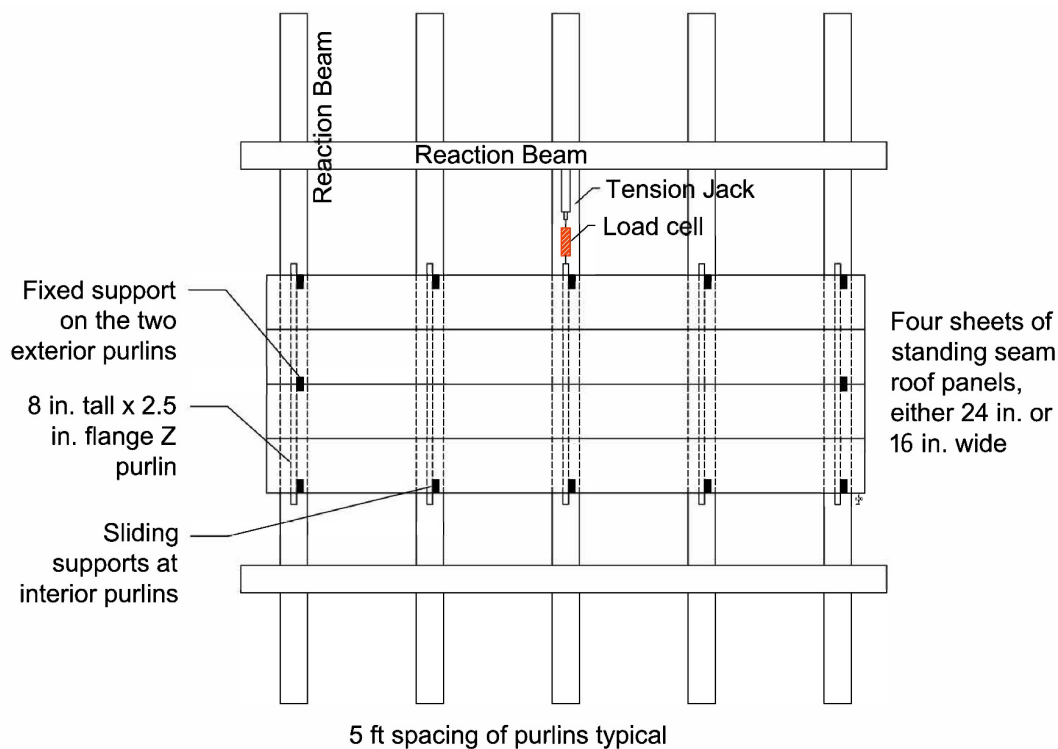


Figure 6-8 Schematic view of test setup

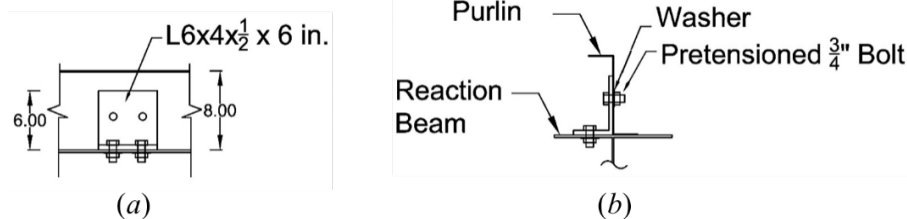


Figure 6-9 Details of fixed support

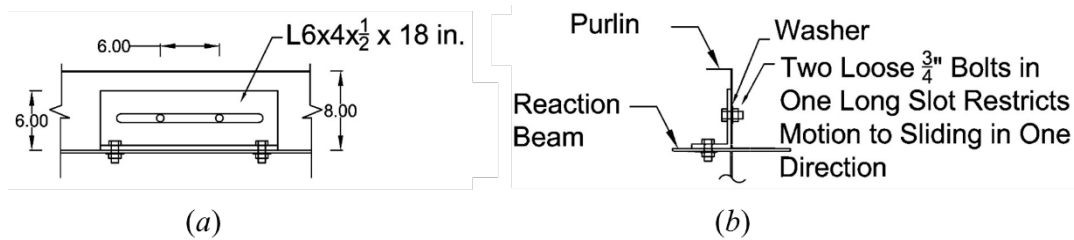


Figure 6-10 Details of sliding support



Figure 6-11 Reaction Frame

6.2.4. Instrumentation

A total of ten string potentiometers were used as shown in Figure 6-12. Five of them (SP01 to SP05) measured the longitudinal displacement of each purlin relative to the ground. Two string potentiometers measured the relative displacement between the middle purlin and the SSR panels (SP09 and SP10). The other three string potentiometers measured the relative slip between adjacent SSR panels (SP06 to SP08). A picture of the complete test setup with instrumentation is shown in Figure 6-13.

6.2.5. Loading Protocol

A tension load was applied by the hydraulic jack to the north end of the middle purlin. The load was applied incrementally with a hydraulic pump followed with a waiting period of approximately 10 seconds to allow the deformation of the specimens to stabilize. Each test was considered completed if any of the following conditions occurred: 1) the specimen failed with substantial loss of strength (typical); 2) any of the displacement transducers reached the limit of

their stroke (e.g. Specimens 2 and 10); 3) the bolt on the middle purlins reached the end of the sliding support (e.g. Specimen 7).

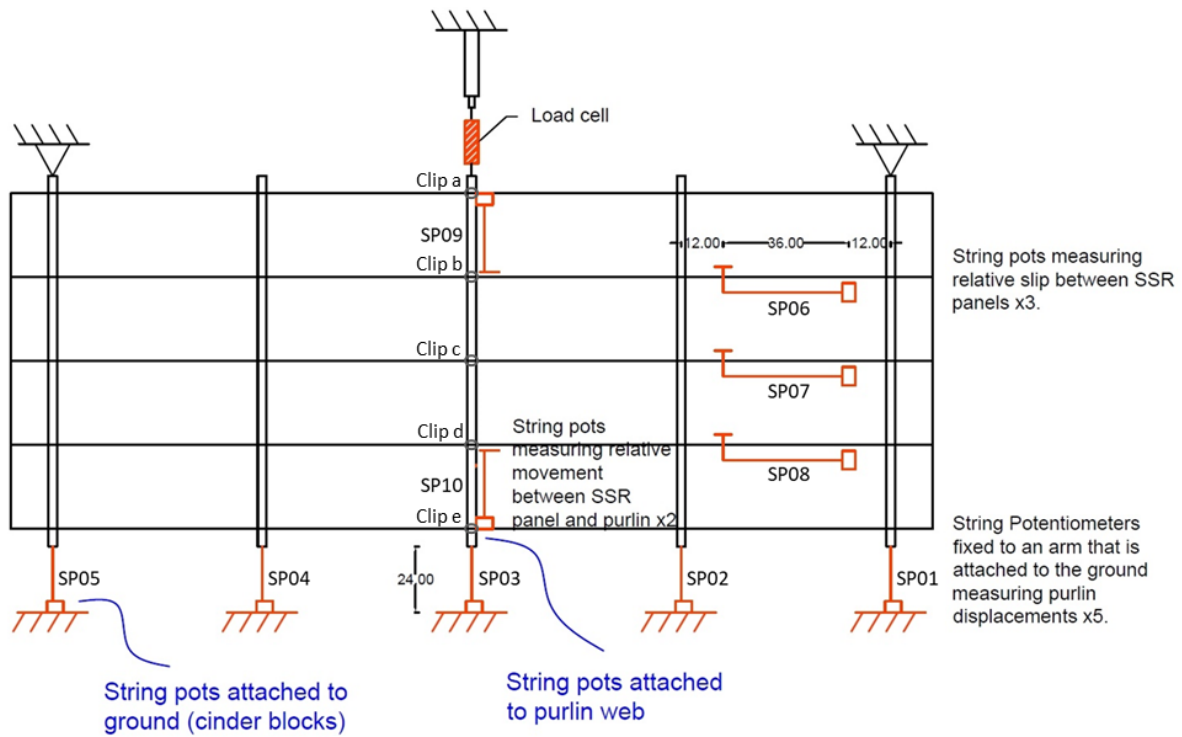


Figure 6-12 Schematic view of instrumentation layout



Figure 6-13 Picture of test setup and specimen prior to testing

6.3. Results and Discussion

Data of applied load and displacements were collected during the tests. Results are presented and discussed in this section. More test results are given in Appendix B.

6.3.1. Test Results

The load path for the specimen, as supported by the deformed shape of specimens, was: 1) load was applied to the middle purlin, 2) load transferred to the SSR panels through clips on the middle purlin, 3) SSR panels transferred load from the middle to the edges through bending and shear of the panels, 4) load transferred to the exterior purlins through clips, and 5) load transferred from exterior purlins to reaction frame. All specimens exhibited a similar progression of behavior. In the early stage of the load-deformation response, the majority of the deformation resulted from bending of the clips (see Figure 6-14). After the clips bent such that the clip leg made an acute angle with the SSR panels, the clip tension contributed to the in-plane load resistance and there was an increase in specimen stiffness. This phenomenon was most notable with clips having increased standoff. Panel buckling occurred (Figure 6-15) with increasing deformation of the clips, resulting in a minor drop of load. As the applied displacement increased, clips could be mechanically detached from the panel seams at the top or bottom depending on the type of clips, or fractured at the bottom due to the bearing of screw fasteners against holes on the base of the clips (as shown in Figure 6-16 and 17), which caused substantial loss of strength and led to the failure of the specimen.



Figure 6-14 Typical deformed configuration with clip bending (from Specimen 7)



Figure 6-15 Typical deformed shape after panel buckling (from Specimen 7)



Figure 6-16 Mechanical detachment of clips from top (left) and bottom (right) (from Specimen 8)



Figure 6-17 Tearing of clip material at the bottom (from Specimen 7)

Figure 6-18 to 28 show the applied load versus middle purlin displacement curve and the progression of failure for each specimen. Clips a, b, c, d, and e refer to the clips attached to the middle purlin in sequential order with Clip a being the one closest to the hydraulic jack (see Figure 6-12). Displacements of the middle purlin when clips failed are given in Table 6-3, and the load at

which the first clip failed for each specimen is also provide which can be considered as the lower bound strength of each clip configuration.

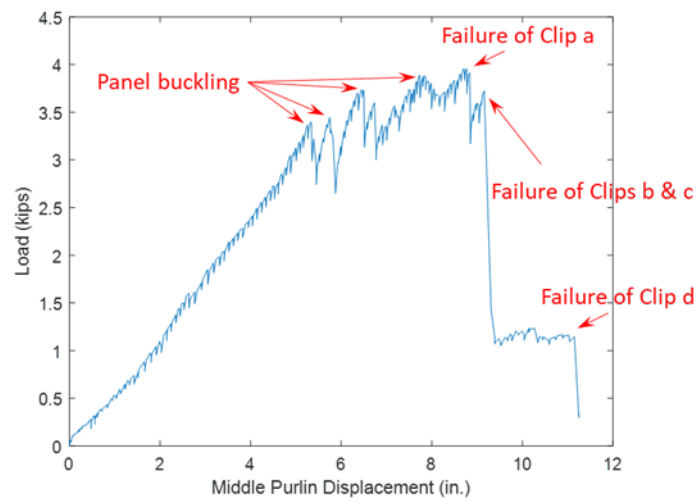


Figure 6-18 Load-displacement curve of Specimen 1

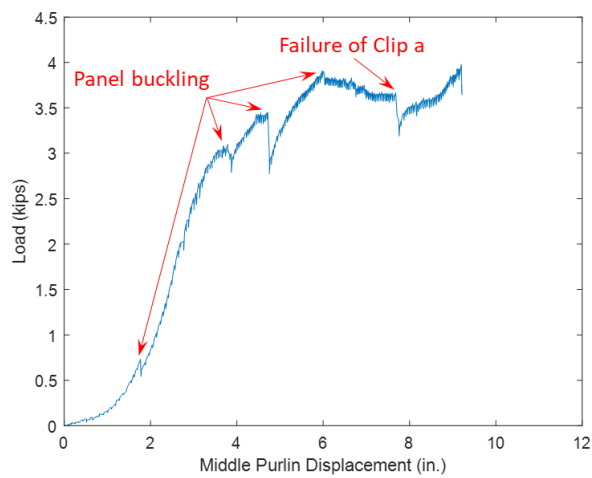


Figure 6-19 Load-displacement curve of Specimen 2

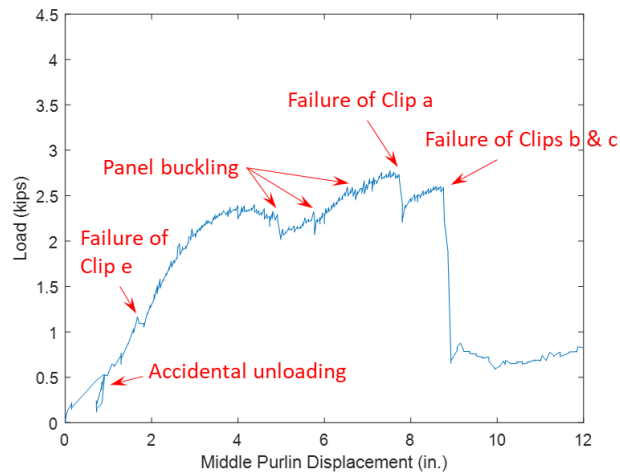


Figure 6-20 Load-displacement curve of Specimen 3

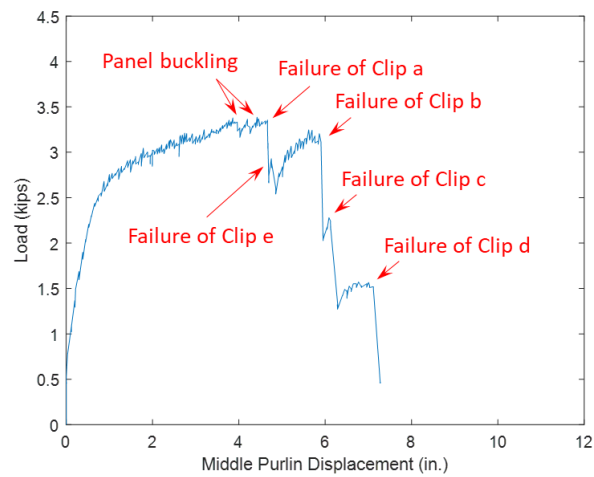


Figure 6-21 Load-displacement curve of Specimen 4

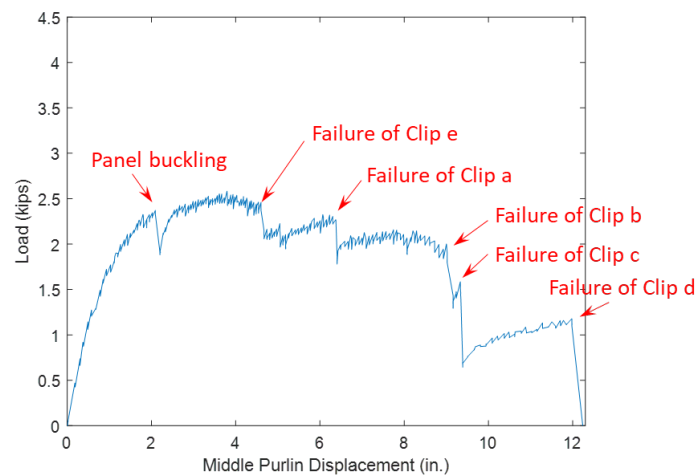


Figure 6-22 Load-displacement curve of Specimen 5

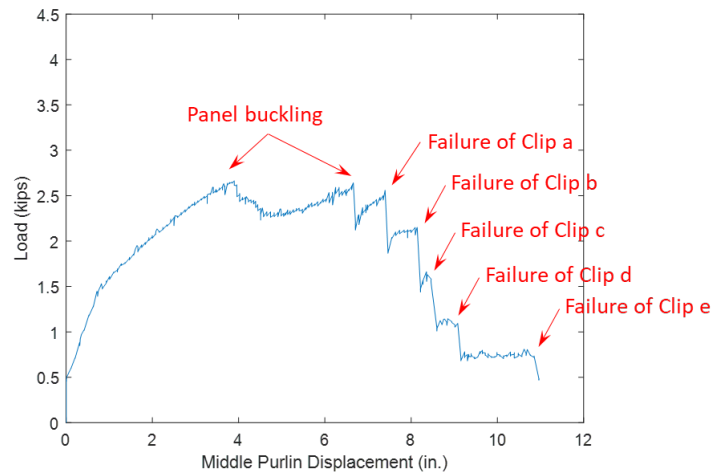


Figure 6-23 Load-displacement curve of Specimen 6

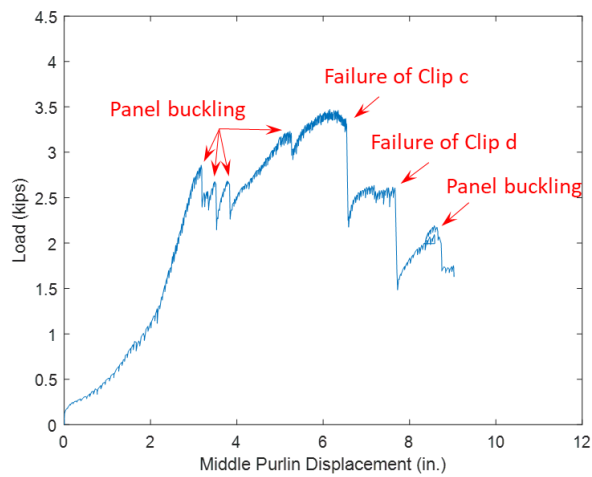


Figure 6-24 Load-displacement curve of Specimen 7

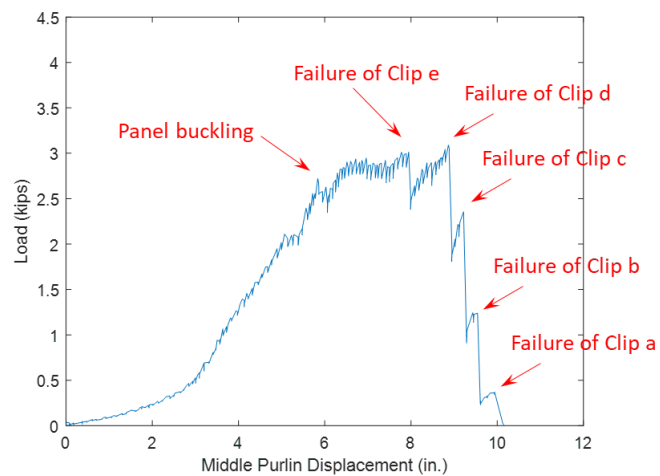


Figure 6-25 Load-displacement curve of Specimen 8

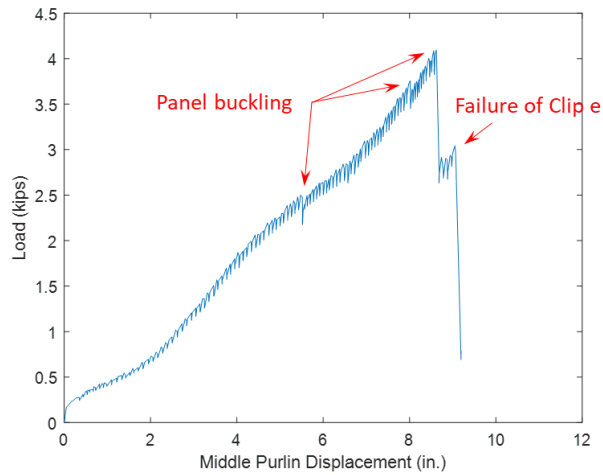


Figure 6-26 Load-displacement curve of Specimen 9

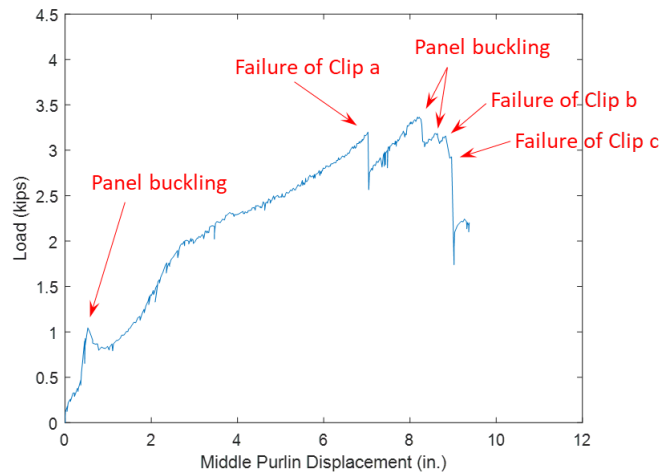


Figure 6-27 Load-displacement curve of Specimen 10

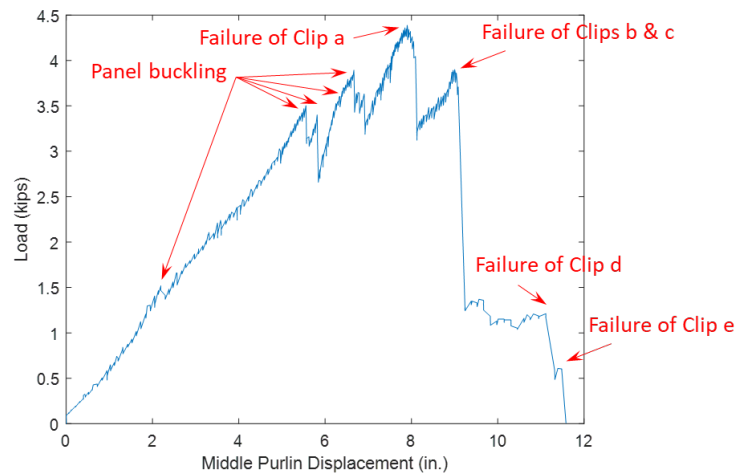


Figure 6-28 Load-displacement curve of Specimen 11

Table 6-3 Displacement of Middle Purlin When Clips Failed and Load at First Clip Failure

Specimen	Middle Purlin Displacement at Clip Failure (in)					Load at First Clip Failure (kip)
	Clip a	Clip b	Clip c	Clip d	Clip e	
1	8.84	9.16	9.16	11.15	-	3.96
2	7.69	-	-	-	-	3.63
3	7.73	8.76	8.76	-	1.67	1.17
4	4.67	5.90	6.12	7.12	4.75	3.35
5	6.37	9.00	9.33	11.98	4.60	2.45
6	7.40	8.14	8.46	9.08	10.85	2.56
7	-	-	6.55	7.66	-	3.31
8	9.94	9.54	9.22	8.89	7.95	3.01
9	-	-	-	-	9.05	3.04
10	7.03	8.83	8.97	-	-	3.20
11	8.09	9.08	9.08	11.12	11.48	4.39

- Clip failure did not occur during the test.

Figure 6-29 shows an example of the displacement curves for all five purlins of Specimen 1 (herein denoted as P1, P2, P3, P4, and P5, corresponding to those which the string potentiometers SP01, SP02, SP03, SP04, and SP05 were attached to, respectively). It can be observed that the exterior purlins P1 and P5 exhibited nearly zero displacement at the fixed supports, while the middle purlin P3 where the load was applied experienced the largest displacement. The two interior purlins P2 and P4 on the sliding supports also moved due to the rigid body movement and in-plane bending of the SSR panels. During the early stages of a typical test, the displacement of P3, δ_3 , was found to be approximately three times the displacement of P2, δ_2 , or the displacement of P4, δ_4 . This is demonstrated by the following calculations where clip deformation dominated the displacements at the beginning of the test while the roof panels moved effectively as a rigid body.

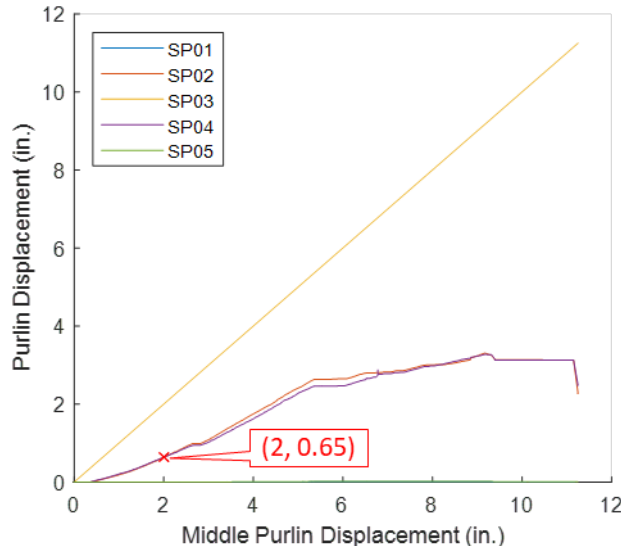


Figure 6-29 Example displacement curves for all purlins (from Specimen 1)

A free-body diagram of the clips and panels is shown in Figure 6-30. There are five clips at each purlin, and therefore the stiffness of the clips is shown as $5k_c$, where k_c is the stiffness of one clip. This is a simplifying assumption that all clips contribute equal stiffness even though edge clips may not be as well constrained as interior clips and thus may have lower stiffness in reality.

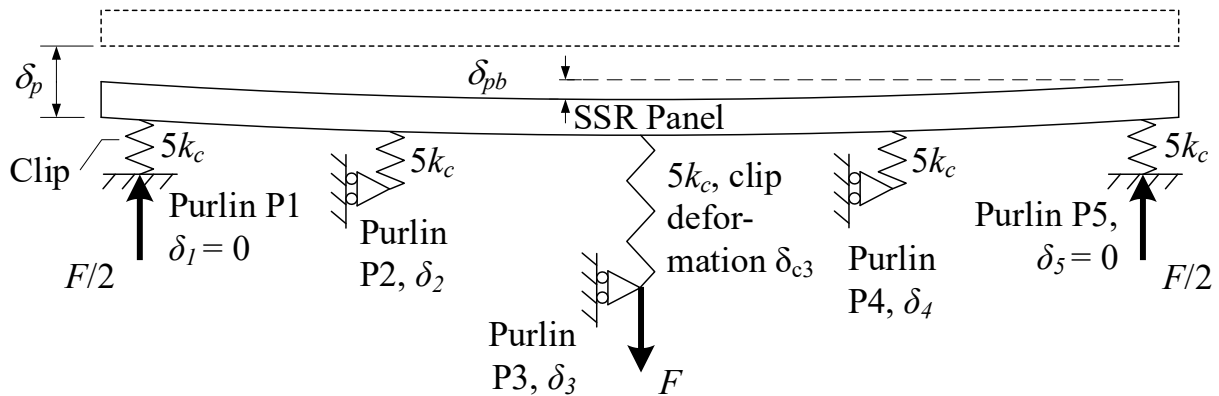


Figure 6-30 Free body diagram of clips and panels

As the specimen is subjected to a load, F , at the middle purlin, the corresponding displacement at the middle purlin, δ_3 , can be calculated as the sum of the clip deformation at purlin P3, δ_{c3} , the midspan deformations of the panel, δ_{pb} , and the clip deformation at the edge purlins (purlin P1 or P5) which is also equal to the panel rigid body displacement, δ_p .

$$\delta_3 = \delta_{c3} + \delta_{pb} + \delta_p \quad (6-1)$$

The clip deformation on the middle purlin, δ_{c3} , is given by:

$$\delta_{c3} = \frac{F}{5k_c} \quad (6-2)$$

During the early stage of the tests, the SSR panels acted as a rigid body and the deformation of the panels due to in-plane bending was negligible compared to the clip deformation. Hence it is assumed that the panel deformation δ_{pb} due to in-plane bending is zero:

$$\delta_{pb} = 0 \quad (6-3)$$

The rigid body displacement of the panels, δ_p , is equal to the deformation of the clips on the exterior purlins at fixed supports and can be obtained by:

$$\delta_p = \frac{F/2}{5k_c} = \frac{F}{10k_c} \quad (6-4)$$

The displacement of the middle purlin can then be obtained by substituting Eq. (6-2), Eq. (6-3), and Eq. (6-4) into Eq. (6-1):

$$\delta_3 = \frac{F}{5k_c} + 0 + \frac{F}{10k_c} = \frac{3F}{10k_c} \quad (6-5)$$

Purlins P2 and P4 were attached to sliding supports with zero reaction force parallel to the purlin direction. Therefore, clips on these purlins exhibited negligible deformation and the displacements of the purlins are equal to the panel rigid body displacement:

$$\delta_2 = \delta_4 = \delta_p = \frac{F}{10k_c} \quad (6-6)$$

The relationship between the displacements of purlin P3 as compared to the displacement of purlins P2 and P4 is therefore given by:

$$\frac{\delta_3}{\delta_2} = \frac{\delta_3}{\delta_4} = \frac{3F/10k_c}{F/10k_c} = 3 \quad (6-7)$$

This explains why the displacement of the middle purlin P3 was observed to be approximately three times larger than P2 and P4. This also verifies that almost all of the deformation came from the clips in early part of the loading.

Using the displacement data collected by string potentiometers SP03, SP09 and SP10, clip deformation, δ_{c3} , and panel displacement, δ_p , could be obtained by $\delta_{c3} = \text{avg}(\delta_{SP09}, \delta_{SP10})$ and $\delta_p = \delta_{SP03} - \delta_{c3}$. Due to panel buckling and rotation of the attachment points on the panels where the string of potentiometers SP09 and SP10 were attached, only the portion of the data collected by these potentiometers at the early stage of the tests (e.g. before middle purlin displacement reached 2 in.) is trusted and included in the plots. As shown in Figure 6-31, clip deformations were larger than panel displacement at the beginning of the tests, and for most of the specimens the ratio of clip deformation to panel displacement is close to 2, which is the theoretical value given by the ratio of Eq. (6-2) to Eq. (6-4). This indicates that the low initial stiffness of the specimens was primarily due to the deformation of the clips.

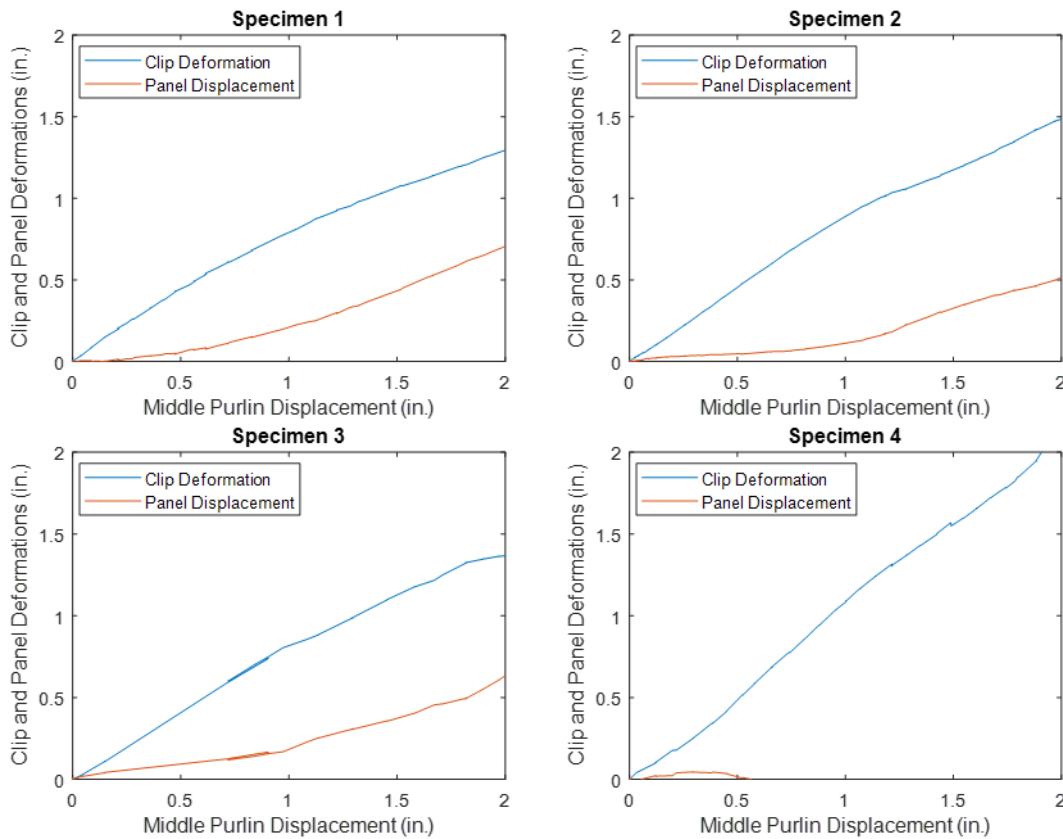


Figure 6-31 Middle purlin clip and panel deformation curves

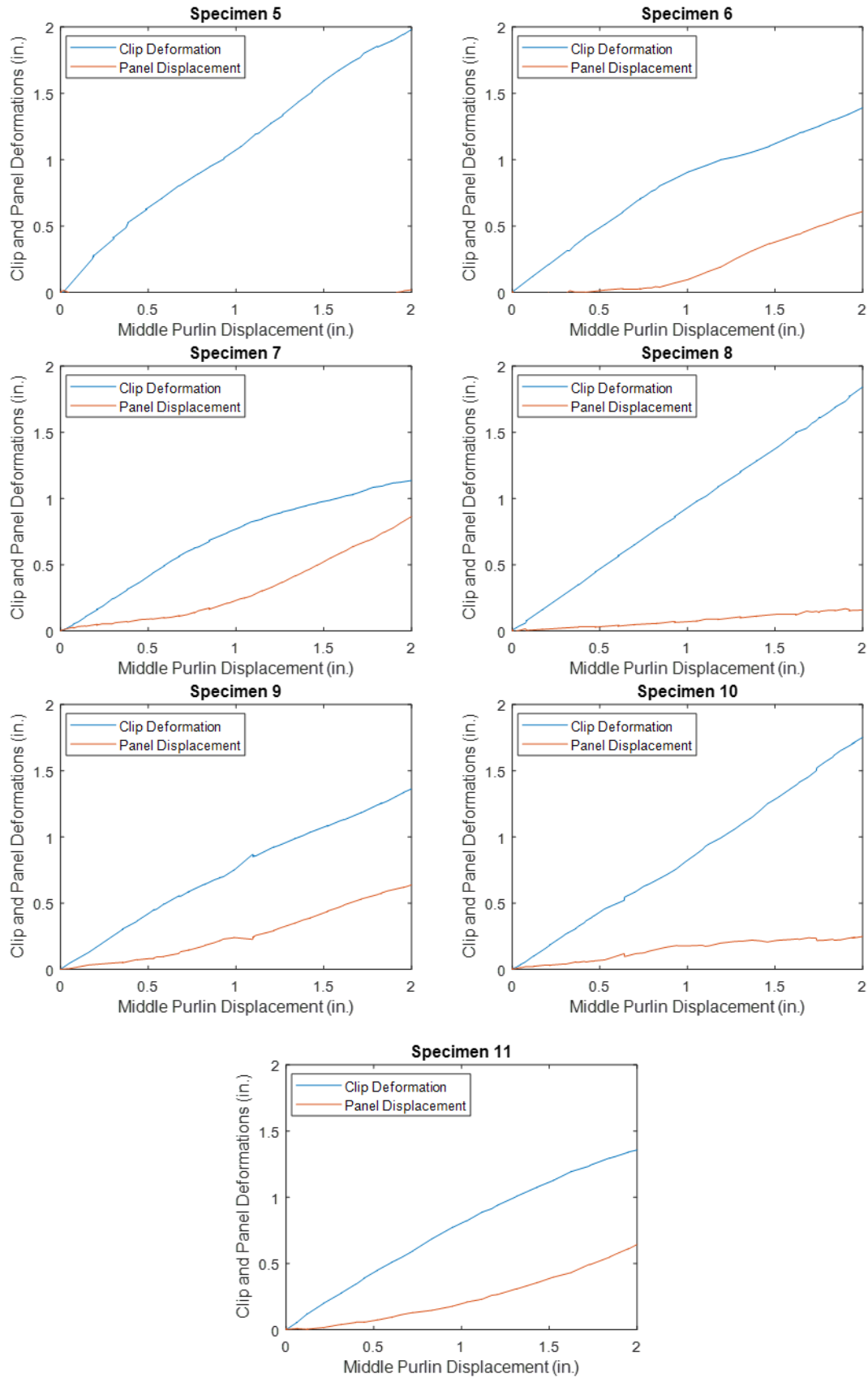


Figure 6-31 (continued) Middle purlin clip and panel deformation curves

As described in Section 6.2, the goal of string potentiometers SP06, SP07, and SP08 was to measure the relative slip between adjacent panels. However, because of panel buckling and panel deformations, the data collected using these potentiometers were found to be inaccurate during the parts of the test with large load (Figure 6-32). Instead, marks were made across the standing seams between adjacent panels so that panel slip could be visually observed and manually measured. The slip was zero or too small to be observed except for Specimens 5 and 6 which had approximately 0.5 in. panel slip at the exterior purlin location, as is shown in Figure 6-33.

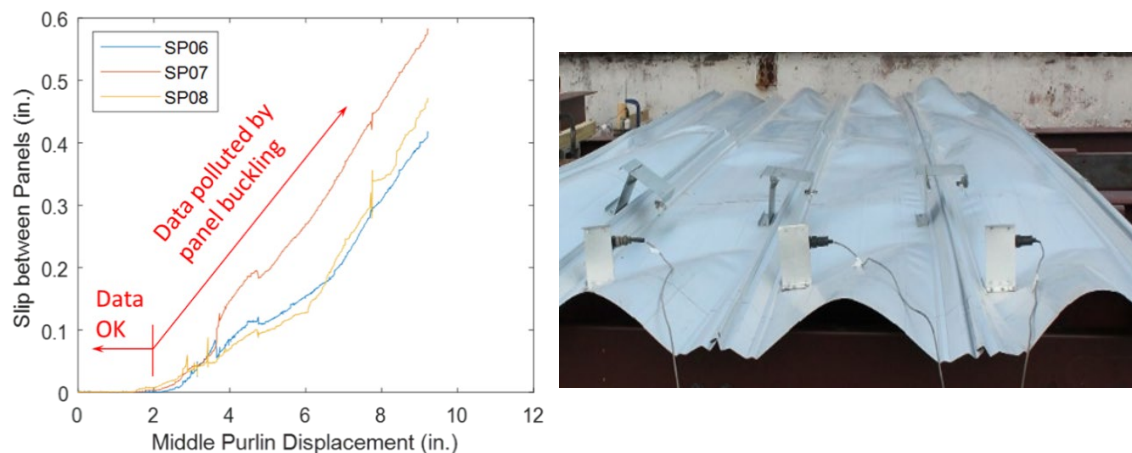


Figure 6-32 Panel slip measurement affected by panel buckling (from Specimen 2)

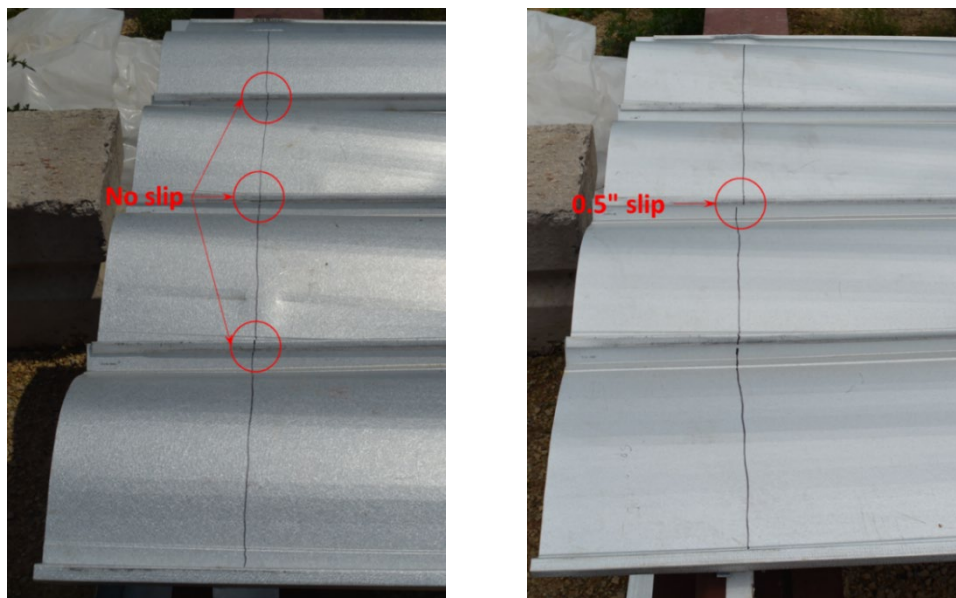


Figure 6-33 Slip between adjacent panels (left: Specimen 4; right: Specimen 5)

To quantify the stiffness, K , of the specimens, two approaches were used. Typical practice for steel deck diaphragms is to define stiffness as the secant stiffness through a point on the load-deformation curve at 40% of the maximum load. Because many of the specimens had load-deformation curves with very low initial stiffness that increased (due to geometric hardening), this approach leads to stiffness that is not representative of the low initial stiffness. The initial stiffness was therefore calculated as 1) the secant stiffness through a point on the load-deformation curve at 40% of the maximum load, $K_{40\%}$, and 2) the secant stiffness through a point on the load-deformation curve at 1 in. middle purlin displacement, $K_{1in.}$. Then, the specimen stiffness, K , is taken equal to $K_{40\%}$ for specimens whose middle purlin displacement associated with 40% peak load is smaller than 1 in. (Specimens 4, 5 and 6), and equal to $K_{1in.}$ for all other specimens (Specimens 1, 2, 3, 7, 8, 9, 10 and 11). Table 6-4 provides the values of the maximum load, F_{max} , and secant stiffnesses of the specimens. Visualization of the secant line used to determine the stiffness of the specimens is provided in Appendix B.

Table 6-4 Maximum Load and Secant Stiffness of Specimens

Specimen	F_{max} (kip)	$K_{40\%}$ (kip/in)	$K_{1in.}$ (kip/in)	K (kip/in)
1	3.96	0.612	0.505	0.505
2	3.98	0.648	0.145	0.145
3	2.78	0.675	0.545	0.545
4	3.39	8.21	2.74	8.21
5	2.58	2.35	1.73	2.35
6	2.66	2.28	1.62	2.28
7	3.47	0.619	0.478	0.478
8	3.09	0.316	0.087	0.087
9	4.09	0.441	0.402	0.402
10	3.37	0.711	0.879	0.879
11	4.39	0.643	0.573	0.573

6.3.2. Effect of Different Parameters

The effect of different parameters (panel type, clip type, thermal insulation, and purlin spacing) on the strength and stiffness of the specimens is discussed in this section. The secant stiffness K is used as a representative stiffness in the following discussion.

6.3.2.1. Panel Type

Table 6-5 and Figure 6-34 show the test results for specimens with different panel types. It can be observed that Specimen 4 with vertical rib panel profile had a smaller strength and higher stiffness compared to Specimen 1 with trapezoidal rib panel profile. The specimens with vertical rib panels exhibited greater stiffness because the vertical ribs better stiffened and restrained the bending deformation of the clips (see Figure 6-35), and the heights of the ribs and clips were potentially smaller, forcing the clips to deform in tension sooner. The trapezoidal rib panel profile exhibited larger strength because the clips for trapezoidal rib panels had small hooks on the top attached to the panels, which postponed the detachment of clips from the panel seam and contributed to a larger strength compared to the vertical ribs attached to clips without hooks.

Table 6-5 Maximum Load and Secant Stiffness $K_{40\%}$ of Specimens with Different Panel Types

Specimen	Panel Profile	Panel Width (in.)	Clip Type	Standoff (in.)	Insulation	Purlin Spacing (ft)	F_{max} (kip)	K (kip/in)
1	Trapezoidal	24	Fixed	0	No	5	3.96	0.505
4	Vertical	16	Fixed	0	No	5	3.39	8.21

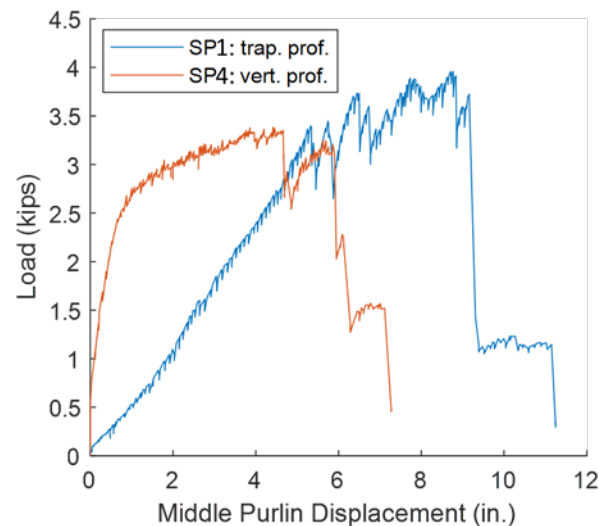


Figure 6-34 Load-displacement curves of specimens with different panel types

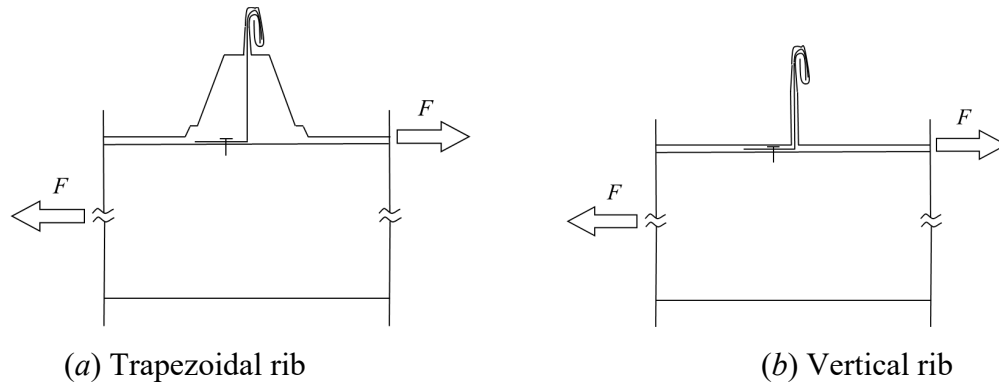


Figure 6-35 Stiffening and restraint of clips by standing seams with different profiles

6.3.2.2. Clip Type

Table 6-6 and Figure 6-36 show the test results for specimens with different clip types. For trapezoidal rib panels, fixed clips with standoff of 0 in. or 0.5 in. had similar strength and stiffness. However, for floating clips, the stiffness was sensitive to the clip standoff, and it decreased as the clip standoff increases. This is because a larger clip standoff allows the clips to deform more, leading to a lower stiffness of the specimen.

It is also noted that Specimen 3 had a lower strength and slightly higher stiffness than other specimens with trapezoidal rib panel profile, which may be attributed to its different clip construction. Because the section of the clips in Specimen 3 that was embedded in the seam was smaller, the sliding tab might pull out prematurely, resulting in a reduced strength. However, the bases of the clips were thicker, which may contribute to a higher stiffness.

For vertical rib panels, even 0.4 in. standoff had a large effect on stiffness and Specimens 5 and 6 had less than 30% of the stiffness of Specimen 4 with zero standoff. The amount that the vertical rib restrains clip deformation is less effective if there is a standoff.

Table 6-6 Maximum Load and Secant Stiffness $K_{40\%}$ of Specimens with Different Clip Types

Specimen	Panel Profile	Panel Width (in.)	Clip Type	Standoff (in.)	Insulation	Purlin Spacing (ft)	F_{max} (kip)	$K_{40\%}$ (kip/in)
1	Trapezoidal	24	Fixed	0	No	5	3.96	0.505
7	Trapezoidal	24	High Fixed	0.5	No	5	3.47	0.478
3	Trapezoidal	24	Floating	0.4	No	5	2.78	0.545
2	Trapezoidal	24	Floating	0.5	No	5	3.98	0.145
8	Trapezoidal	24	High Floating	1.5	No	5	3.09	0.087
4	Vertical	16	Fixed	0	No	5	3.39	8.21
5	Vertical	16	Floating	3/8	No	5	2.58	2.35
6	Vertical	16	Floating	0.4	No	5	2.66	2.28

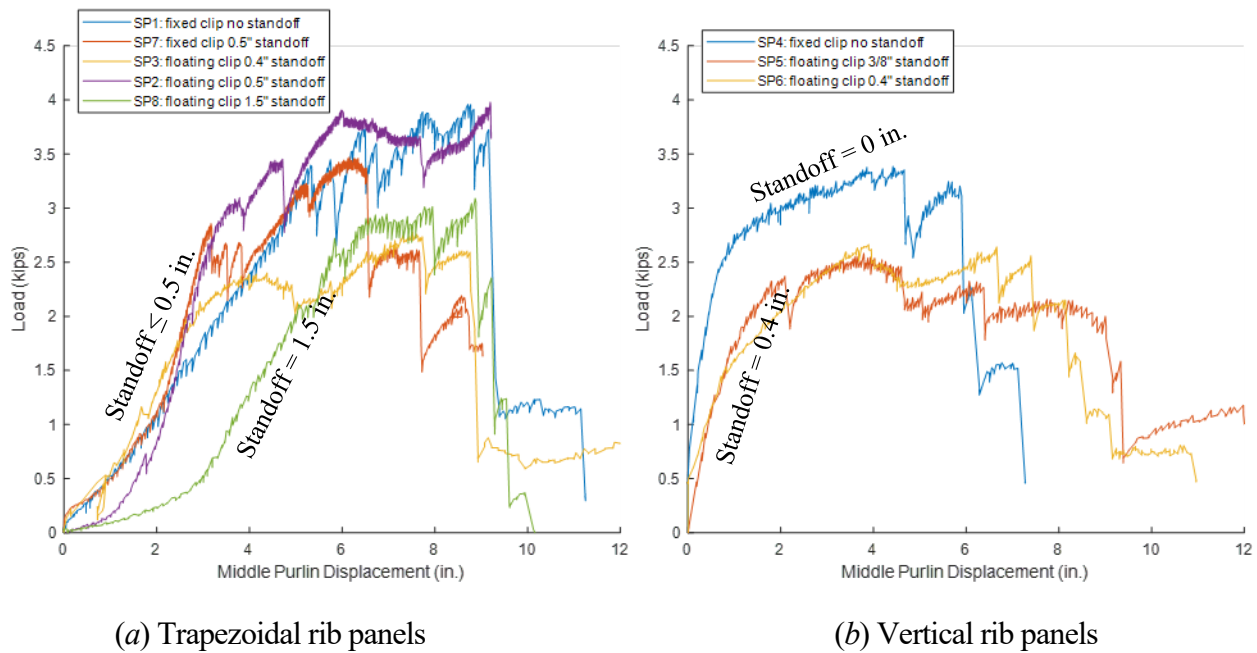


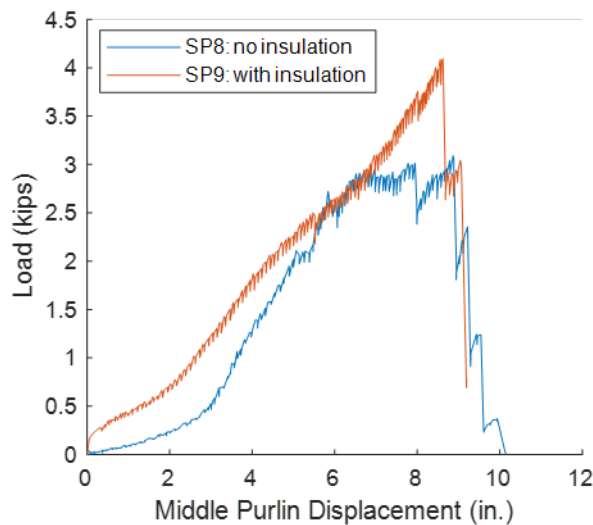
Figure 6-36 Load-displacement curves of specimens with different clip types

6.3.2.3. Thermal Insulation

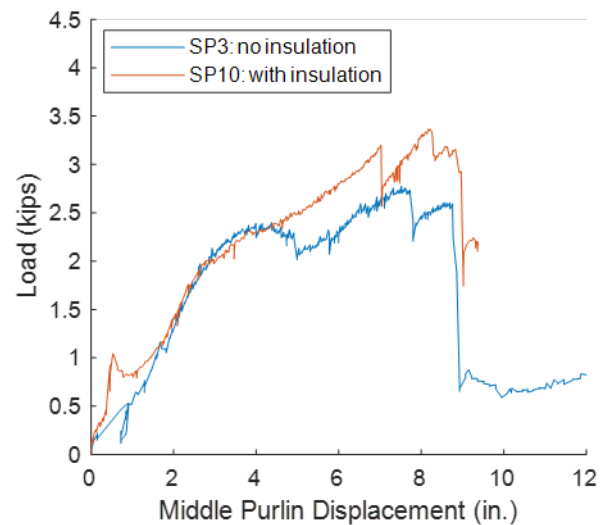
Table 6-7 and Figure 6-37 show the test results for specimens with and without thermal insulation. It is observed that including thermal insulation increases the strength and stiffness of the specimens. This is because the thermal blocks and blanket insulation can fill the void below the panel and bring the rib into bearing earlier thereby reducing the clip rotation, which contributed to a larger strength and stiffness of the specimen.

Table 6-7 Maximum Load and Secant Stiffness $K_{40\%}$ of Specimens with or without Thermal Insulation

Specimen	Panel Profile	Panel Width (in)	Clip Type	Standoff (in.)	Insulation	Purlin Spacing (ft)	F_{max} (kip)	$K_{40\%}$ (kip/in)
8	Trapezoidal	24	High Floating	1.5	No	5	3.09	0.087
9	Trapezoidal	24	High Floating	1.5	9 in. insulation 1 in. thermal block	5	4.09	0.402
3	Trapezoidal	24	Floating	0.4	No	5	2.78	0.545
10	Trapezoidal	24	Floating	0.4	6 in. insulation no thermal block	5	3.37	0.879



(a) Floating clip with 1.5 in. standoff



(b) Floating clip with 0.4 in. standoff

Figure 6-37 Load-displacement curves of specimens with or without thermal insulation

6.3.2.4. Purlin Spacing

Figure 6-38 shows the test results for specimens with different purlin spacing. It can be observed that purlin spacing has a negligible effect on strength and stiffness of the specimens. This is because all of the force transfer occurred between the loaded purlin and the reaction purlins. Additional interior purlins were not found to significantly stiffen the SSR panels against panel buckling.

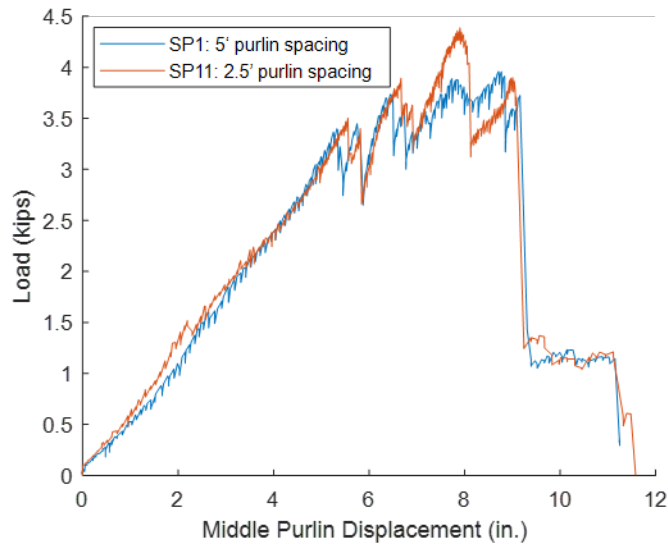


Figure 6-38 Load-displacement curves of specimens with different purlin spacing

6.4. Application in Design and Practice

6.4.1. Concepts

For metal buildings, tension rod braces (X braces) are typically used to transfer horizontal loads on the roof to the vertical system. The rod bracing coupled with the in-plane stiffness of the SSR system can also provide lateral bracing to the rafters. Figure 6-39 shows the assumed lateral displacements of a rafter as it undergoes lateral torsional buckling, wherein the rafter is constrained against translation at the intersections of rod bracing with the purlin line. Figure 6-40 shows a section view of two adjacent rafters undergoing lateral torsional buckling in the same direction. It is assumed that the SSR system provides lateral bracing for rafters between these tension rod bracing points, while the rafter flange braces restrain the twist of the rafter and thus provide torsional bracing.

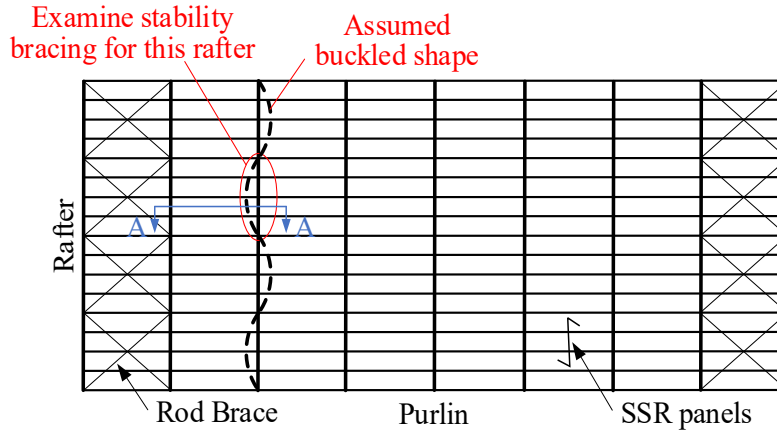


Figure 6-39 Plan view of typical metal building roof framing with assumed buckled shape of rafter

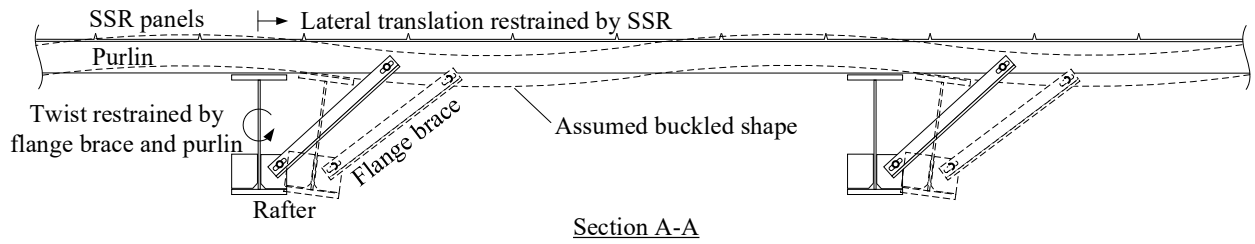


Figure 6-40 Section view of adjacent rafters braced by SSR and flange braces with assumed buckled shape

AISC 360-16 (AISC 2016a) Appendix 6 specifies the required strength and stiffness for lateral and torsional bracing of beams. The lateral bracing provided by the SSR system is categorized as point bracing at the purlin locations. The required bracing strength, P_{Lbro} , and the required bracing stiffness, β_{Lbro} , are given as follows:

$$P_{Lbro} = 0.02 \left(\frac{M_r C_d}{h_o} \right) \quad (6-8)$$

$$\beta_{Lbro} = \frac{1}{\phi} \left(\frac{10 M_r C_d}{L_{br} h_o} \right) \quad (6-9)$$

where

M_r = required flexural strength of the rafter within the panel under consideration, using LRFD or ASD load combinations

C_d = 1.0, except in the following case:

= 2.0 for the brace closest to inflection point in a beam subject to double curvature bending

h_o = distance between rafter flange centroids

ϕ = resistance factor

L_{br} = unbraced length adjacent to the point brace

Torsional bracing of the rafter, provided by the flange braces and purlins, is considered as point bracing. The required bracing strength, M_{Tbro} , and the required bracing stiffness, β_{Tbro} , are given as follows:

$$M_{Tbro} = 0.02M_r \quad (6-10)$$

$$\beta_{Tbro} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \quad (6-11)$$

where

$$\beta_T = \frac{1}{\phi} \frac{2.4L}{nEI_{yeff}} \left(\frac{M_r}{C_b}\right)^2 \quad (6-12)$$

$$\beta_{sec} = \text{infinity for a cross-frame like the flange brace-purlin assembly} \quad (6-13)$$

E = modulus of elasticity of steel = 29,000 ksi

I_{yeff} = effective out-of-plane moment of inertia = $I_{yc} + (t/c)I_{yt}$

I_{yc} = moment of inertia of the compression flange about the y -axis

I_{yt} = moment of inertia of the tension flange about the y -axis

L = length of span

C_b = lateral-torsional buckling modification factor

c = distance from the neutral axis to the extreme compressive fibers

n = number of braced points within the span

t = distance from the neutral axis to the extreme tensile fibers

β_T = overall brace system required stiffness

β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any

AISC 360-16 states that lateral bracing, torsional bracing, or a combination of the two shall be provided to prevent the relative displacement of the top and bottom flanges (i.e., to prevent twist). Lateral bracing should be attached at or near the beam compression flange to restrain its lateral movement when buckled, meaning that the lateral bracing provided by the SSR can be considered as lateral bracing for positive bending moment (top flange in compression). Since lateral bracing

and torsional bracing are provided by the SSR system and flange brace separately, AISC allows combined lateral bracing and torsional bracing using the following interaction equation given in AISC 360-16, Section 6.3 Commentary:

$$\frac{\beta_{Tbr}}{\beta_{Tbro}} + \frac{\beta_{Lbr}}{\beta_{Lbro}} \geq 1.0 \quad (6-14)$$

where β_{Tbr} and β_{Lbr} are the provided torsional and lateral bracing stiffness, respectively and β_{Tbro} and β_{Lbro} are the required torsional and lateral bracing stiffness, respectively.

For negative bending moment (bottom flange in compression), lateral force from the bottom flange of the buckled rafter is transferred through the flange brace to the purlin. In this case, the flange brace plays two roles: a load path to transfer lateral load from the bottom flange to the SSR for lateral bracing, and a component of a triangular subassemblage that supplies torsional bracing to the rafter. AISC 360-16, Section 6.3 Commentary states that when point torsional bracing is combined with lateral bracing at the tension flange, Eq. (6-14) applies and, in addition, the torsional brace stiffness should satisfy the following:

$$\beta_{Tbr} \geq \min(\beta_{Lbro} h_o^2, \beta_{Tbro}) \quad (6-15)$$

where β_{Lbro} is the required lateral bracing stiffness for point bracing as given in Eq. (6-9).

To examine the requirement for rafter bracing, it is necessary to obtain the bracing strength and stiffness provided by the purlins, SSR panels, and rafter flange braces. Since the strength and initial stiffness of the SSR specimens was dominated by the clips, it is expected that the strength and stiffness will scale proportionally with the number of clips on the purlin. The strength per clip, f_c , was calculated by dividing the maximum load of the specimens by the number of clips on the middle purlin (5 clips). For stiffness per clip, Eq. (6-5) is rearranged to solve for the stiffness per clip, k_c , as given by:

$$k_c = \frac{3F}{10\delta_3} = \frac{3}{10} K \quad (6-16)$$

where K is the stiffness of the specimen provided in Table 6-4.

Table 6-8 provides the values of the strength and stiffness per clip for the SSR test specimens where strength per clip is the recorded maximum force from the test divided by five clips and the stiffness per clip is calculated using Eq. (6-16). It is noted that the clips on the edges of the panels (e.g. clips a and e in Figure 6-12) might not contribute as much to the strength and stiffness of the specimens as the interior clips because they are not as well constrained by the SSR panels. However, they were conservatively treated as identical in the calculations, so the actual strength and stiffness per clip may be slightly larger than reported. For practical application of these results, the closest match of a building's SSR configuration to one of the specimen configurations should be identified, and the lateral bracing strength, P_{Lbr} , and the stiffness, β_{Lbr} , provided by the SSR can be obtained as follows:

$$P_{Lbr} = n_c f_c \quad (6-17)$$

$$\beta_{Lbr} = n_c k_c \quad (6-18)$$

where n_c is the number of SSR clips on one purlin over a length that is half the distance to adjacent rafters (assumes a tributary width of SSR system used to brace one rafter).

It should be noted that the resulting values for lateral bracing strength and stiffness do not consider the effect of wind and gravity loads. The testing program did not include vertical loads. Uplift associated with wind may cause a reduction in SSR lateral bracing strength because clips are subjected to additional tension and may pull out of the seam when subjected to smaller lateral loads. Conversely, the same uplift may lead to an increase in lateral bracing stiffness because of the geometric stiffening effect. Because the interaction between lateral loading and gravity / uplift loading is not well understood for clips, this interaction is neglected in this work, but recommended for future study.

Table 6-8 Provided Strength and Stiffness per Clip of SSR for Lateral Bracing of Rafter

Specimen	Configuration						f_c (kip/clip)	k_c (kip/in/clip)
	Panel Profile	Panel Width	Clip Type	Standoff	Insulation	Purlin Spacing		
1	Trap.	24"	Fixed	0"	No	5'	0.792	0.152
2	Trap.	24"	Floating	0.5"	No	5'	0.796	0.044
3	Trap.	24"	Floating	0.4"	No	5'	0.556	0.164
4	Vert.	16"	Fixed	0"	No	5'	0.678	2.463
5	Vert.	16"	Floating	3/8"	No	5'	0.516	0.705
6	Vert.	16"	Floating	0.4"	No	5'	0.532	0.684
7	Trap.	24"	High Fixed	0.5"	No	5'	0.694	0.143
8	Trap.	24"	High Floating	1.5"	No	5'	0.618	0.026
9	Trap.	24"	High Floating	1.5"	9" insul. 1" therm. blk.	5'	0.818	0.121
10	Trap.	24"	Floating	0.4"	6" insul. no therm. blk.	5'	0.674	0.264
11	Trap.	24"	Fixed	0"	No	2.5'	0.878	0.172

To obtain the torsional bracing strength and stiffness, the subassembly of the purlin, rafter and flange brace is analyzed using a truss analogy as shown in Figure 6-41. The inflection points in the purlin are assumed to be located at midspan and the rafter is assumed to be axially rigid. For an applied moment at the top of the rafter, M , the moment and axial force diagram of the structure is shown in Figure 6-42. It is noted that this model assumes a flange brace on one side, but a similar approach could be followed to determine bracing stiffness for flange braces on both sides of the rafter.

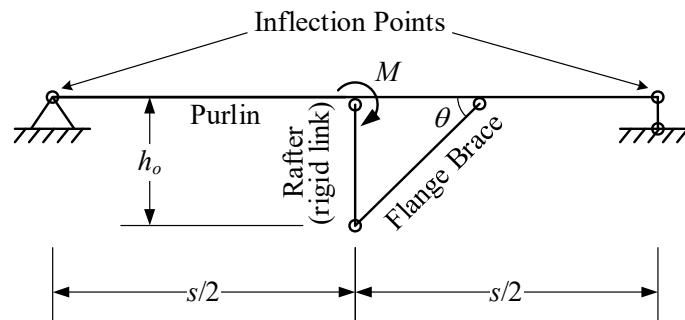


Figure 6-41 Structure assemblage for analyzing torsional bracing strength and stiffness of rafter provided by flange brace and purlin

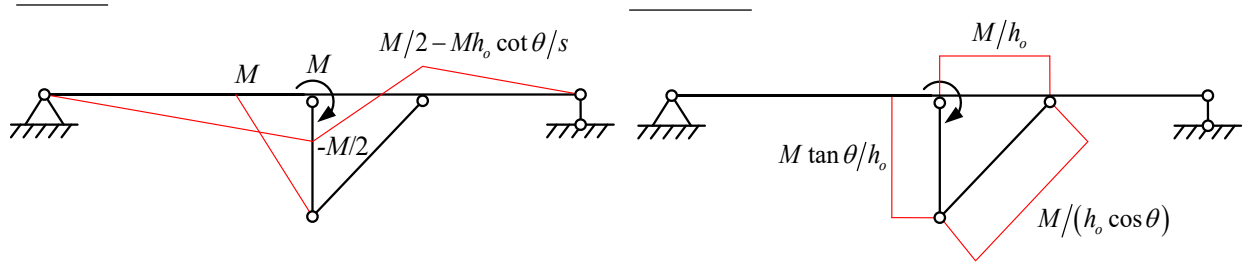


Figure 6-42 Moment and axial force diagram of the assemblage subjected to concentrated moment applied at the top of rafter

The strength of the torsional bracing provided by the flange brace and the purlin, i.e., the maximum moment that can be applied, is reached when the limit state of any component occurs. The torsional bracing strength can therefore be obtained by setting the maximum moment acting on the purlin equal to its flexural strength and by setting the maximum axial force experienced by the purlin and flange brace equal to their compressive strength (note that the applied moment can be reversed, and that the compressive strength is typically smaller than the tensile strength), which is also limited by the connections of the flange brace to the rafter and purlin, and is given by:

$$M_{Tbr} = \min(2M_{a,p}, P_{n,p} h_o, P_{n,b} h_o \cos \theta, V_{n,bolt} h_o \cos \theta) \quad (6-19)$$

where $M_{a,p}$ is the available flexural strength of the purlin to resist rafter bracing loads (after consideration of gravity and wind loads), $P_{n,p}$ is the compressive strength of the purlin between the points of rafter and flange brace attachments, $P_{n,b}$ is the compressive strength of the flange brace, and $V_{n,bolt}$ is the nominal shear strength of the bolted connections (due to bolt shear, bearing, and tear-out) at the ends of flange brace.

The torsional bracing stiffness is found by determining the rafter twist, ϕ , associated with an arbitrary applied moment, M . Using the principle of virtual work, the rotation of the rafter section when subjected to the applied moment, M , can be obtained as follows:

$$\phi = \frac{M}{EI_p} \left(\frac{s}{12} + \frac{h_o^2}{3s \tan^2 \theta} - \frac{h_o}{6 \tan \theta} \right) + \frac{M}{EA_p h_o \tan \theta} + \frac{M}{EA_b h_o \cos^2 \theta \sin \theta} \quad (6-20)$$

where I_p is the moment of inertia of the purlin, A_p is the cross-section area of the purlin, and A_b is the cross-section area of the flange brace.

Therefore, the torsional bracing stiffness provided by the flange brace and the purlin is given by:

$$\beta_{Tbr} = \frac{M}{\phi} = \frac{E}{\frac{1}{I_p} \left(\frac{s}{12} + \frac{h_o^2}{3s \tan^2 \theta} - \frac{h_o}{6 \tan \theta} \right) + \frac{1}{A_p h_o \tan \theta} + \frac{1}{A_b h_o \cos^2 \theta \sin \theta}} \quad (6-21)$$

Eq. (6-21) provides a theoretical estimate of the torsional bracing stiffness based on the flange brace configuration shown in Figure 6-41. It is noted that this equation is based on an idealized configuration and does not consider double flange braces or some sources of flexibility (e.g. connection flexibility). However, a similar procedure could be followed to obtain the torsional bracing stiffness for two-sided flange bracing systems or systems with additional sources of flexibility.

To summarize, checking the adequacy of the rafter lateral bracing can be conducted in two steps as follows:

1. Check the lateral bracing initial stiffness. Calculate the required lateral bracing stiffness, β_{Lbro} , and required torsional bracing stiffness, β_{Tbro} , using Eq. (6-9) and Eq. (6-11), respectively. Calculate the provided lateral bracing stiffness, β_{Lbr} , and provided torsional bracing stiffness, β_{Tbr} , using Eq. (6-18), and Eq. (6-21) (or equivalent approach), respectively. Substitute these four values into Eq. (6-14), and the resulting interaction inequality needs to be satisfied. For negative bending, Eq. (6-15) also needs to be satisfied.
2. Check the lateral bracing strength. Calculate the required lateral bracing strength, P_{Lbro} , and required torsional bracing strength, M_{Tbro} , using Eq. (6-8) and Eq. (6-10), respectively. Calculate the provided lateral bracing strength, P_{Lbr} , and provided torsional bracing strength, M_{Tbr} , using Eq. (6-17), and Eq. (6-19), respectively. One of the following two inequalities needs to be satisfied: $P_{Lbr} > P_{Lbro}$ or $M_{Tbr} > M_{Tbro}$.

6.4.2. Example

A prototype metal building designed by MBMA (NBM 2018) located in Orlando, Florida is selected to calculate typical values of required strength and stiffness for rafter bracing. Figure 6-43 shows the roof framing details of the prototype metal building. Rafter dimensions are: flange width

$b_f = 6$ in., thickness $t_f = 0.25$ in., web depth $h = 25$ in., and web thickness $t_w = 0.1345$ in. Rod brace spacing is 20 ft and purlin spacing is 5 ft. The SSR system has a similar configuration (panel type, clip type, etc.) to that of Specimen 1 in Table 6-8. The moment of inertia of the purlins $I_p = 10.15$ in.⁴. Bracing requirements for the rafter segment between grid lines 2-3 and E-G is evaluated in this section. It should be noted that this example only provides the check for the particular rafter segment of interest and positive moment is assumed. In practice, it would be necessary to check each bracing span individually considering all the stability requirements. Also, this example uses LRFD, but a similar procedure could be used with ASD.

Assuming that the middle portion of the rafter is buckling between the points where the diagonal rod bracing attach to the rafter (i.e. identified as “rafter segment of interest” in Figure 6-43), the unbraced length for rafter lateral bracing is the spacing of purlins, $L_{br} = 5$ ft. This is consistent with the definition of L_{br} for point bracing in AISC 360-16 Section 6.1 commentary. It is noted that the test setup used a span equal to 20 ft (corresponding to $L_{br} = 10$ ft), but because the initial stiffness of the specimens was dominated by clip deformation and not SSR panel deformations, the initial stiffness is expected to be similar if the span were half as long.

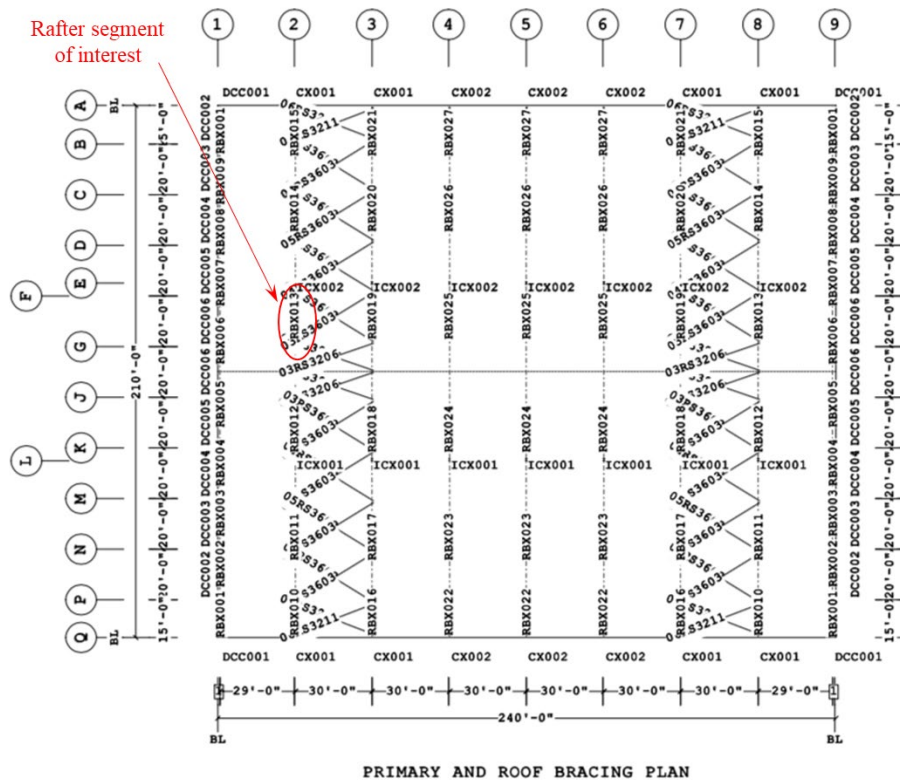


Figure 6-43 Plan view of roof framing detail of prototype metal building

It is assumed that the moment demand is $M_r = M_y = 2818$ kip-in for the rafter with compression at the top flange. With $C_d = 1.0$, and $h_o = 25.25$ in., the required strength of rafter lateral bracing is given by:

$$P_{Lbro} = 0.02 \left(\frac{M_r C_d}{h_o} \right) = 0.02 \left(\frac{(2818 \text{ kip-in.})(1.0)}{25.25 \text{ in.}} \right) = 2.2 \text{ kip} \quad (6-22)$$

The required stiffness of rafter lateral bracing is given by:

$$\beta_{Lbro} = \frac{1}{\phi} \left(\frac{10 M_r C_d}{L_{br} h_o} \right) = \frac{1}{0.75} \left(\frac{10(2818 \text{ kip-in.})(1.0)}{(5 \text{ ft})(12 \text{ in./ft})(25.25 \text{ in.})} \right) = 24.8 \text{ kip/in.} \quad (6-23)$$

The required flexural strength of rafter torsional bracing is given by:

$$M_{Tbro} = 0.02 M_r = 0.02(2818 \text{ kip-in.}) = 56.4 \text{ kip-in.} \quad (6-24)$$

For the selected rafter segment, values of the following quantities can be obtained: $I_{yeff} = I_y = 9 \text{ in.}^4$ for doubly symmetric section, $L = 20 \text{ ft}$, $n = 3$. A uniform moment is assumed with $C_b = 1.0$ which leads to $\beta_T = 7790 \text{ kip-in.}$ The flange brace and purlin can be considered as a cross frame and therefore the stiffness of the rafter web does not reduce the torsional bracing stiffness, i.e. $\beta_{sec} = \text{infinity}$. Therefore,

$$\beta_{Tbro} = \beta_T = 7790 \text{ kip-in.} \quad (6-25)$$

As is shown in Figure 6-43, the spacing of rafters is 30 ft. The spacing of the clips on the purlins is equal to the panel width (24 in.), and thus there are $n_c = (30 \text{ ft})(12 \text{ ft/in.})/(24 \text{ in.}) = 15$ clips on each purlin within one rafter spacing. Using the values for Specimen 1 in Table 6-8, the lateral bracing strength and stiffness provided by the SSR is given by:

$$P_{Lbr} = n_c f_c = (15)(0.792 \text{ kip/ft}) = 11.9 \text{ kip} \quad (6-26)$$

$$\beta_{Lbr} = n_c k_c = (15)(0.152 \text{ kip/in./ft}) = 2.28 \text{ kip/in.} \quad (6-27)$$

Assume that the flange brace is at an angle of $\theta = 45^\circ$ and the flange brace consists of an L2×2×1/8. From the AISC Steel Construction Manual (AISC 2016b) and the AISI Cold-Formed

Steel Design Manual (AISI 2017) we have: $M_{n,p} = 124$ kip-in., $P_{n,p} = 57.8$ kip, $P_{n,b} = 10.7$ kip, where $M_{n,p}$ is the nominal flexural strength of the purlin. For this example, the flexural strength of the purlin available to resist rafter bracing loads after consideration of gravity and wind loads is given as, $M_{a,p} = 37.2$ kip-in (calculations not shown here). It is also assumed that limit states associated with the bolted connections on the two ends of flange brace do not control (which may not be typical). Therefore,

$$\begin{aligned} M_{Tbr} &= \min(2M_{a,p}, P_{n,p}h_o, P_{n,b}h_o \cos \theta) \\ &= \min[2(37.2 \text{ kip-in.}), (57.8 \text{ kip})(25.25 \text{ in.}), (10.7 \text{ kip})(25.25 \text{ in.})(\cos 45^\circ)] \quad (6-28) \\ &= 74.4 \text{ kip-in.} \end{aligned}$$

The torsional bracing stiffness provided by the flange brace and the purlin is given by:

$$\begin{aligned} \beta_{Tbr} &= \frac{E}{\frac{1}{I_p} \left(\frac{s}{12} + \frac{h_o^2}{3s \tan^2 \theta} - \frac{h_o}{6 \tan \theta} \right) + \frac{1}{A_p h_o \tan \theta} + \frac{1}{A_b h_o \cos^2 \theta \sin \theta}} \\ &= 29000 \text{ ksi} \left/ \left[\frac{1}{10.15 \text{ in.}^4} \left(\frac{360 \text{ in.}}{12} + \frac{(25.25 \text{ in.})^2}{3(360 \text{ in.})(\tan^2 45^\circ)} - \frac{25.25 \text{ in.}}{6(\tan 45^\circ)} \right) \right. \right. \\ &\quad \left. \left. + \frac{1}{(1.05 \text{ in.}^2)(25.25 \text{ in.})(\tan 45^\circ)} + \frac{1}{(0.491 \text{ in.}^2)(25.25 \text{ in.})(\cos^2 45^\circ)(\sin 45^\circ)} \right] \right] \quad (6-29) \\ &= 10122 \text{ kip-in.} \end{aligned}$$

Check the adequacy of the rafter bracing:

1. Check the rafter bracing stiffness. The requirement is given as follows:

$$\begin{aligned} \frac{\beta_{Tbr}}{\beta_{Tbro}} + \frac{\beta_{Lbr}}{\beta_{Lbro}} &> 1.0 \\ \frac{10122 \text{ kip-in.}}{7790 \text{ kip-in.}} + \frac{2.3 \text{ kip/in.}}{24.8 \text{ kip/in.}} &= 1.39 \end{aligned}$$

$$1.39 > 1.0 \quad \text{OK}$$

Since this portion of the rafter is subjected to positive bending, the additional check on torsional bracing stiffness associated with negative bending is not required.

2. Check the rafter bracing strength. The requirement is that one of the following two inequalities must be satisfied:

$$P_{Lbr} \geq P_{Lbro}$$

$$11.9 \text{ kip} \geq 2.2 \text{ kip} \quad \text{OK}$$

$$M_{Tbr} \geq M_{Tbro}$$

$$74.4 \text{ kip-in.} \geq 56.4 \text{ kip-in.} \quad \text{OK}$$

The provided rafter bracing is therefore adequate.

It should be noted that this example only provides the check of bracing requirements for the particular rafter segment of interest. In practice, it would be necessary to check each bracing span individually.

6.4.3. Discussion

Based on the example calculations provided in Section 6.4.2, it is found that the lateral bracing provided by the SSR assembly can contribute to bracing the rafter against lateral torsional buckling. Rafter bracing can be provided by a combination of torsional bracing from rafter flange bracing and lateral bracing from the SSR assembly. This consideration of SSR assembly in the rafter bracing calculations may allow the frequency or size of the flange braces to be reduced. In some cases, such as SSR configurations with vertical rib panels and zero standoff clips, it may be possible to eliminate flange braces in the positive moment region of the rafter.

It is noted that the lateral bracing strength and stiffness provided by the SSR system is based on the results of the experimental study which did not include vertical loads associated with gravity loads or wind loads. As described in Section 6.4.1, gravity loads and wind loads may affect the lateral bracing stiffness and strength associated with each clip. The interaction between vertical loads and lateral bracing loads deserves further study.

6.5. Conclusions and Recommendations

6.5.1. Conclusions

In this study, 11 SSR specimens with different configurations (panel type, clip type, thermal insulation, and purlin spacing) were tested to investigate the ability of SSR panels to restrain the longitudinal movement of the purlins thus providing lateral bracing for rafters and preventing lateral torsional buckling. Based on the analysis of test results, the following observations were made:

- 1) The in-plane stiffness of an SSR system is governed primarily by bending of the clips that connect the SSR panels to the purlins.
- 2) Specimens with vertical rib panels exhibit significantly higher stiffness than those with trapezoidal rib panels because the vertical ribs provide some restraint against clip bending (i.e., shorter clear height of clip for bending). For instance, a specimen with vertical rib panels had 16 times larger stiffness than a similar specimen with trapezoidal ribs (Specimen 4 vs. Specimen 1).
- 3) The clip standoff dimension can have a substantial effect on SSR system stiffness. For trapezoidal rib panels, specimens with fixed clips and a zero or 0.5 in. clip standoff had similar stiffness (Specimens 1 and 7), but with floating clips the stiffness decreased as the clip standoff increased (Specimens 2, 3 and 8). For vertical rib panels, even a small increase in standoff can cause a substantial drop in stiffness as demonstrated by a 3.5 times drop in stiffness as the standoff is increased from 0 in. to 0.4 in. (Specimen 4 vs. Specimens 5 and 6).
- 4) Thermal blocks and blanket insulation can increase the stiffness of the SSR system. Addition of 9 in. insulation and 1 in. thermal blocks had 4.5 times larger stiffness for specimens with 1.5 in. standoff high floating clips (Specimen 9 vs. Specimen 8), and the use of 6 in. insulation (no thermal block) led to 60% increase in stiffness for specimens with shorter 0.4 in. standoff floating clips (Specimen 10 vs. Specimen 3).
- 5) For the test setup used in this study, the purlin spacing was shown to have negligible effect on strength or stiffness of the specimens.

An approach for evaluating lateral bracing requirements of a rafter considering both the flange braces and SSR lateral bracing was identified based on AISC 360-16. The requirement for rafter

lateral bracing can be satisfied by a combination of torsional bracing provided by rafter flange bracing and lateral bracing from the SSR system. If SSR panels are present on the roof, which is typical in metal buildings, considering the SSR panels may allow the frequency or size of the flange braces to be reduced. In some cases, it may even be possible to eliminate flange braces in the positive moment regions of the rafter if the SSR roof provides enough lateral bracing stiffness and strength.

6.5.2. Recommendations for Future Work

It is recommended that more experimental tests be conducted to further characterize the effect of different parameters on the in-plane strength and stiffness of SSR systems. Parameters that should be considered include: panel type (trapezoidal vs. vertical), panel width, panel thickness, manufacturer, clip type (fixed vs. floating), standoff dimension, and tightness of the seam as measured by pressure used in seaming or resulting seam thickness, among which the panel type and clip configuration may be the most influential parameters based on the limited test results from this study. The current experimental program examined eleven specific SSR assemblies, but does not give sufficient information about the many SSR combinations that are possible. Further work is required to produce generalized tables of stiffness and strength that are applicable to a broad range of SSR assemblies. Also, in future experimental programs, multiple repetitions should be included to examine the variability in results. Since the in-plane stiffness of SSR systems has been shown in the current tests to be governed almost exclusively by clip deformations and the strength of the system is associated with clip failure or clip pull-out, it may be possible to conduct (or leverage existing data from) small-scale tests on individual clips to characterize system stiffness and strength. For these types of small-scale tests to be useful, it should be verified that they can capture the clip deformation and failure modes observed in this study. Furthermore, the effect of gravity and uplift loads on the ability of the clips to resist lateral bracing loads requires further investigation. Lastly, a study is warranted on the purlin and flange brace assembly to investigate the effects of far-side and near-side flange braces for rafters, the angle of the flange braces, lapped purlins over the rafter, and the effect of gravity/uplift loads on the overall strength and stiffness of the assembly.

CHAPTER 7. SUMMARY

To address the research gaps related to common diaphragm systems in steel buildings, including concrete-filled steel deck diaphragms and bare steel deck diaphragms in a typical steel building system, and standing seam roof diaphragms in a typical metal building system, four separate studies were conducted. A summary of these four studies is provided herein, while the conclusions of each study are presented at the end of their respective chapters.

In the first study, existing data on steel deck diaphragms was analyzed for the purposes of determining acceptance criteria for linear and nonlinear analyses consistent with the performance-based seismic design levels of ASCE 41. A series of m -factors (ductility measures) and nonlinear modeling parameters (multi-linear cyclic backbone curves) were determined for bare steel deck diaphragms and concrete-filled steel deck diaphragms. The resulting analyses form the basis for new ASCE 41 provisions and are aligned with the strength and stiffness provisions provided by AISI S310 and AISC 341. The provisions are recommended for adoption in the first edition of AISC 342, which is intended to replace ASCE 41 Chapter 9 for structural steel systems.

In the second study, nonlinear finite element models that can capture the nonlinear behavior of shear stud connections and tension cracking of the concrete slab using solid elements for concrete-filled metal deck, spring elements for shear stud connections, and shell elements for steel beams, were developed and validated against experimental pushout tests and cantilever diaphragm tests. The validated modeling scheme was then applied to diaphragm configurations with various lengths of collectors and geometric irregularities such as reentrant corners and openings to further investigate the shear force distribution and the progression of shear connection failure in these types of diaphragms.

In the third study, three-dimensional building models that can capture nonlinear diaphragm behavior and its interaction with the nonlinear vertical LFRS was developed for a series of 1, 4, 8, and 12-story archetype buildings with buckling-restrained braced frames (BRBF) for the vertical system and with three designs for the diaphragms. The modeling scheme uses computationally efficient calibrated frame and truss elements to capture the realistic nonlinear behavior of both the BRBFs and the diaphragms. Modal analysis, nonlinear static pushover analyses, and nonlinear

response history analyses using 44 ground motion records scaled to three hazard levels were performed to investigate the behavior and seismic performance of the buildings.

In the fourth study, eleven full-scale in-plane bending tests on SSR systems were conducted. Each specimen was a 22 ft long SSR unit with panels and purlins that represent the portion of a roof between points where roof X bracing attaches to the rafter. Each specimen had a unique configuration to study the effect of different parameters including SSR panel profiles, SSR panel width, type of clip and screw fasteners, clip standoff, use of thermal blocks, and presence of blanket insulation. Results from the tests were analyzed and compared to the bracing requirements in AISC 360-16 for an example rafter.

REFERENCES

- ACI. (2014). *Building code requirements for structural concrete* (ACI 318-14) and commentary (ACI 318R-14). American Concrete Institute.
- AISI. (2016a). *Seismic Provisions for Structural Steel Buildings*, (AISI 341-16). American Iron and Steel Institute.
- AISC. (2016b). *Specification for structural steel buildings* (AISC 360-16). American Institute of Steel Construction. Chicago, IL: AISC.
- AISC. (2019). *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings* (AISC 342). American Institute of Steel Construction. Chicago, IL: AISC.
- AISI. (2013). *Test standard for cantilever test method for cold-formed steel diaphragms*, (AISI S907). American Iron and Steel Institute.
- AISI. (2016). *North American standard for the design of profiled steel diaphragm panels*, (AISI S310-16). American Iron and Steel Institute.
- Alashker, Y., El-Tawil, S. and Sadek, F. (2010). Progressive collapse resistance of steel-concrete composite floors. *Journal of Structural Engineering*, 136(10), 1187-1196.
- Almoney, K. and Murray, T.M., (1998). "Gravity Loading Base Tests Using Standing Seam CFR Panels and 8 in. Deep Purlins," Report No. CE/VPI-ST98/08, Charles Via Department of Civil and Environmental Engineering, Virginia Polytechnic Institute and State University.
- Androutsos, C. and Hosain, M. U. (1992). Composite beams with headed studs in narrow ribbed metal deck. In *Composite Construction in Steel and Concrete II* (pp. 771-782). ASCE.
- Ariyaratana, C. and Fahnestock, L. A. (2011). Evaluation of buckling-restrained braced frame seismic performance considering reserve strength. *Engineering Structures*, 33(1), 77-89.
- ASCE. (2016). *Minimum design loads for buildings and other structures* (ASCE standard). Reston, Va.: American Society of Civil Engineers.
- ASCE (2017). *Seismic evaluation and retrofit of existing buildings* (ASCE 41-17). Reston, Va.: American Society of Civil Engineers.
- ASTM. (2019). Standard Specification for Carbon Structural Steel, (*ASTM A36 / A36M-19*). ASTM International, West Conshohocken, PA, 2019, www.astm.org
- Atlayan, O. and Charney, F. A. (2014). Hybrid buckling-restrained braced frames. *Journal of Constructional Steel Research*, 96, 95-105.

- Avellaneda-Ramirez, R.E., Eatherton, M.R., Easterling, W.S., Hajjar, J.F., and Schafer B.W. (2021). "Experimental Investigation of Concrete on Steel Deck Diaphragms using Cantilever Diaphragm Tests". Cold-Formed Steel Research Consortium Report Series, Report Number CFSRC R-2021-02.
- Ayhan, D., Madsen, R. L. and Schafer, B. W. (2016). Progress in the Development of ASCE 41 for Cold-Formed Steel. Proceedings of the 23rd International Specialty Conference on Cold-Formed Steel Structures, Baltimore MD. 417-432.
- Ayoub, A. and Filippou, F. C. (2000). Mixed formulation of nonlinear steel-concrete composite beam element. *Journal of Structural Engineering*, 126(3), 371-381.
- Baskar, K., Shanmugam, N. E. and Thevendran, V. (2002). Finite-element analysis of steel–concrete composite plate girder. *Journal of Structural Engineering*, 128(9), 1158-1168.
- Bathgate, C.S. and Murray, T.M., (1995). "Gravity Loading of Z-Purlin Supported Starshield Building Roof Covering Systems," Report No. CE/VPI-ST95/03, Charles Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.
- Borgsmiller, J.T., Murray, T.M. and Sumner, E.A., (1994). "Gravity Loading Tests of Z-Purlin Supported SLX 264-FL Roof Covering System," Report No. CE/VPI-ST94/01, Charles Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.
- Briggs, N. E., Bond, R. B., Madhavan, M., Padilla-Llano, D. A., Coleman, K., Hug, C., Schafer, B. W., Eatherton, M. R., Easterling, W. S., and Hajjar, J. F. (2021) "Cyclic Behavior of Composite Connections in Composite Floor Diaphragms" IX Connections AISC – ECCS Workshop on Connections in Steel Structures
- Brooks, S.D. and Murray, T.M., (1990). "A Method for Determining the Strength of Z- and C-Purlin Supported Standing Seam Roof Systems," Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, October 23-24.
- Bryant, M.R., Murray, T.M. and Sumner, E.A., (1999). "Gravity Loading Base Tests Using LTC Standing Seam Panels and 8 in. Deep Z-Purlins," Report No. CE/VPI-ST99/02, Charles Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.
- Carballo, M. (1989). *Strength of Z-purlin supported standing seam roof systems under gravity loading* (Master thesis, Virginia Tech).
- Celebi, M., Bongiovanni, G., Safak, E., and Brady, A.G. (1989) "Seismic Response of a Large-Span Roof Diaphragm" *Earthquake Spectra*, Vol. 5, No. 2, pp. 337-350.

- Chen, C. H. (2010). *Performance-based seismic demand assessment of concentrically braced steel frame buildings* (Doctoral dissertation, UC Berkeley).
- Coffin, L.F. (1954). A study of the effects of the cyclic thermal stresses on a ductile metal. *Translat. ASME*, 76, 931-950.
- Dassault Systèmes Simulia. (2013). ABAQUS 6.13 User's manual. *Dassault Systems, Providence, RI*, 305, 306.
- Ding, C. (2015). *Monotonic and cyclic simulation of screw-fastened connections for cold-formed steel framing* (Master thesis, Virginia Tech).
- Drucker, D. C. and Prager, W. (1952). Soil mechanics and plastic analysis or limit design. *Quarterly of applied mathematics*, 10(2), 157-165.
- Earls, C.J., Pugh, A.D. and Murray, T.M. (1991). "Base Test for Z-Purlin Under Gravity Load With SSR System," Report No. CE/VPI-ST91/08, Charles Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.
- Easterling, W. S. (1987). *Analysis and design of steel-deck-reinforced concrete diaphragms* (Doctoral dissertation, Iowa State University).
- Easterling, W. S. and Porter, M. (1988). Composite diaphragm behavior and strength. *9th International Specialty Conference on Cold-Formed Steel Structures*, pp. 387–404
- Easterling, W. S., and Porter, M. (1994a). Steel-deck-reinforced concrete diaphragms. I. *Journal of Structural Engineering*, 120(2), pp. 560–576.
- Easterling, W. S., and Porter, M. (1994b). Steel-Deck-Reinforced Concrete Diaphragms. II. *Journal of Structural Engineering*, 120(2), pp. 577–596.
- Eatherton, M.R., Easterling, W.S., Hajjar, J.F., Schafer, B.W., Torabian, S., Coleman, K. Bond, R.B., Briggs, N.E., Fischer, A.W., Foroughi, H., Avellaneda-Ramirez, R.E., and Wei, G. (2020). SDII Year 5 Final Report (internal), Cold-Formed Steel Research Consortium Report Series.
- EERI. (1996). Northridge Earthquake Reconnaissance Report, Vol. 2 Earthquake Spectra - Supplement C to Volume 11 Earthquake Engineering Research Institute.
- Elkady, A. and D. G. Lignos. (2014). "Modeling of the Composite Action in Fully Restrained Beam-to-Column Connections: Implications in the Seismic Design and Collapse Capacity of Steel Special Moment Frames." *Earthquake Engineering and Structural Dynamics* 43 (13): 1935-1954. doi:10.1002/eqe.2430.

- Ellobody, E. and Young, B. (2006). Performance of shear connection in composite beams with profiled steel sheeting. *Journal of Constructional Steel Research*, 62(7), 682-694.
- European Committee for Standardization. (1992). Eurocode-2: Design of concrete structures, Part1: General rules and rules for building. CEN; 1992.
- Fan, J. S. and Liu, W. (2014). Tests on Shear Studs Using Profiled Steel Sheeting Subjected to Cyclic Loading. In *Applied Mechanics and Materials* (Vol. 578, pp. 196-200). Trans Tech Publications Ltd.
- Farahi, M. and M. Mofid. (2013). "On the Quantification of Seismic Performance Factors of Chevron Knee Bracings, in Steel Structures." *Engineering Structures* 46: 155-164. doi:10.1016/j.engstruct.2012.06.026.
- FEMA. (2009). Quantification of building seismic performance factors, (*FEMA P695*). Applied Technology Council, Federal Emergency Management Agency.
- Fisher, J.M., (1998). "Diaphragm Test Report on Varco-Pruden SLR Panel Roof System," CSD Report No. 980424, Computerized Structural Design, S.C, Milwaukee, WI.
- Galal, M. A., Bandyopadhyay, M. and Banik, A. K. (2019). Dual effect of axial tension force developed in catenary action during progressive collapse of 3D composite semi-rigid jointed frames. *Structures* (Vol. 19, June, pp. 507-519). Elsevier.
- Gnanasambandam, C. (1995). *Headed stud shear connectors in solid slabs and in slabs with wide ribbed metal deck* (Doctoral dissertation, University of Saskatchewan).
- Horita, Y., Tagawa, Y. H. Y. and Asada, H. (2012, September). Push-out test of headed stud in composite girder using steel deck - An effect of stud length of projecting part from steel deck on shear strength. In *Proceedings of the 15th World Conference on Earthquake Engineering (15WCEE), Lisbon, Portugal* (pp. 24-28).
- Jayas, B. S. and Hosain, M. U. (1988). Behaviour of headed studs in composite beams: push-out tests. *Canadian Journal of Civil Engineering*, 15(2), 240-253.
- Johnson, G. R. and Holmquist, T. J. (1994). An improved computational constitutive model for brittle materials. In *AIP conference proceedings* (Vol. 309, No. 1, pp. 981-984). American Institute of Physics.
- Johnson, R. P. and Yuan, H. (1998). Existing Rules and New Tests for Stud Shear Connectors in Troughs of Profiled Sheeting. *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, 128(3), 244-251.

- Katwal, U., Tao, Z. and Hassan, M. K. (2018). Finite element modelling of steel-concrete composite beams with profiled steel sheeting. *Journal of Constructional Steel Research*, 146, 1-15.
- Katwal, U., Tao, Z., Hassan, M. K., Uy, B. and Lam, D. (2020). Load sharing mechanism between shear studs and profiled steel sheeting in push tests. *Journal of Constructional Steel Research*, 174, 106279.
- Khorami, M., Alvansazyazdi, M., Shariati, M., Zandi, Y., Jalali, A. and Tahir, M. (2017). Seismic performance evaluation of buckling restrained braced frames (BRBF) using incremental nonlinear dynamic analysis method (IDA).
- Kiggins, S. and Uang, C.-M. (2006). “Reducing Residual Drift of Buckling-Restrained Braced Frames as a Dual System”, *Engineering Structures*, Vol. 28, pp. 1525-1532.
- Kircher, C., Deierlein, G., Hooper, J., Krawinkler, H., Mahin, S., Shing, B. and Wallace, J. (2010). *Evaluation of the FEMA P-695 methodology for quantification of building seismic performance factors* (No. Grant/Contract Reports (NISTGCR)-10-917-8).
- Lee, J. and Fenves, G. L. (1998). Plastic-damage model for cyclic loading of concrete structures. *Journal of engineering mechanics*, 124(8), 892-900.
- Leng, J., Buonopane, S. G. and Schafer, B. W. (2020). Incremental dynamic analysis and FEMA P695 seismic performance evaluation of a cold-formed steel-framed building with gravity framing and architectural sheathing. *Earthquake Engineering & Structural Dynamics*, 49(4), 394-412.
- Lloyd, R. M. and Wright, H. D. (1990). Shear connection between composite slabs and steel beams. *Journal of Constructional Steel Research*, 15(4), 255-285.
- Lubliner, J., Oliver, J., Oller, S. and Oñate, E. (1989). A plastic-damage model for concrete. *International Journal of solids and structures*, 25(3), 299-326.
- Luttrell, L., Mattingly, J., Schultz, W. and Sputo, T., (2015). Steel Deck Institute diaphragm design manual - 4th Edition, (DDM04). Glenshaw, Pennsylvania.
- Lyons, J. C. (1994). *Strength of welded shear studs* (Master thesis, Virginia Tech).
- Majdi, Y., Hsu, C. T. T. and Zarei, M. (2014). Finite element analysis of new composite floors having cold-formed steel and concrete slab. *Engineering Structures*, 77, 65-83.

- Malakoutian, M., J. W. Berman, P. Dusicka and A. Lopes. 2015. "Quantification of Linked Column Frame Seismic Performance Factors for use in Seismic Design." *Journal of Earthquake Engineering*: 1-24. doi:10.1080/13632469.2015.1104750.
- Manie, S., Moghadam, A. S. and Ghafory-Ashtiany, M. (2014). Collapse Safety Assessment of Low-Rise Buildings with Mass Irregularities In Plan.
- Manson, S.S. (1954). Behaviour of Materials under Conditions of Thermal Stress. NACA TN-2933. National Advisory Committee for Aeronautics.
- Martin, É. (2002). *Inelastic response of steel roof deck diaphragms under simulated dynamically applied seismic loading* (Master thesis, Ecole Polytechnique de Montreal).
- Mirza, O. and Uy, B. (2010). Effects of the combination of axial and shear loading on the behaviour of headed stud steel anchors. *Engineering Structures*, 32(1), 93-105.
- Moharrami, M. and Koutromanos, I. (2016). Triaxial constitutive model for concrete under cyclic loading. *Journal of Structural Engineering*, 142(7), 04016039.
- Moharrami, M. and Koutromanos, I. (2017). Finite element analysis of damage and failure of reinforced concrete members under earthquake loading. *Earthquake Engineering & Structural Dynamics*, 46(15), 2811-2829.
- Mottram, J. T. and Johnson, R. P. (1989). *Push tests on studs welded through profiled steel sheeting*. University of Warwick, Department of Engineering.
- Neilsen, M. (1984). *Effects of gravity load on composite floor diaphragm behavior* (Master thesis, Iowa State University).
- Nellinger, S., Odenbreit, C., Obiala, R. and Lawson, M. (2017). Influence of transverse loading onto push-out tests with deep steel decking. *Journal of Constructional Steel Research*, 128, 335-353.
- Nguyen, H. T. and Kim, S. E. (2009). Finite element modeling of push-out tests for large stud shear connectors. *Journal of Constructional Steel Research*, 65(10-11), 1909-1920.
- Nie, J. G., Wang, Y. H. and Cai, C. S. (2012). Elastic rigidity of composite beams with full width slab openings. *Journal of Constructional Steel Research*, 73, 43-54.
- Nilson, A. (1960). Shear Diaphragms of Light Gage Steel. *Journal of the Structural Division*, 86(11), pp. 111–140.

- Newell J, Uang CM and Benzoni G. (2006). Subassemblage testing of core brace bucklingrestrained braces (G Series). University of California San Diego. Report no. TR2006/01; 2006
- O'Brien, P., Eatherton, M.R. and Easterling, W.S. (2017). Characterizing the load-deformation behavior of steel deck diaphragms using past test data, Cold-Formed Steel Research Consortium Report Series, Report Number CFSRC R-2017-02. <http://jhir.library.jhu.edu/handle/1774.2/40633>
- Özkılıç, Y. O., Bozkurt, M. B. and Topkaya, C. (2018). Evaluation of seismic response factors for BRBFs using FEMA P695 methodology. *Journal of Constructional Steel Research*, 151, 41-57.
- Pallarés, L. and Hajjar, J. F. (2010). Headed steel stud anchors in composite structures, Part I: Shear. *Journal of Constructional Steel Research*, 66(2), 198-212.
- Piotter, J. and Murray, T.M. (2000). "Cantilever Diaphragm Tests – Starshield Roof Systems," Report No. CE/VPI-ST-00/01, Charles Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.
- Porter, M., and Easterling, W. (1988). Behavior, analysis, and design of steel-deck-reinforced concrete diaphragms. *Report No. ISU-ERI-Ames-88305 Project 1636*, Iowa State University.
- Porter, M. and Greimann, L. (1980). Seismic resistance of composite floor diaphragms. *Report No. ISU-ERI-AMES-80133*, Iowa State University.
- Porter, M. and Greimann, L. (1982). Composite steel deck diaphragm slabs - design modes. *6th International Specialty Conference on Cold-Formed Steel Structures*, pp. 467–484.
- Prins, M. D. (1985). *Elemental tests for the seismic resistance of composite floor diaphragms* (Master thesis, Iowa State University).
- Queiroz, F. D., Vellasco, P. C. G. S. and Nethercot, D. A. (2007). Finite element modelling of composite beams with full and partial shear connection. *Journal of Constructional Steel Research*, 63(4), 505-521.
- Rambo-Roddenberry, M. D. (2002). *Behavior and Strength of Welded Stud Shear Connectors* (Doctoral dissertation, Virginia Tech).
- Rana, M. M., Uy, B. and Mirza, O. (2015). Experimental and numerical study of end anchorage in composite slabs. *Journal of Constructional Steel Research*, 115, 372-386.

- Rehman, N., Lam, D., Dai, X. and Ashour, A. F. (2016). Experimental study on demountable shear connectors in composite slabs with profiled decking. *Journal of Constructional Steel Research*, 122, 178-189.
- Robinson, H. (1988). Multiple stud shear connections in deep ribbed metal deck. *Canadian Journal of Civil Engineering*, 15(4), 553-569.
- Rodriguez, M. E., Restrepo, J. I. and Blandón, J. J. (2007). Seismic design forces for rigid floor diaphragms in precast concrete building structures. *Journal of structural engineering*, 133(11), 1604-1615.
- Sabelli, Rafael, Sabol, Thomas A., and Easterling, Samuel W. (2011). “Seismic design of composite steel deck and concrete-filled diaphragms: A guide for practicing engineers,” *NEHRP Seismic Design Technical Brief No. 5*, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 11-917-10.
- Sadek, F., El-Tawil, S. and Lew, H. S. (2008). Robustness of composite floor systems with shear connections: Modeling, simulation, and evaluation. *Journal of Structural Engineering*, 134(11), 1717-1725.
- Schafer, B. W., Smith, B. H., Torabian, S., Meimand, V. and Eatherton, M. R. (2018). Modeling and performance of thin-walled steel deck in roof diaphragms under seismic demands. In *8th International Conference on Thin-Walled Structures*. Lisbon, Portugal.
- Sebastian, W. M. and McConnel, R. E. (2000). Nonlinear FE analysis of steel-concrete composite structures. *Journal of Structural Engineering*, 126(6), 662-674.
- Seek, M. W., Avci, O. and McLaughlin, D. (2021). Effective standoff in standing seam roof systems. *Journal of Constructional Steel Research*, 180, 106590.
- Shen, M. H. and Chung, K. F. (2017). Experimental investigation into stud shear connections under combined shear and tension forces. *Journal of Constructional Steel Research*, 133, 434-447.
- Shen, M. H., Chung, K. F., Elghazouli, A. Y. and Tong, J. Z. (2020). Structural behaviour of stud shear connections in composite floors with various connector arrangements and profiled deck configurations. *Engineering Structures*, 210, 110370.

- Sockalingam, B., (2015). “Test Report on VP Buildings Inc.’s SSR Roof Panels,” Report No. C2016-1, ENCON® Technology, Inc. Tulsa, OK.
- Spangler, D. and Murray, T.M. (1989). “Integration of Standing Seam Roof Systems,” Report No. CE/VPI-ST89/07, Charles Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.
- Sublett, C. N. (1991). *Strength of welded headed studs in ribbed metal deck on composite joists* (Master thesis, Virginia Tech).
- Tahmasebinia, F., Ranzi, G. and Zona, A. (2013). Probabilistic three-dimensional finite element study on composite beams with steel trapezoidal decking. *Journal of Constructional Steel Research*, 80, 394-411.
- Torabian, S. (2018a) “Button Punch Sidelaps (Cyclic testing program)” NUCOR-Verco/Vulcraft Group, Project number 103-042-18, 11 July 2018 (released by Verco for public use)
- Torabian, S., Fratamico, D., Shannahan, K. and Schafer, B.W. (2018b). “Cyclic performance and behavior characterization of steel deck sidelap and framing Connections.” Proceedings of the Wei-Wen Yu International Specialty Conference: Cold- Formed Steel Structures – 2018, November 7-8, St. Louis, MO.
- Torabian, S., Eatherton, M.R., Easterling, W.S., Hajjar, J.F. and Schafer, B.W. (2019). SDII Building Archetype Design v2.0. CFSRC Report R-2019-04, jhir.library.jhu.edu/handle/1774.2/62106.
- Trout, A.M., (2000). *Further Study of the Gravity Loading Base Test Method* (Master thesis, Virginia Tech).
- Veismoradi, S., Amiri, G. G. and Darvishan, E. (2016). Probabilistic seismic assessment of buckling restrained braces and yielding brace systems. *International Journal of Steel Structures*, 16(3), 831-843.
- Yanez, S. J., Dinehart, D. W. and Santhanam, S. (2017). Composite steel joist analysis using experimental stiffness factor from push-out tests. *Journal of Constructional Steel Research*, 137, 1-7.
- Yu, M., Zha, X. and Ye, J. (2010). The influence of joints and composite floor slabs on effective tying of steel structures in preventing progressive collapse. *Journal of Constructional Steel Research*, 66(3), 442-451.

Zaruma, S. and Fahnestock, L. A. (2018). Assessment of design parameters influencing seismic collapse performance of buckling-restrained braced frames. *Soil Dynamics and Earthquake Engineering*, 113, 35-46.

APPENDIX A. ADDITIONAL INFORMATION FOR COMPUTATIONAL STUDY USING NONLINEAR 3D BUILDING MODELS

A.1. Member Sizes of Archetype Buildings

The sizes of beams, columns, and BRBs of the archetype buildings are given in Table A-1,

Table A-2, and Table A-3, respectively.

Table A-1 Beam Sizes of Archetype Buildings

Archetype building	Story	x direction (long direction)			y direction (short direction)		
		Edge beam		Interior beam	Edge beam		Interior beam
		At BRBF	Other		At BRBF	Other	
1-story (bare steel deck roof)	1	W12X16	W12X16	W12X19	W12X26	W12X26	W12X30
1-story (concrete on steel deck roof)	1	W14X26	W14X22	W16X26	W16X36	W16X26	W21X48
4-story	1	W24X62	W16X26	W16X31	W16X67	W16X31	W21X48
	2	W24X62	W16X26	W16X31	W16X57	W16X31	W21X48
	3	W24X62	W16X26	W16X31	W16X40	W16X31	W21X48
	4	W12X16	W12X16	W12X19	W12X22	W12X22	W14X30
8-story	1	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	2	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	3	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	4	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	5	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	6	W24x76	W16x26	W16x31	W16x89	W16x31	W21x48
	7	W24x76	W16x26	W16x31	W16x89	W16x31	W21x48
	8	W24x76	W16x26	W16x31	W16x77	W16x31	W21x48
12-story	1	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	2	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	3	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	4	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	5	W24x84	W16x26	W16x31	W16x100	W16x31	W21x48
	6	W24x76	W16x26	W16x31	W16x89	W16x31	W21x48
	7	W24x76	W16x26	W16x31	W16x89	W16x31	W21x48
	8	W24x76	W16x26	W16x31	W16x77	W16x31	W21x48
	9	W24x62	W16x26	W16x31	W16x67	W16x31	W21x48
	10	W24x62	W16x26	W16x31	W16x57	W16x31	W21x48
	11	W24x62	W16x26	W16x31	W16x40	W16x31	W21x48
	12	W14x30	W14x30	W12x19	W14x26	W14x26	W14x30

Table A-2 Column Sizes of Archetype Buildings

Archetype building	Story	Edge column					Interior column
		At BRBF (x direction)	At BRBF (y direction)		Corner	Other	
			Center	Outer			
1-story (bare steel deck roof)	1	W12X40	W12X40	W12X40	W10X30	W10X30	W10X30
1-story (concrete on steel deck roof)	1	W14X48	W14X48	W14X48	W10X30	W10X30	W10X30
4-story	1-2	W14X109	W14X48	W14X82	W10X33	W10X39	W10X49
	3-4	W14X48	W14X48	W14X48	W10X30	W10X33	W10X30
8-story	1-2	W14x342	W14x82	W14x500	W10x60	W12x87	W12x120
	3-4	W14x283	W14x68	W14x342	W10x49	W10x77	W12x87
	5-6	W14x193	W14x61	W14x257	W10x39	W10x60	W10x77
	7-8	W14x132	W14x48	W14x145	W10x33	W10x49	W10x54
12-story	1-2	W14x342	W14x82	W14x500	W10x60	W12x87	W12x120
	3-4	W14x283	W14x68	W14x342	W10x49	W10x77	W12x87
	5-6	W14x193	W14x61	W14x257	W10x39	W10x60	W10x77
	7-8	W14x132	W14x48	W14x145	W10x33	W10x49	W10x54
	9-10	W14x82	W14x48	W14x82	W10x30	W10x39	W10x45
	11-12	W14x48	W14x48	W14x48	W10x30	W10x33	W10x30

Table A-3 BRB Core Areas (A_{core}), Yield-to-Length Ratios (YLR) and Approximate Stiffness Modification Factors (KF) of Archetype Buildings

Archetype building	Story	x direction (long direction)			y direction (short direction)		
		A_{core} (in ²)	YLR	KF	A_{core} (in ²)	YLR	KF
1-story (bare steel deck roof)	1	1.5	0.85	1.37	1.5	0.77	1.45
1-story (concrete on steel deck roof)	1	3	0.84	1.41	4	0.74	1.54
4-story	1	8	0.86	1.37	10	0.80	1.50
	2	7	0.86	1.37	8	0.79	1.46
	3	5	0.85	1.37	6	0.79	1.46
	4	2	0.85	1.37	3	0.79	1.45
8-story	1	10	0.86	1.37	12	0.76	1.51
	2	10	0.86	1.37	12	0.75	1.53
	3	9	0.86	1.37	12	0.75	1.53
	4	9	0.86	1.37	11	0.75	1.53
	5	7	0.86	1.37	9	0.79	1.46
	6	6	0.86	1.37	7	0.79	1.46
	7	4	0.85	1.37	5	0.77	1.45
	8	2	0.85	1.37	2	0.77	1.45
12-story	1	14	0.71	1.41	16	0.65	1.54
	2	13	0.71	1.41	16	0.65	1.55
	3	13	0.71	1.41	16	0.65	1.55
	4	12	0.71	1.41	15	0.65	1.53
	5	12	0.71	1.41	15	0.65	1.53
	6	11	0.71	1.41	14	0.65	1.53
	7	11	0.71	1.41	13	0.65	1.53
	8	11	0.71	1.41	12	0.65	1.53
	9	9	0.73	1.37	10	0.68	1.46
	10	8	0.73	1.37	8	0.68	1.46
	11	6	0.73	1.37	5	0.69	1.45
	12	2	0.73	1.37	2	0.69	1.45

A.2. Modification of Pinching4 Backbone Parameters for Diaphragm Models

The backbone parameters (stresses and strains) of the Pinching4 material model were modified as follows so that the diaphragm shear strength per unit length is consistently represented. The equations used for the modification are derived based on Figure A-1.

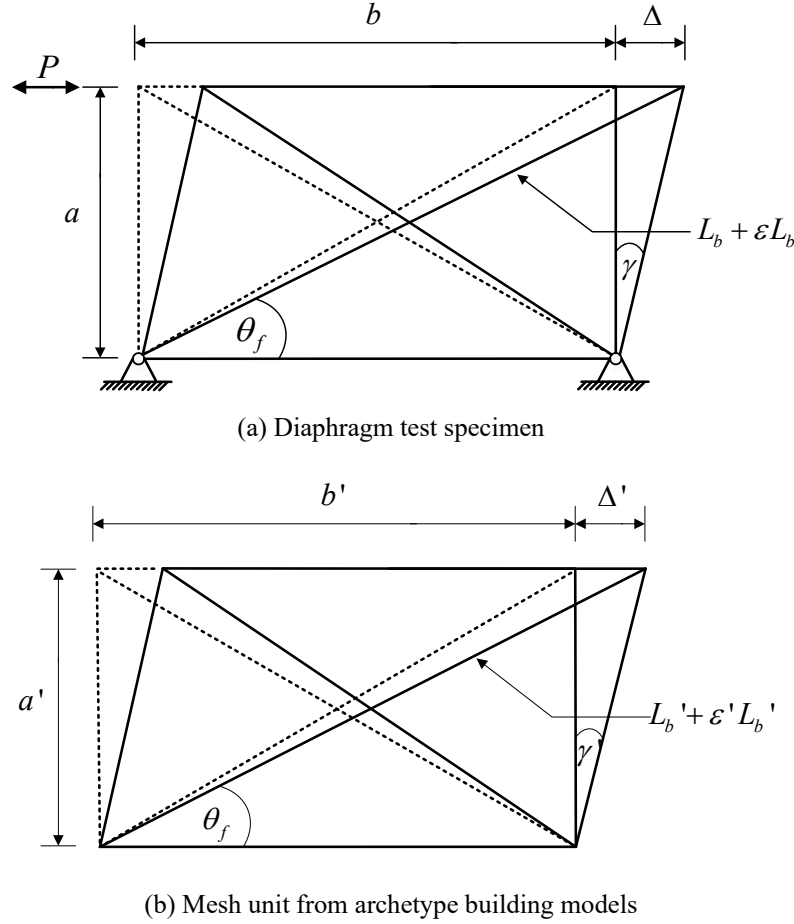


Figure A-1 Comparison of the diaphragm test specimen and archetype diaphragm mesh unit

1) Stresses

The force in the diagonal trusses, F_b , is given by:

$$F_{bi} = \sigma_i A \quad (\text{A-1})$$

where σ is the stress in the diagonal trusses, A is the area of the diagonal trusses and i is the number ranging from 1 to 4 (corresponding to the Pinching4 stress values).

The relationship between the force, P , and F_b can be established using:

$$\cos \theta = \frac{P_i}{2F_{bi}} \quad (\text{A-2})$$

where θ is the angle in undeformed position (initial angle) which can be obtained using:

$$\theta = \tan^{-1}(a/b) \quad (\text{A-3})$$

where b is the span of the diaphragm specimen and a is the depth of the diaphragm specimen. Substituting Equation A-2 into Equation A-1 yields:

$$P_i = 2\sigma_i A \cos \theta \quad (\text{A-4})$$

The shear strength per unit length of the specimen, S , can be found by dividing Equation A-4 by the span of the diaphragm:

$$S_i = \frac{2}{b} \sigma_i A \cos \theta \quad (\text{A-5})$$

Then the modified stresses for the archetype building models, σ'_i , can be obtained using:

$$S_i = \frac{2}{b'} \sigma'_i A' \cos \theta' \quad (\text{A-6})$$

$$\sigma'_i = \frac{b' S_i}{2 A' \cos \theta'} \quad (\text{A-7})$$

where b' and A' are the span of each mesh unit and the area of the diagonal trusses in the archetype building models, respectively, and θ' is the initial angle that can be obtained by:

$$\theta' = \tan^{-1}(a'/b') \quad (\text{A-8})$$

where a' is the depth of each mesh unit in the archetype building models.

The modified backbone stresses were then scaled by the factors provided in Table 5-7 and used in the Pinching4 material model of the archetype building models.

2) Strains

The relationship between the diaphragm deflection, Δ , and the strain in the truss member, ε , can be established (based on the deformed geometry) using:

$$\cos \theta_f = \frac{b + \Delta}{L_b + \varepsilon L_b} \quad (\text{A-9})$$

where θ_f is the angle in deformed position (final angle) and L_b is the undeformed length of the truss member which can be obtained using:

$$L_b = \sqrt{b^2 + a^2} \quad (\text{A-10})$$

The diaphragm deflection, Δ , is given by:

$$\Delta = \gamma a \quad (\text{A-11})$$

where γ is the shear angle. Substituting Equation A-11 into Equation A9 yields:

$$\gamma_i = \frac{1}{a} \left[(L_b + \varepsilon_i L_b) \cos \theta_f - b \right] \quad (\text{A-12})$$

Then the modified strains for the archetype building model, ε'_i , can be obtained using:

$$\gamma_i = \frac{1}{a'} \left[(L'_b + \varepsilon'_i L'_b) \cos \theta'_f - b' \right] \quad (\text{A-13})$$

$$\varepsilon'_i = \frac{1}{L' \cos \theta'_f} (\gamma_i a' + b') - 1 \quad (\text{A-14})$$

where L'_b is the undeformed length of the truss member in each mesh unit in the archetype building model and all other terms were defined previously.

Table A-4 provides the values of the Pinching4 material model parameters for the archetype building diaphragm models.

Table A-4 Pinching4 Material Model Parameters Used for Archetype Building Models

	Backbone				Pinching			Strength Degradation					Stiffness Degradation					Energy Dissip.
Diaphragm	ε_1, σ_1 (ksi)	ε_2, σ_2 (ksi)	ε_3, σ_3 (ksi)	ε_4, σ_4 (ksi)	$r_{\Delta^+},$ r_{Δ^-}	$r_{F^+},$ r_{F^-}	$u_{F^+},$ u_{F^-}	gF_1	gF_2	gF_3	gF_4	gF_{lim}	$gK_1,$ gD_1	$gK_2,$ gD_2	$gK_3,$ gD_3	$gK_4,$ gD_4	$gK_{lim},$ gD_{lim}	gE
Bare Deck 1	0.0010, 17.95	0.0022, 23.39	0.0042, 24.84	0.0107, 9.93	0.20, 0.35	0.20, 0.35	0.10, 0.12	0	0.35	0	0.70	0.90	0, 0	0, 0.50	0, 0	0, 0.75	0, 0.90	4.31
Composite 1	0.0005, 82.18	0.0006, 98.94	0.0014, 139.07	0.0134, 62.58	-0.06, -0.06	0.12, 0.12	0.11, 0.11	0	0.83	0.0	0.46	0.33	1.09, 0.14	0.76, 0.47	0.32, 0.12	0.75, 0.10	1.04, 0.61	4.29
Bare Deck 2	0.0010, 28.77	0.0022, 37.48	0.0042, 39.80	0.0107, 15.92	0.20, 0.35	0.20, 0.35	0.10, 0.12	0	0.35	0	0.70	0.90	0, 0	0, 0.50	0, 0	0, 0.75	0, 0.90	4.31
Composite 2	0.0005, 121.00	0.0006, 145.68	0.0014, 204.76	0.0134, 92.14	-0.06, -0.06	0.12, 0.12	0.11, 0.11	0	0.83	0.0	0.46	0.33	1.09, 0.14	0.76, 0.47	0.32, 0.12	0.75, 0.10	1.04, 0.61	4.29

A.3. Additional Information about Nonlinear Response History Analysis Results

Table A-5 and Table A-6 provide details for the medians of peak story drifts and diaphragm shear demands of the archetype buildings from the nonlinear response history results. Figure A-2 shows the lognormal cumulative distribution plots for all the archetype buildings.

Table A-5 Medians of Peak Story Drifts at Each Story of Archetype Buildings under Three Ground Motion Levels

Archetype Building	Diaphragm Design	Story	Median of Peak Story Drift at Each Story (%)								
			DE			MCE			ACMR _{10%}		
			x	y	Result.	x	y	Result.	x	y	Result.
1-story (bare steel deck roof)	Alt. 1	1	2.2	2.6	3.2	3.3	3.9	5.3	4.3	4.9	6.6
	Trad./Alt. 2	1	2.1	3.0	3.5	3.4	4.3	5.5	4.5	5.8	7.3
1-story (concrete on steel deck roof)	Alt. 1	1	2.5	2.4	3.1	3.7	3.8	5.1	4.7	4.7	6.6
	Trad./Alt. 2	1	2.5	2.4	3.0	3.9	3.6	5.3	4.6	4.5	6.8
4-story	Alt. 1	1	2.6	2.4	3.2	5.3	3.8	5.7	6.0	4.6	6.9
		2	1.7	1.7	2.1	2.8	2.8	3.9	3.8	3.6	4.6
		3	1.5	1.6	1.8	2.0	2.3	2.7	2.2	2.8	3.1
		4	2.1	2.8	3.0	2.4	3.3	3.5	2.7	3.6	3.8
		whole building	2.7	2.9	3.4	5.3	3.9	5.7	6.0	4.7	6.9
	Trad./Alt. 2	1	2.6	2.3	3.1	5.1	3.9	5.8	6.0	4.8	7.0
		2	1.6	1.6	2.1	2.7	3.0	3.9	3.7	3.7	4.7
		3	1.5	1.7	1.9	2.1	2.5	2.9	2.4	3.0	3.4
		4	2.1	3.3	3.5	2.5	4.2	4.3	2.8	4.5	4.9
		whole building	2.7	3.4	3.7	5.1	4.4	5.8	6.0	5.3	7.0
8-story	Alt. 1	1	2.5	2.4	3.0	3.7	4.0	5.2	5.1	5.0	6.5
		2	1.6	1.8	2.1	2.5	3.0	4.0	3.5	3.9	4.9
		3	1.2	1.1	1.4	2.0	2.0	2.7	2.3	2.5	3.1
		4-6	1.0-1.2	0.9-1.3	1.2-1.5	1.5-1.6	1.3-1.7	1.8-2.0	1.6-1.7	1.6-1.8	2.0-2.1
		7	1.4	1.6	1.8	1.6	1.9	2.1	1.7	2.1	2.3
		8	1.9	2.6	2.8	2.0	2.9	3.1	2.0	3.0	3.2
		whole building	2.5	2.9	3.3	3.7	4.0	5.2	5.1	5.0	6.5
	Trad./Alt. 2	1	2.4	2.6	3.1	3.7	4.2	5.4	4.6	5.3	6.7
		2	1.5	1.8	2.2	2.6	3.0	3.9	3.3	4.1	4.9
		3	1.2	1.1	1.4	2.0	2.2	2.7	2.3	2.7	3.2
		4-6	0.9-1.2	1.0-1.3	1.2-1.5	1.5-1.6	1.4-1.6	1.8-1.8	1.6-1.7	1.7-1.9	2.0-2.2
		7	1.5	1.6	1.9	1.7	2.0	2.3	1.8	2.0	2.4
		8	1.8	2.8	2.9	2.0	3.3	3.4	2.1	3.4	3.4
		whole building	2.5	3.4	3.6	3.7	4.3	5.4	4.6	5.5	6.7
12-story	Alt. 1	1	2.0	2.4	3.0	4.1	4.5	7.0	6.2	6.7	10.2
		2	1.5	2.0	2.4	3.2	3.9	5.5	5.1	5.9	8.6
		3	1.2	1.5	1.9	2.4	3.0	3.8	4.0	3.9	6.2
		4	1.1	1.3	1.5	1.9	2.1	2.7	2.9	2.8	4.0
		5-10	1.2-1.4	1.0-1.5	1.3-1.7	1.4-1.8	1.4-1.8	1.6-2.2	1.6-2.1	1.4-2.2	1.9-2.6
		11	1.4	1.9	2.1	1.6	2.3	2.5	1.7	2.2	2.7
		12	2.2	2.8	3.0	2.3	3.2	3.3	2.3	3.1	3.3
		whole building	2.4	3.3	3.6	4.1	4.5	7.0	6.2	6.7	10.2
	Trad./Alt. 2	1	1.9	2.5	3.0	4.1	4.1	6.0	6.6	5.9	10.6
		2	1.5	2.1	2.4	3.1	3.8	4.9	5.6	4.9	8.8
		3	1.2	1.5	1.8	2.4	2.8	3.6	3.8	3.9	6.1
		4	1.1	1.3	1.5	1.9	2.1	2.7	3.1	2.9	3.6
		5-10	1.1-1.4	1.0-1.4	1.3-1.7	1.4-1.7	1.3-1.7	1.6-2.0	1.5-2.3	1.6-2.2	1.9-2.7
		11	1.4	2.1	2.3	1.6	2.4	2.5	1.7	2.4	2.7
		12	2.3	3.1	3.4	2.4	3.5	3.6	2.4	3.6	3.8
		whole building	2.6	3.5	3.9	4.1	4.7	6.0	6.7	6.0	10.6

Table A-6 Medians of Diaphragm Shear Demands for Archetype Buildings and Comparison to Design Shear

Archetype Building	Diaphragm Design	Story	Median of F_{peak} (kip)									F_{px} (kip)	$\frac{F_{peak}}{F_{px}}$
			DE			MCE			ACMR _{10%}				
			x	y	x or y	x	y	x or y	x	y	x or y		
1-story (bare steel deck roof)	Alt. 1	1	215	291	292	259	357	357	288	401	404.2	419	0.70
	Trad. or Alt. 2	1	240	293	298	299	342	351	336	380	395.2	262	1.14
1-story (concrete on steel deck roof)	Alt. 1	1	367	608	608	442	703	703	470	791	793.2	855	0.71
	Trad. or Alt. 2	1	366	607	607	436	706	706	468	775	783.5	542	1.12
4-story	Alt. 1	1	746	655	816	961	830	994	1031	927	1113	977	0.83
		2	662	773	796	828	929	952	899	989	1025	914	0.87
		3	613	629	727	708	701	818	738	732	840.8	850	0.86
		4	352	441	442	386	480	480	391	504	506.3	419	1.06
	Trad. or Alt. 2	1	757	638	796	958	777	1008	1032	849	1095	524	1.52
		2	626	738	766	782	899	936	878	957	1018	524	1.46
		3	606	645	743	700	736	822	729	765	849.5	524	1.42
		4	384	384	406	431	405	432	443	411	442.7	262	1.55
8-story	Alt. 1	1	740	583	769	1046	810	1092	1175	919	1233	1012	0.76
		2	687	691	820	917	909	1093	1084	1038	1211	980	0.84
		3	656	713	743	844	858	965	937	935	1062	948	0.78
		4	686	563	726	862	659	867	904	676	916.9	916	0.79
		5	616	527	681	725	576	755	778	614	793.8	883	0.77
		6	621	519	624	671	545	702	688	576	704.3	851	0.73
		7	570	512	600	624	547	638	625	550	645.9	839	0.72
		8	331	350	371	346	379	381	351	380	386	419	0.89
	Trad. or Alt. 2	1	738	577	760	1056	749	1084	1220	855	1289	524	1.45
		2	684	706	787	877	869	1048	1062	1007	1198	524	1.50
		3	641	674	741	867	830	956	941	936	1058	524	1.41
		4	681	535	691	811	630	848	883	653	919.6	524	1.32
		5	602	488	623	728	542	734	766	580	781.4	524	1.19
		6	613	493	617	669	532	678	684	546	713.2	524	1.18
		7	548	517	583	620	554	650	630	565	668.7	524	1.11
		8	362	328	373	386	355	396	377	357	396.2	262	1.42
12-story	Alt. 1	1	934	691	965	1259	934	1343	1509	1108	1620	1024	0.94
		2	862	828	984	1124	1194	1329	1431	1326	1688	1003	0.98
		3	827	846	943	1069	1168	1298	1243	1372	1619	981	0.96
		4	790	771	879	963	1017	1100	1064	1076	1256	959	0.92
		5	767	680	810	925	850	954	997	914	1058	938	0.86
		6	729	640	760	883	727	920	936	784	999	916	0.83
		7	691	593	743	826	691	882	870	745	921	895	0.83
		8	750	610	764	848	640	871	882	682	894	873	0.88
		9	654	573	690	734	628	777	764	641	811	851	0.81
		10	644	556	656	758	616	758	756	623	767	839	0.78
		11	652	568	665	729	603	729	729	615	729	839	0.79
		12	412	391	431	436	419	449	435	434	451	419	1.03
	Trad. or Alt. 2	1	943	636	944	1253	839	1256	1570	1014	1669	524	1.80
		2	876	765	937	1113	1049	1243	1422	1275	1607	524	1.79
		3	831	826	949	1046	1091	1228	1268	1391	1524	524	1.81
		4	800	756	868	955	965	1097	1095	1090	1265	524	1.66
		5	774	645	797	908	787	939	1018	887	1086	524	1.52
		6	737	556	739	885	662	897	934	752	963	524	1.41
		7	719	542	751	789	630	837	857	680	915	524	1.43
		8	753	539	753	837	621	837	868	644	873	524	1.44
		9	651	512	670	710	580	753	762	606	771	524	1.28
		10	627	529	635	731	563	735	761	572	763	524	1.21
		11	667	567	685	706	606	710	733	621	742	524	1.31
		12	461	362	461	469	387	469	469	389	469	262	1.76

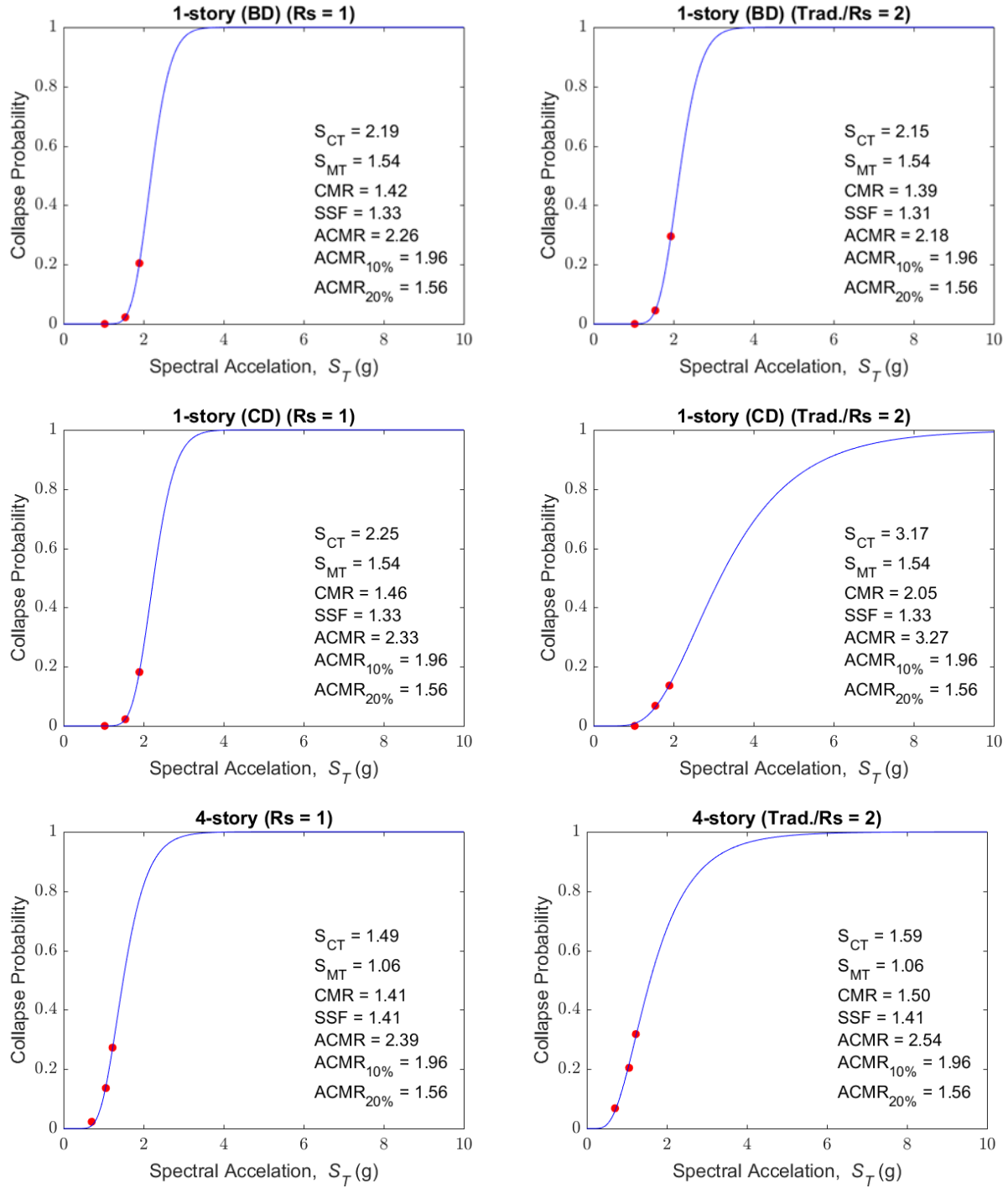


Figure A-2 Lognormal cumulative distribution plots for the archetype buildings

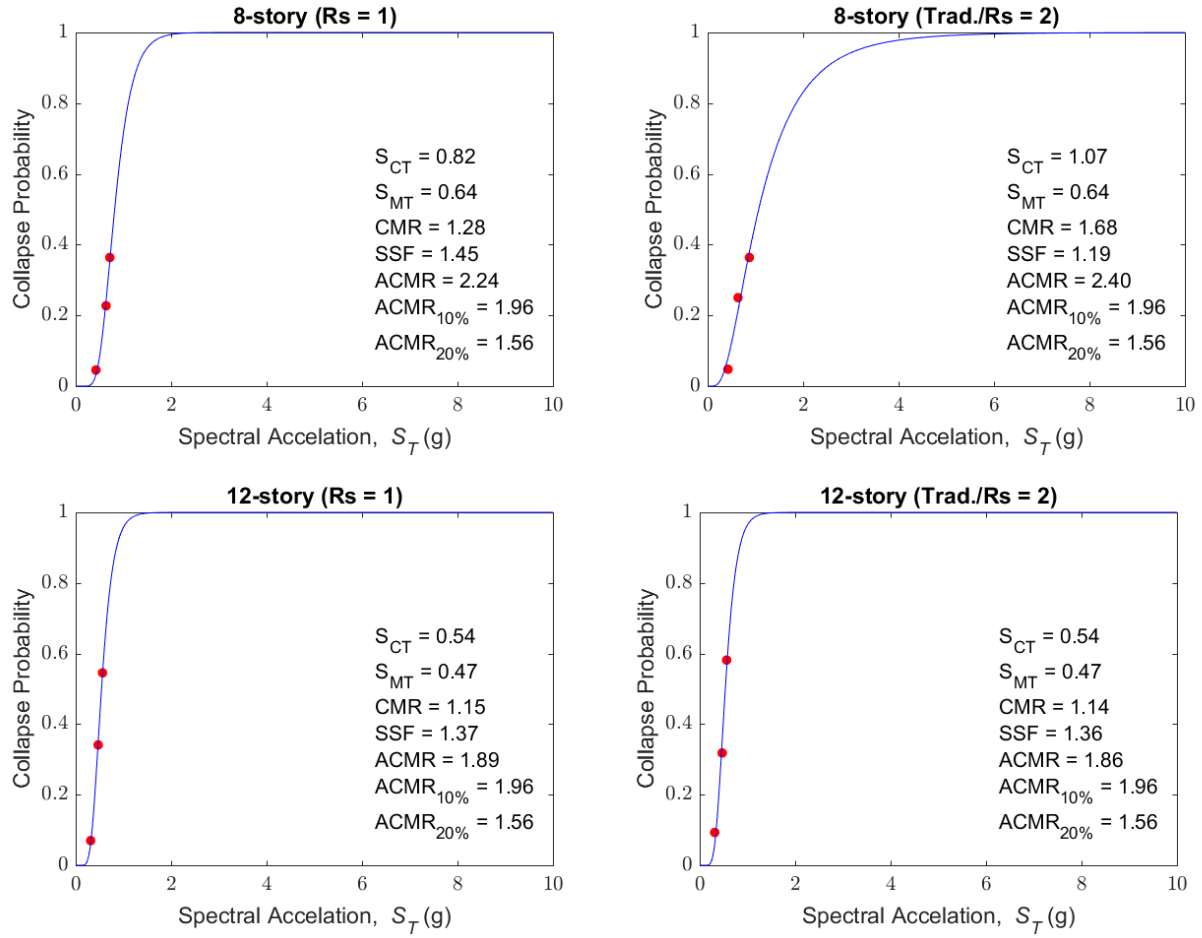


Figure A-2 (Continued) Lognormal cumulative distribution plots for the archetype buildings

APPENDIX B. ADDITIONAL INFORMATION FOR EXPERIMENTAL STUDY ON STANDING SEAM ROOF

This section provides more information of the test results for each specimen, including plots of load-displacement curves, purlin displacements, clip and panel deformations on middle purlins, and pictures of deformed shape and damage of the specimens.

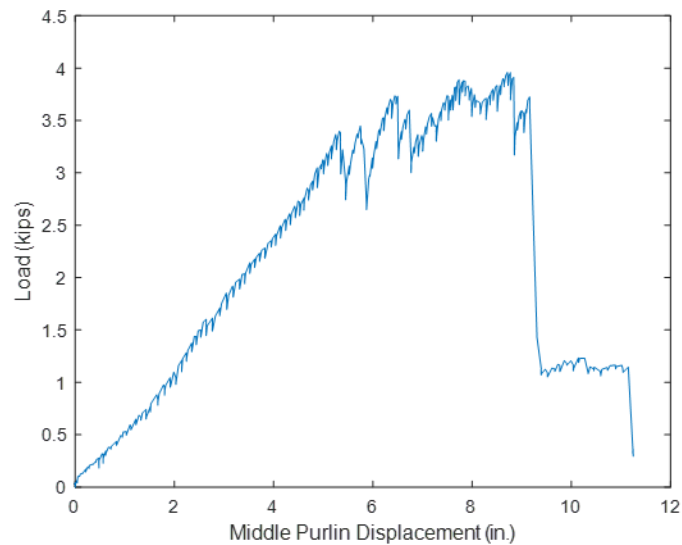


Figure B-1 Load-displacement curve for Specimen 1

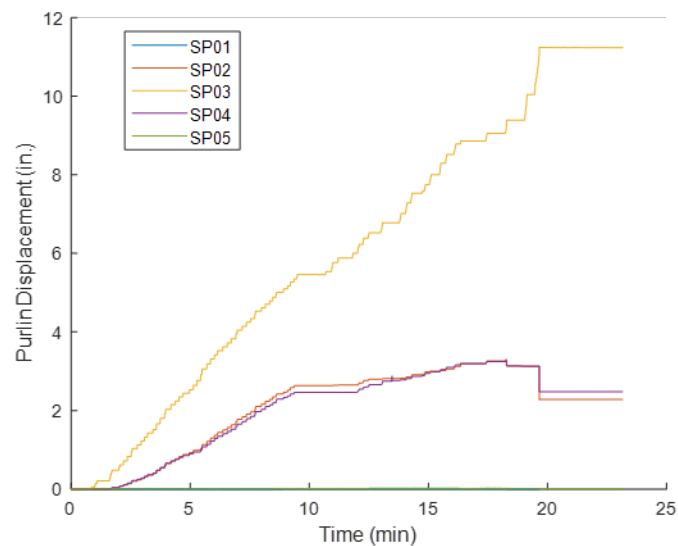


Figure B-2 Purlin displacements of Specimen 1



Figure B-3 Deformed shape of Specimen 1

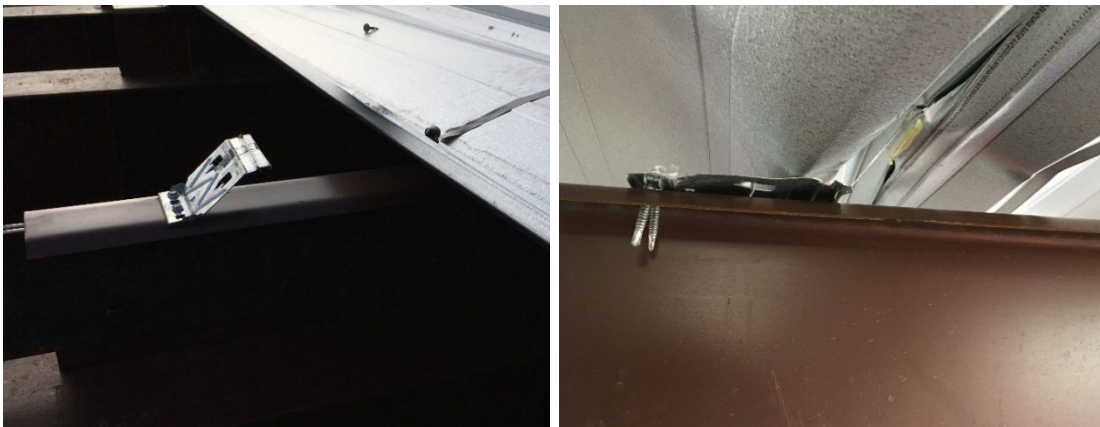


Figure B-4 Clip damage of Specimen 1

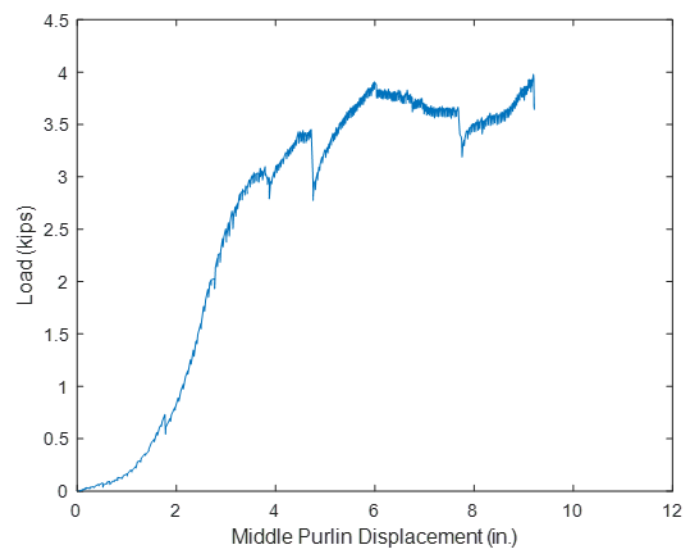


Figure B-5 Load-displacement curve for Specimen 2

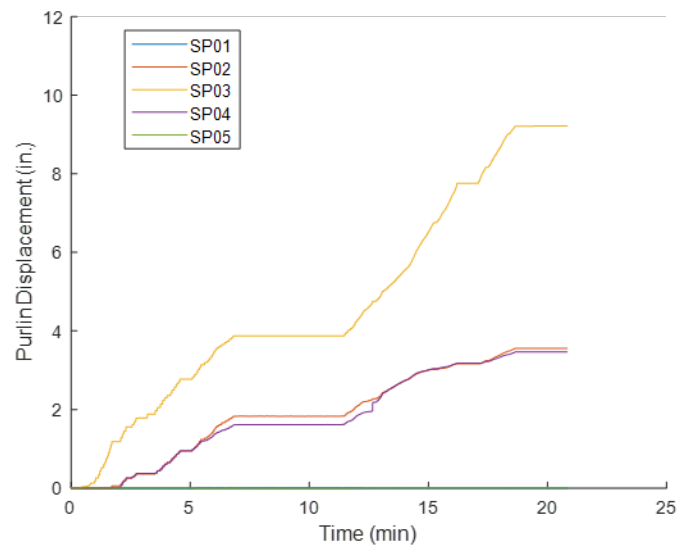


Figure B-6 Purlin displacements of Specimen 2



Figure B-7 Deformed shape of Specimen 2



Figure B-8 Clip damage of Specimen 2

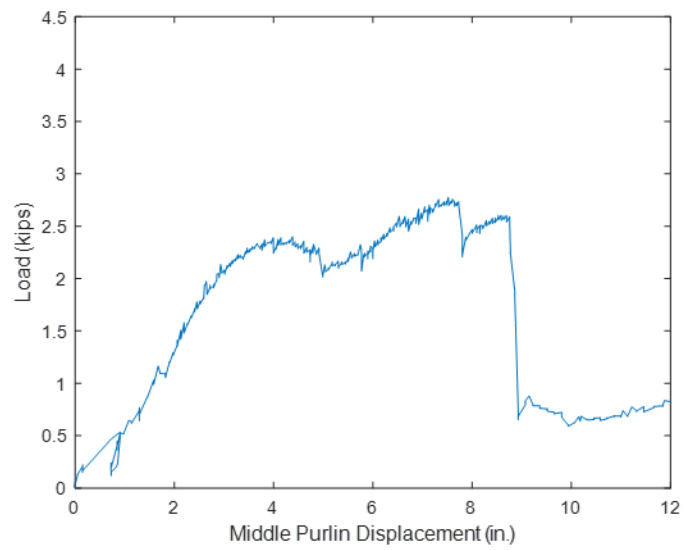


Figure B-9 Load-displacement curve for Specimen 3

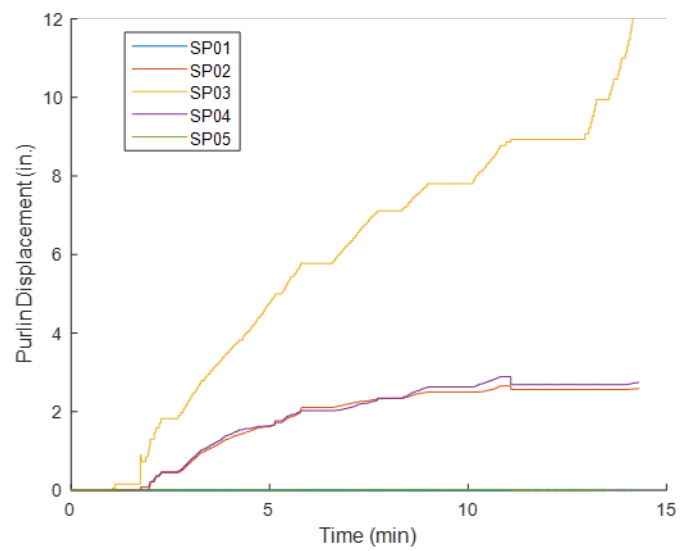


Figure B-10 Purlin displacements of Specimen 3



Figure B-11 Deformed shape of Specimen 3



Figure B-12 Clip damage of Specimen 3

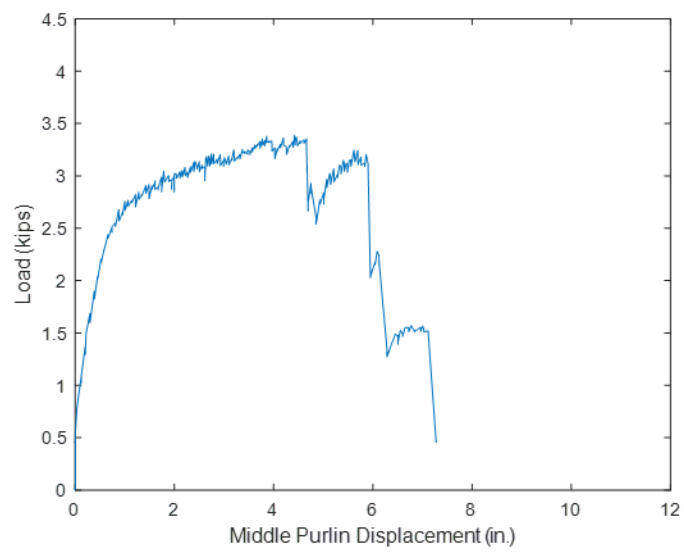


Figure B-13 Load-displacement curve for Specimen 4

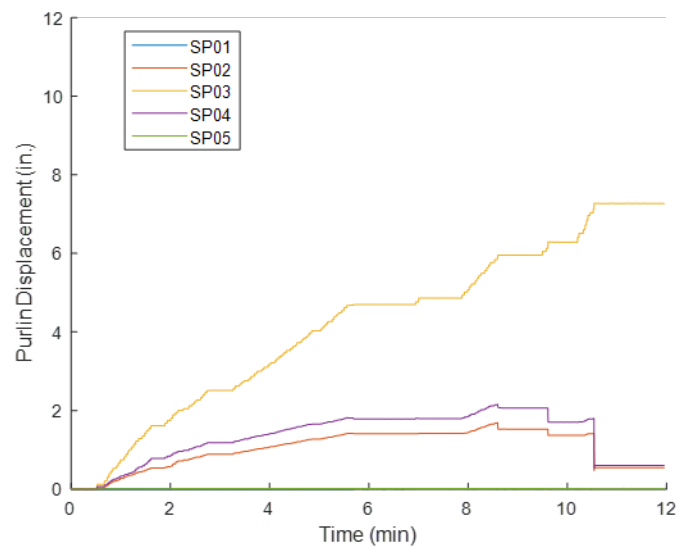


Figure B-14 Purlin displacements of Specimen 4



Figure B-15 Deformed shape of Specimen 4



Figure B-16 Clip damage of Specimen 4

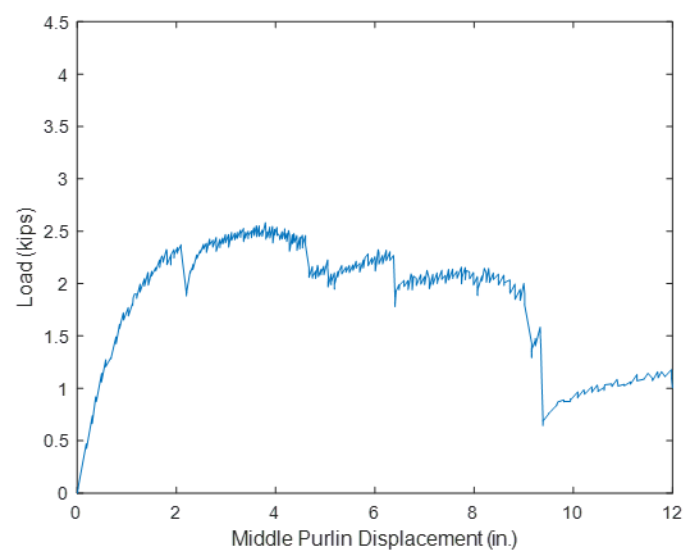


Figure B-17 Load-displacement curve for Specimen 5

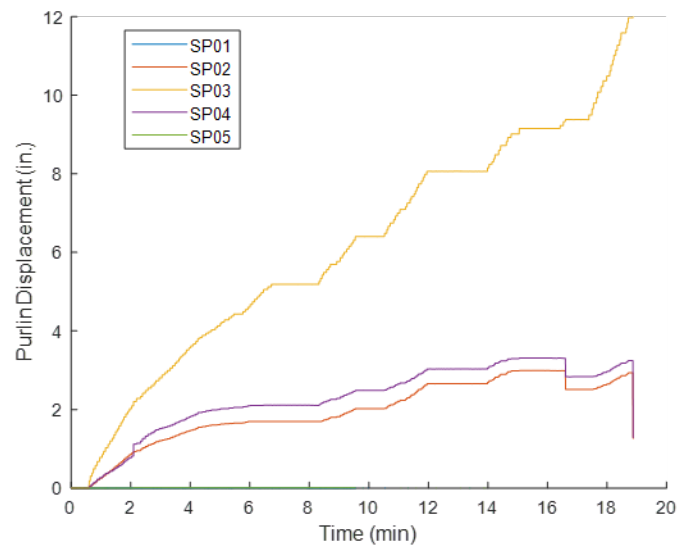


Figure B-18 Purlin displacements of Specimen 5

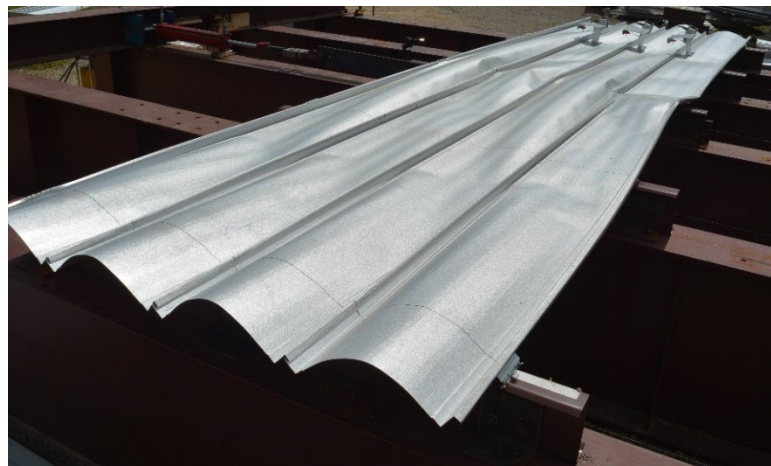


Figure B-19 Deformed shape of Specimen 5



Figure B-20 Clip damage of Specimen 5

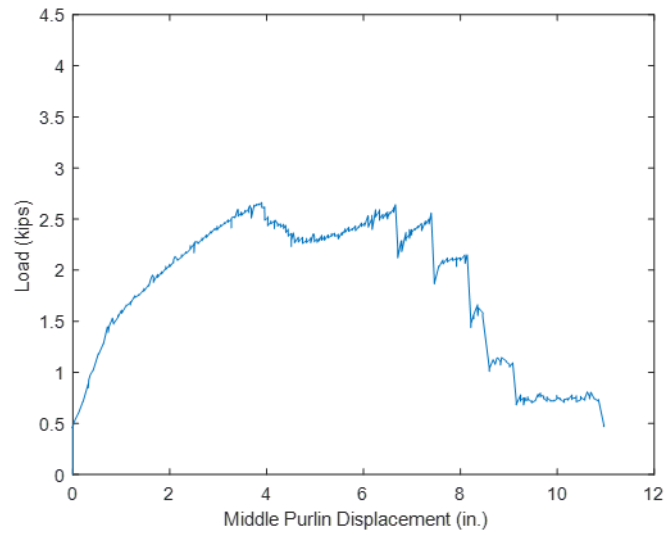


Figure B-21 Load-displacement curve for Specimen 6

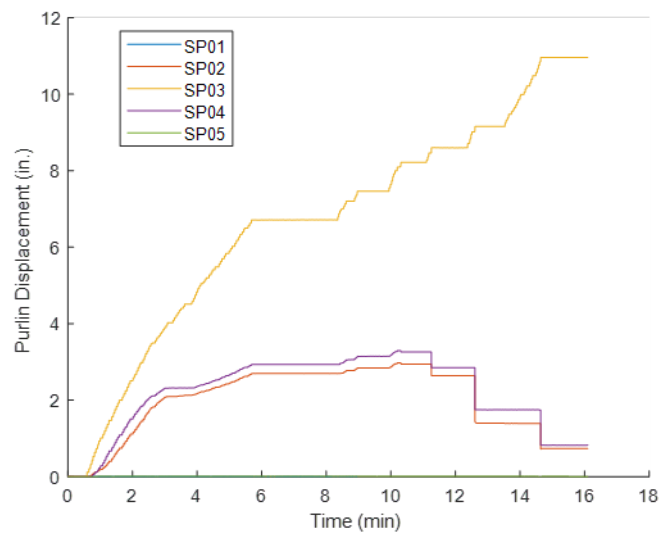


Figure B-22 Purlin displacements of Specimen 6



Figure B-23 Deformed shape of Specimen 6



Figure B-24 Clip damage of Specimen 6

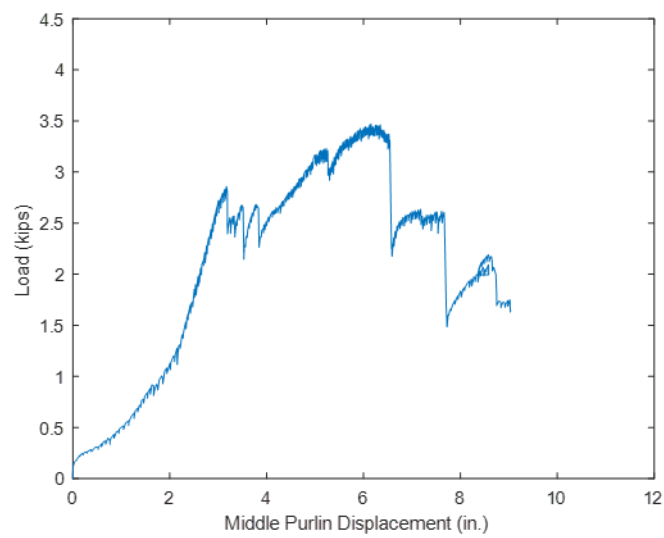


Figure B-25 Load-displacement curve for Specimen 7

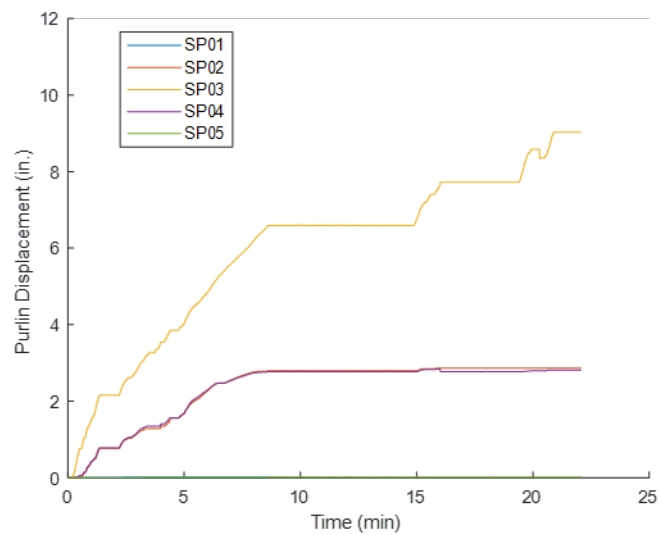


Figure B-26 Purlin displacements of Specimen 7



Figure B-27 Deformed shape of Specimen 7



Figure B-28 Clip damage of Specimen 7

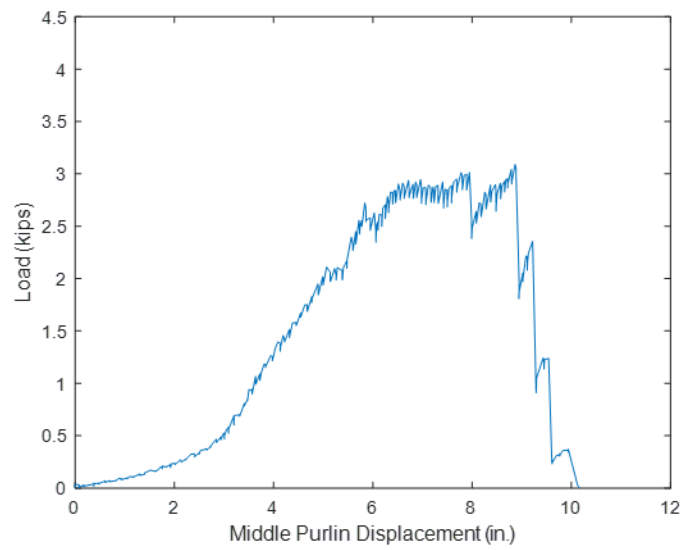


Figure B-29 Load-displacement curve for Specimen 8

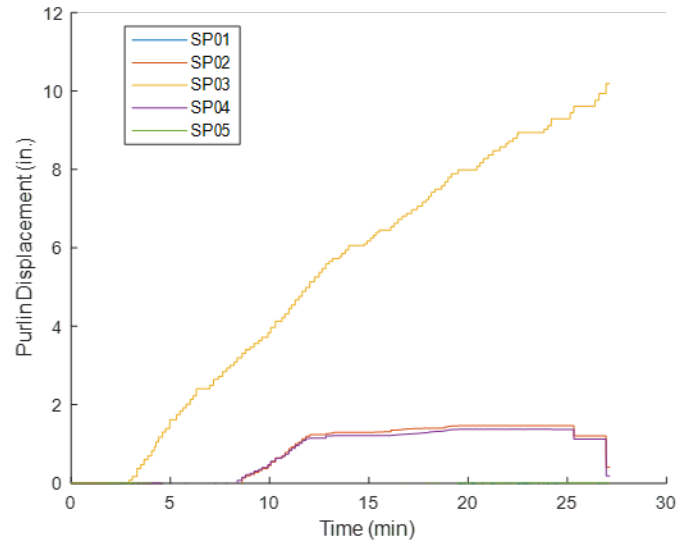


Figure B-30 Purlin displacements of Specimen 8



Figure B-31 Deformed shape of Specimen 8



Figure B-32 Clip damage of Specimen 8

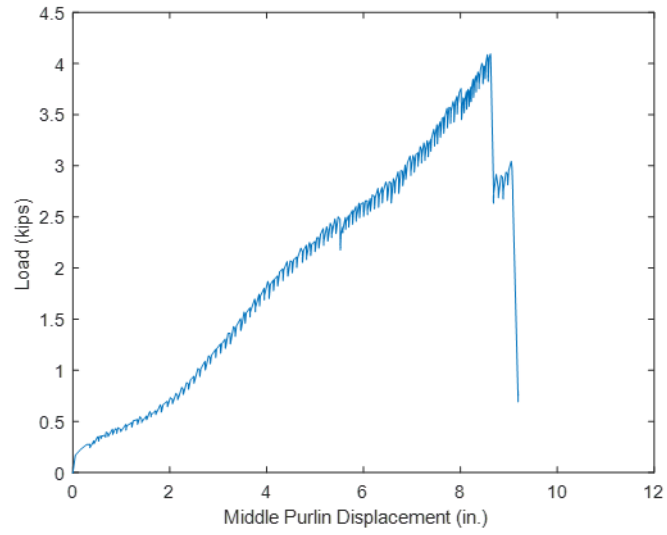


Figure B-33 Load-displacement curve for Specimen 9

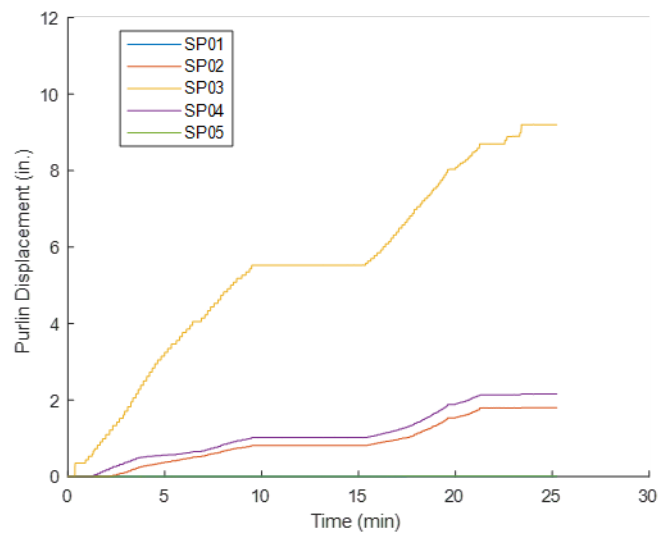


Figure B-34 Purlin displacements of Specimen 9



Figure B-35 Deformed shape of Specimen 9



Figure B-36 Clip damage of Specimen 9

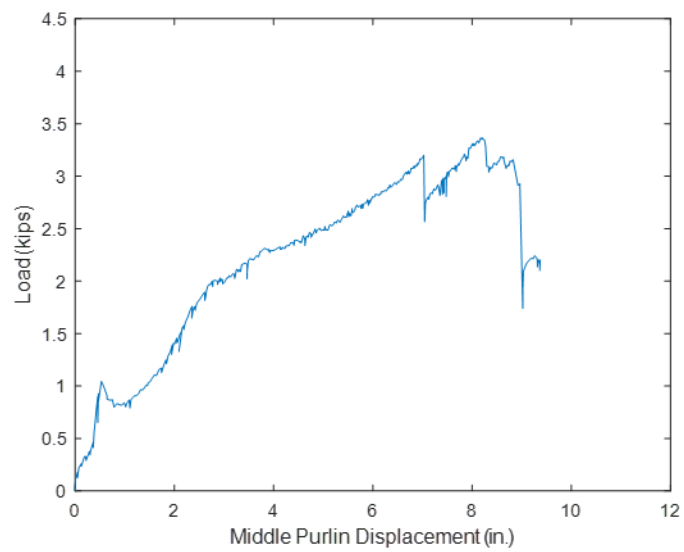


Figure B-37 Load-displacement curve for Specimen 10

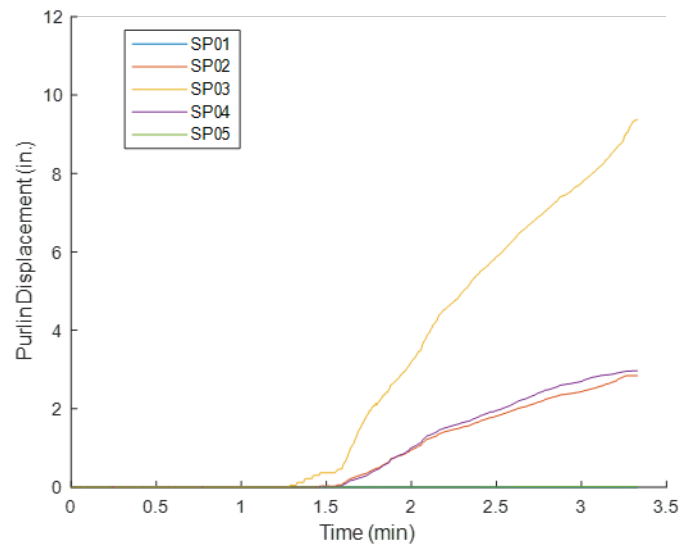


Figure B-38 Purlin displacements of Specimen 10



Figure B-39 Deformed shape of Specimen 10



Figure B-40 Clip damage of Specimen 10

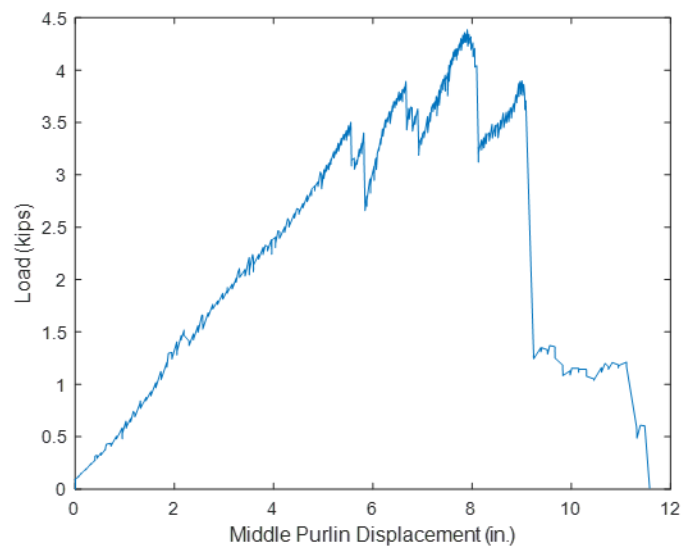


Figure B-41 Load-displacement curve for Specimen 11

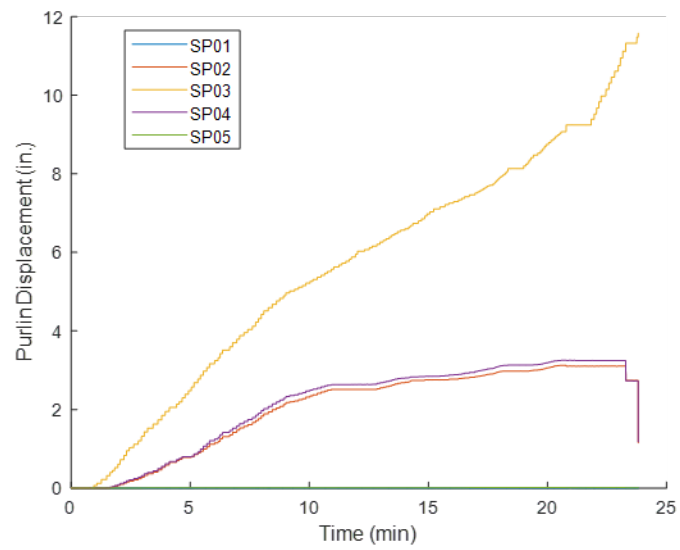


Figure B-42 Purlin displacements of Specimen 11



Figure B-43 Deformed shape of Specimen 11



Figure B-44 Clip damage of Specimen 11