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SEISMIC DRIFT DEMANDS OF REINFORCED CONCRETE BUILDINGS

by

© Ahmad Shooshtari

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Canada

To my people

especially those who are suffering

from earthquakes

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Abstract

During the last two decades a significant amount of analytical and experimental research has been conducted on seismic behaviour of reinforced concrete structures. The outcome of this research resulted in design recommendations, including those for computation of lateral drift demands. Most current building codes recommend simple procedures for estimating inelastic displacement demands by specifying an amplification factor to be applied to elastic drifts, obtained from design spectra. This simple approach neglects to incorporate the effects of a number of structural and ground motion parameters.

A comprehensive analytical research has been conducted in the current study, which involved more than 350 analyses of R/C buildings in Canada, to gain insight into inelastic seismic drift demands of these buildings. The investigation consists of three major phases. The first phase includes the development of an improved version of an existing general purpose dynamic analysis software. The improvements consist of four major modifications; a) implementation of a hysteretic model for P-M interaction effects, b) implementation of a hysteretic model for masonry infill panels, c) introduction of a new method to consider the P- Δ effect, and d) introduction of a new feature to conduct inelastic static "Push-Over" analysis.

The second phase involves a parametric study to establish importance of structural and ground motion parameters on seismic drift demands of R/C buildings. A 10-storey frame building, with and without reinforced concrete shear walls, was considered for the parametric study. An inventory of earthquake motions, consisting of 17 records,

was selected for this purpose. The results indicated that inelastic shear, anchorage slip, P- Δ , and presence of masonry infills all played important roles on drift response. The P-M interaction on flexural behavior during response did not affect drift response for the structures considered. Therefore, this feature was not considered in the subsequent phase of the investigation, where inelastic drift demands were established.

A total of 12 different types of buildings with different structural systems and heights were designed for response history analysis to establish seismic drift demands of R/C buildings. This constituted the third phase of the study. The structures considered included frame buildings with or without reinforced concrete shear walls and/or masonry walls. Three types of ground motion were used to generate the drift data. These included; i) the artificial records generated for the geographic locations selected, conforming the uniform hazard response spectra recently proposed by the Geological Survey Canada with 10% probability of occurrence in 50 years, ii) previously recorded critical earthquake motions, iii) artificial records scaled to give the same design base shears as those obtained using NBCC-95. The results indicate that building located in eastern Canada remained essentially elastic with storey drift demands limited to approximately 0.5%, with the exception of a 15-story bare frame building, which showed a drift demand of approximately 1%. For building in the west, significant inelastic behavior were observed in columns, beams and shear walls of most buildings. The inter-storey drift demand was approximately 2.5%, except for a 15-storey bare frame building, which indicated an inter-storey drift demand of about 3.0%. The effect of masonry wall in reducing drift demands and ductility ratio was very significant. The results further indicated that the reduction in initial fundamental period of structures, due to the presence of masonry infills, results in a proportional reduction in seismic drift demands.

The current project also included an investigation on the credibility of push-over analysis as a tool for use in computing drift and ductility demands. It was found that its usefulness may be limited to structures behaving predominantly in the force mode, under code recommended distribution of equivalent static loads. Buildings of up to 10 stories, considered in the current investigation, showed good correlations between static and dynamic ductility demands, as well as static and dynamic drift demands when the comparison is made for a load stage producing the same roof displacement. Furthermore, the yield patterns observed under dynamic and static loads were very similar, signifying that push-over analysis may be used to identify potentially critical regions of structures during seismic response.

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Table of Contents

Abstract	i
Acknowledgment	iv
Table of Contents	v
List of Tables	ix
List of Figures	xii

Chapter 1

Introduction	1
1.1 General	1
1.2 Objectives and Scope	2
1.3 Previous Research	4
1.3.1 Drift Response of RC Buildings	4
1.3.2 M-P Interaction Models	6
1.3.3 Infill Masonry Panel	7
1.3.4 Inelastic Static Analysis	9
1.3.5 Displacement Based Design	12
1.3.6 Computer Programs for Nonlinear Analysis	14

Chapter 2

Computer Program	18
2.1 Introduction	18
2.2 Program Concept	18

2.3 Flexural Modelling	22
2.4 Modelling of Inelastic Shear	26
2.5 Modelling Anchorage Slip	30
2.6 Damping Matrix	31
2.7 Ductility Ratio	32
2.7.1 Flexural Ductility Ratio	32
2.7.2 Slip Ductility Ratio	33
2.7.3 Shear Ductility Ratio	34
2.7.4 Combined Ductility Ratio	35
2.8 Program Modifications	35
2.8.1 Axial Force-Moment Interaction Model	36
2.8.2 Hysteretic Model for Infill Panels	37
2.8.3 Static Inelastic Analysis (Push-Over)	40
2.8.4 P- Δ Effect	40
2.9 Verification of DRAIN-RC	45

Chapter 3

Selection of Structures and Earthquake Records	48
3.1 Introduction	48
3.2 Selection of Reference Structures	48
3.3 Modelling Structures for Inelastic Analysis	51
3.3.1 Moment-Curvature Relationship	53
3.3.2 Moment -Anchorage Slip Rotation Relationship	55
3.3.3 Moment-Shear Distortion Relationship	55
3.3.4 Modelling Effective Widths of Panel Struts	56
3.4 Selection of Earthquake Records	57

Chapter 4

Dynamic Inelastic Analysis	60
4.1 Introduction	60
4.2 Ground Motion Parameters	61
4.2.1 Frequency Content	61
4.2.2 Earthquake Intensity	63
4.3 Effects of Structural Parameters	65
4.3.1 P-M Interaction	65
4.3.2 Anchorage Slip	67
4.3.3 Inelastic Shear Deformation	68
4.3.4 P- Δ Effect	69
4.3.5 Effects of Structural Stiffness, Mass and Period	70
4.3.6 Effect of Flexural Strength	71

Chapter 5

Drift Demands	72
5.1 Introduction	72
5.2 Ground Motion Records Used	72
5.3 Computation of Drift Demands	75
5.3.1 Western Canada	76
5.3.1.1 Artificial Records	76
5.3.1.2 Previously Recorded Actual Earthquakes	80
5.3.1.3 Scaled Artificial Records to Produce NBCC-95 Base Shears	83
5.3.1.4 Conclusions on Drift Demands in the West	85

5.3.2 Eastern Canada	86
5.3.2.1 Artificial Records	86
5.3.2.2 Previously Recorded Actual Earthquakes	87
5.3.2.3 Scaled Artificial Records to Produce NBCC-95 Base Shears	88
5.3.2.4 Conclusions on Drift Demands in the East	89
 Chapter 6	
Static Inelastic (Push-Over) Analysis	91
6.1 Introduction	91
6.2 Push-Over Analysis of Buildings	91
6.2.1 Moment Resisting Frames	92
6.2.2 Frame-Shear Wall Buildings	97
6.3 Significance of Push-Over Analysis	99
 Chapter 7	
Summary and Conclusion	103
7.1 Summary	103
7.2 Conclusion	104
7.3 Recommendation for Further Research	107
 Bibliography	108
Tables	118
Figures	135

List of Tables

Table 2.1 Comparison of results for a single column in pure sway with P- Δ and without rigid end offset (example 1)	118
Table 2.2 Comparison of results for a single column in pure sway with P- Δ and with rigid end offset (example 2)	118
Table 2.3 Comparison of results for a single column with no sway with P- Δ and without rigid end offset (example 3)	118
Table 2.4 Comparison of results for a single column with no shear with P- Δ and without rigid end offset (example 4)	119
Table 2-5. Comparison of results for a single column. (example 5)	119
Table 2-6. Comparison of results for two bay- two storey frame analysis (example 6)	120
Table 2-7. Comparison of results for a single column with P- Δ and inelastic behaviour	120
Table 3.1- Data for design of buildings	121

Table 3-2. Reinforcement arrangements and sectional size of ten-storey frame building for western Canada	121
Table 3-3. Reinforcement arrangements and sectional size of ten-storey frame-shear wall building for western Canada	122
Table 3-4. Reinforcement arrangements and sectional size of ten-storey frame building for eastern Canada	122
Table 3-5. Reinforcement arrangements and sectional size of ten-storey frame-shear wall building for eastern Canada	123
Table 3-6. Artificial and selected ground motion records used in dynamic analysis.	124
Table 4.1. Comparison of drift response with and without inelastic shear deformation.	125
Table 4.2. Comparison of drift response with and without inelastic shear deformation for different level of yield in shear.	126
Table 4.3. Comparison of drift response with and without P- Δ	127
Table 5-1. Scale factor of artificial records for buildings in western Canada. .	128
Table 5-2. Scale factor of artificial records for buildings in eastern Canada. .	129

Table 5-3.Scale factor of artificial records for 10-storey infilled frame, $f_m=8.6$ MPa	130
Table 5-4.Scale factor of artificial records for 10-storey infilled frame, $f_m=15.2$ MPa	131
Table 5-5.Maximum strut forces recorded in masonry walls, expressed in percentage of compressive capacity	132
Table 6.1 Maximum roof displacement of buildings under dynamic load condition	133
Table 6.2 The static load stage at first initial yielding, collapsed due to instability and required to attain the same roof displacements due dynamic analysis.	134

List of Figures

Fig. 1.1 Axial Force-Moment Interaction Model by Takayanagi and Schnobrich [14].	135
Fig. 1.2 Axial Force-Moment Interaction Model by Sattcioglu [15].	135
Fig. 2.1 Average Acceleration Method.	136
Fig. 2.2 Elastic Element with Three Springs.	137
Fig. 2.3 Actual Beam Subjected to Different End Moments	137
Fig. 2.4 Moment Curvature Relationship for Actual and Idealized Beam	138
Fig. 2.5 Modified Tkeda's Hysteretic Model [46]	139
Fig. 2.6 Moment-Cord Rotation(due to shear) Relationship for Actual and Idealized Beam . .	140
Fig. 2.7 Hysteretic Model for Shear [49]	141
Fig. 2.8 Hysteretic Anchorage Slip Model [47]	142
Fig. 2.9 Ductility Definition	143
Fig. 2.10 Axial Force-Moment Interaction Diagram	143
Fig. 2.11 Procedure of Re-Loading with P-M Interaction	144
Fig. 2.12 Hysteretic Behavior of a Equivalent Trust Element (Infill Panel) [20,21]	145
Fig. 2.13 Rotation at Ends of a Beam Element	145
Fig. 2.14 Verification Examples 1 to 4	146
Fig. 2.15 Verification Example 5	147
Fig. 2.16 Verification Example 6	147
Fig.2.17 Verification Example. Single Column with Inelastic Behaviour	148
Fig. 3.1 Plan of Moment Resisting Frame Building	149
Fig. 3.2 Plan of Frame-Shear Wall Building	149

Fig. 3.3 Elevation View of 15, 10 and 5-Storey Frame Buildings without Infilled Masonry Panel	150
Fig. 3.4 Elevation View of 10-Storey Frame Building with 1,2 and 3 Bays Covered by Infilled Masonry Panel	151
Fig. 3.5 Elevation View of 15, 10 and 5-Storey Frame Shear Wall Buildings	152
Fig. 3.6 Two Dimensional Lumped Frame Used in Analysis	153
Fig. 3.7 Idealized Moment-Curvature Relationship	154
Fig. 3.8 Idealized Moment-Slip Rotation Relationship	154
Fig. 3.9 Shear Force- Shear Displacement Relationship [61]	155
Fig. 3.10 Equivalent Diagonal Strut [65]	155
Fig. 3.11 Response Spectra of Artificial Records for Vancouver	156
Fig. 3.12 Response Spectra of Artificial Records for Ottawa	157
Fig. 3.13 Comparison of Response Spectra of Artificial Records and UHS for	158
 Fig. 4.1 Drift Response of 10-Storey MRF Subjected to Selected Records for Western Canada	 159
Fig. 4.2 Drift Response of 10-Storey MRF Subjected to 1994 Northridge Records for Western Canada	159
Fig. 4.3 Drift Response of 10-Storey MRF Subjected to Artificial Records for Western Canada	160
Fig. 4.4 Drift Response of 10-Storey SHW Subjected to Selected Records for Western Canada	160
Fig. 4.5 Drift Response of 10-Storey SHW Subjected to 1994 Northridge Records for Western Canada	161
Fig. 4.6 Drift Response of 10-Storey SHW Subjected to Artificial Records for Western Canada	161
Fig. 4.7 Drift Response of 10-Storey MRF Subjected to Selected Records for Eastern Canada	162
Fig. 4.8 Drift Response of 10-Storey MRF Subjected to Artificial Records for Eastern Canada	162

Fig. 4.9 Drift Response of 10-Storey SHW Subjected to Selected Records for Eastern Canada	163
Fig. 4.10 Drift Response of 10-Storey SHW Subjected to Artificial Records for Eastern Canada	163
Fig. 4.11 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to Long Event No.2 for Eastern Canada	164
Fig. 4.12 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay records for Eastern Canada	165
Fig. 4.13 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada	166
Fig. 4.14 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to 1988 Saguenay E.Q. for Eastern Canada	166
Fig. 4.15 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada	167
Fig. 4.16 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay record for Eastern Canada	168
Fig. 4.17 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to Long Event No.2 for Eastern Canada	169
Fig. 4.18 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to 1988 Saguenay record for Eastern Canada	170
Fig. 4.19 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	171
Fig. 4.20 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected 1940 El Centro record 2 for Western Canada	172
Fig. 4.21 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	173
Fig. 4.22 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to 1940 El Centro E.Q. for Western Canada	173
Fig. 4.23 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	174
Fig. 4.24 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to 1940 El Centro E.Q. for Western Canada	175
Fig. 4.25 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to Long Event No.2 for Western Canada	176

Fig. 4.26 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to 1940 El Centro E.Q. for Western Canada	177
Fig. 4.27 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	178
Fig. 4.28 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to 1940 El Centro E.Q. for Western Canada	179
Fig. 4.29 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	180
Fig. 4.30 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to 1940 El Centro E.Q. for Western Canada	181
Fig. 4.31 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada	182
Fig. 4.32 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay E.Q. for Eastern Canada	183
Fig. 4.33 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada	184
Fig. 4.34 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay E.Q. for Eastern Canada	185
Fig. 4.35 Effect of P-M Interaction and Yield Moment on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	186
Fig. 4.36 Effect of P-M Interaction and Yield Moment on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	186
Fig. 4.37 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	187
Fig. 4.38 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	188
Fig. 4.39 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	189
Fig. 4.40 Effect of P-M Interaction on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	190
Fig. 4.41 Effect of P-M Interaction on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	190
Fig. 4.42 Effect of P-M Interaction on Ductility Response. 10-Storey MRF Subjected	

to Long Event No. 2 for Western Canada	191
Fig. 4.43 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	192
Fig. 4.44 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada	193
Fig. 4.45 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	194
Fig. 4.46 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to 1940 El Centro for Western Canada	195
Fig. 4.47 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada	196
Fig. 4.48 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay for Eastern Canada	197
Fig. 4.49 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada	198
Fig. 4.50 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay for Eastern Canada	199
Fig. 4.51 Effect of Anchorage Slip on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	200
Fig. 4.52 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	201
Fig. 4.53 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	202
Fig. 4.54 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada	203
Fig. 4.55 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	204
Fig. 4.56 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW Subjected to 1940 El Centro for Western Canada	205
Fig. 4.57 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada	206
Fig. 4.58 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay for Eastern Canada	207

Fig. 4.59 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada	208
Fig. 4.60 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay for Eastern Canada	209
Fig. 4.61 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	210
Fig. 4.62 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	211
Fig. 4.63 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	212
Fig. 4.64 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada	213
Fig. 4.65 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada	214
Fig. 4.66 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to 1940 El Centro for Western Canada	215
Fig. 4.67 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada	216
Fig. 4.68 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay for Eastern Canada	217
Fig. 4.69 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada	218
Fig. 4.70 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay for Eastern Canada	219
Fig. 4.71 Effect of P- Δ on Ductility Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada	220
Fig. 4.72 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada	221
Fig. 4.73 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	222
Fig. 4.74 Effect of Period on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	223
Fig. 4.75 Effect of Stiffness on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	224

Fig. 4.76 Effect of Mass on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	225
Fig. 4.77 Effect of Flexural Strength on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada	226
Fig. 5.1 Comparisons of NBCC-95 Design Response Spectra, UHS, and the Spectra for Artificial Records.	227
Fig. 5.2. Ratio of Base Shear versus Period for Scaled Artificial Records Used for 5-Storey Shear Wall Building.	228
Fig. 5.3 Ratio of Base Shear versus Period for Scaled Artificial Records Used for 15-Storey Frame Building.	229
Fig. 5.4 Drift and Ductility Response for 5-Storey MRF Subjected to Artificial Records for Western Canada.	230
Fig. 5.5 Drift and Ductility Response for 10-Storey MRF Subjected to Artificial Records for Western Canada.	231
Fig. 5.6 Drift and Ductility Response for 15-Storey MRF Subjected to Artificial Records for Western Canada.	232
Fig. 5.7 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=100$ mm, $f_m=8.6$ MPa)	233
Fig. 5.8 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=200$ mm, $f_m=8.6$ MPa)	234
Fig. 5.9 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=100$ mm, $f_m=8.6$ MPa)	235
Fig. 5.10 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=200$ mm, $f_m=8.6$ MPa)	236
Fig. 5.11 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=100$ mm, $f_m=8.6$ MPa)	237
Fig. 5.12 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=200$ mm, $f_m=8.6$ MPa)	238
Fig. 5.13 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Artificial Records ($f_m=8.6$ MPa).	239
Fig. 5.14 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=100$ mm, $f_m=15.2$ MPa)	240

Fig. 5.15 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=200$ mm, $f_m=15.2$ MPa)	241
Fig. 5.16 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=100$ mm, $f_m=15.2$ MPa)	242
Fig. 5.17 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=200$ mm, $f_m=15.2$ MPa)	243
Fig. 5.18 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=100$ mm, $f_m=15.2$ MPa)	244
Fig. 5.19 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=200$ mm, $f_m=15.2$ MPa).	245
Fig. 5.20 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Artificial Records ($f_m=15.2$ MPa).	246
Fig. 5.21 Drift Response for 5-Storey SHW Subjected to Artificial Records for Western Canada.	247
Fig. 5.22 Drift Response for 10-Storey SHW Subjected to Artificial Records for Western Canada.	248
Fig. 5.23 Drift Response for 15-Storey SHW Subjected to Artificial Records for Western Canada.	249
Fig. 5.24 Ductility Response for 5-Storey SHW Subjected to Artificial Records for Western Canada.	250
Fig. 5.25 Ductility Response for 10-Storey SHW Subjected to Artificial Records for Western Canada.	251
Fig. 5.26 Ductility Response for 15-Storey SHW Subjected to Artificial Records for Western Canada.	252
Fig. 5.27 Drift Response for 5-Storey MRF Subjected to All Selected Records for Western Canada.	253
Fig. 5.28 Drift Response for 15-Storey MRF Subjected to All Selected Records for Western Canada.	254
Fig. 5.29 Drift and Ductility Response for 5-Storey MRF Subjected to Actual Records for Western Canada.	255
Fig. 5.30 Drift and Ductility Response for 10-Storey MRF Subjected to Actual Records for Western Canada.	256
Fig. 5.31 Drift and Ductility Response for 15-Storey MRF Subjected	

to Actual Records for Western Canada.	257
Fig. 5.32 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=100$ mm, $f_m=8.6$ MPa)	258
Fig. 5.33 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=200$ mm, $f_m=8.6$ MPa)	259
Fig. 5.34 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=100$ mm, $f_m=8.6$ MPa)	260
Fig. 5.35 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=200$ mm, $f_m=8.6$ MPa)	261
Fig. 5.36 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=100$ mm, $f_m=8.6$ MPa)	262
Fig. 5.37 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=200$ mm, $f_m=8.6$ MPa)	263
Fig. 5.38 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=100$ mm, $f_m=15.2$ MPa).	264
Fig. 5.39 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=200$ mm, $f_m=15.2$ MPa).	265
Fig. 5.40 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=100$ mm, $f_m=15.2$ MPa).	266
Fig. 5.41 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=200$ mm, $f_m=15.2$ MPa).	267
Fig. 5.42 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=100$ mm, $f_m=15.2$ MPa).	268
Fig. 5.43 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=200$ mm, $f_m=15.2$ MPa).	269
Fig. 5.44 Time History of 2 nd Floor Inter-Storey Displacement with Progression of Inelasticity in Masonry Wall	270
Fig. 5.45 Hysteretic Behaviour of Diagonal Strut in 2 nd Floor Masonry Wall (Northridge No. 1, 23% g).	271
Fig. 5.46 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Selected Records ($f_m=8.6$ MPa).	273
Fig. 5.47 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Selected Records ($f_m=15.2$ MPa).	274

Fig. 5.48 Drift Response for 5-Storey SHW Subjected to All Selected Records for Western Canada.	275
Fig. 5.49 Drift Response for 15-Storey SHW Subjected to All Selected Records for Western Canada.	276
Fig. 5.50 Drift Response for 5-Storey SHW Subjected to Actual Records for Western Canada.	277
Fig. 5.51 Drift Response for 10-Storey SHW Subjected to Actual Records for Western Canada.	278
Fig. 5.52 Drift Response for 15-Storey SHW Subjected to Actual Records for Western Canada.	279
Fig. 5.53 Ductility Response for 5-Storey SHW Subjected to Actual Records for Western Canada.	280
Fig. 5.54 Ductility Response for 10-Storey SHW Subjected to Actual Records for Western Canada.	281
Fig. 5.55 Ductility Response for 15-Storey SHW Subjected to Actual Records for Western Canada.	282
Fig. 5.56 Drift and Ductility Response for 5-Storey MRF Subjected to Scaled Artificial Records for Western Canada.	283
Fig. 5.57 Drift and Ductility Response for 10-Storey MRF Subjected to Scaled Artificial Records for Western Canada.	284
Fig. 5.58 Drift and Ductility Response for 15-Storey MRF Subjected to Scaled Artificial Records for Western Canada.	285
Fig. 5.59 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=100$ mm, $f_m=8.6$ MPa).	286
Fig. 5.60 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=200$ mm, $f_m=8.6$ MPa).	287
Fig. 5.61 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Two bays, $t=100$ mm, $f_m=8.6$ MPa).	288
Fig. 5.62 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Two bays, $t=200$ mm, $f_m=8.6$ MPa).	289
Fig. 5.63 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Three bays, $t=100$ mm, $f_m=8.6$ MPa).	290
Fig. 5.64 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Three bays, $t=200$ mm, $f_m=8.6$ MPa).	291

Fig. 5.65 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=100$ mm, $f_m=15.2$ MPa).	292
Fig. 5.66 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=200$ mm, $f_m=15.2$ MPa).	293
Fig. 5.67 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Two bays, $t=100$ mm, $f_m=15.2$ MPa).	294
Fig. 5.68 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=200$ mm, $f_m=15.2$ MPa).	295
Fig. 5.69 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Three bays, $t=100$ mm, $f_m=15.2$ MPa).	296
Fig. 5.70 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=200$ mm, $f_m=15.2$ MPa).	297
Fig. 5.71 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Scaled Artificial Records ($f_m=8.6$ MPa).	298
Fig. 5.72 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Scaled Artificial Records ($f_m=15.2$ MPa).	299
Fig. 5.73 Drift Response for 5-Storey SHW Subjected to Scaled Artificial Records for Western Canada.	300
Fig. 5.74 Drift Response for 10-Storey SHW Subjected to Scaled Artificial Records for Western Canada.	301
Fig. 5.75 Drift Response for 15-Storey SHW Subjected to Scaled Artificial Records for Western Canada.	302
Fig. 5.76 Ductility Response for 5-Storey SHW Subjected to Scaled Artificial Records for Western Canada.	303
Fig. 5.77 Ductility Response for 10-Storey SHW Subjected to Scaled Artificial Records for Western Canada.	304
Fig. 5.78 Ductility Response for 15-Storey SHW Subjected to Scaled Artificial Records for Western Canada.	305
Fig. 5.79 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Artificial Records for Western Canada.	306
Fig. 5.80 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Selected Records for Western Canada.	307
Fig. 5.81 The Relationship Between Maximum Drift and The Fundamental Period of Buildings	

Subjected to Scaled Artificial Records for Western Canada.	308
Fig. 5.82 Drift and Ductility Response for 5-Storey MRF Subjected to Artificial Records for Eastern Canada.	309
Fig. 5.83 Drift and Ductility Response for 10-Storey MRF Subjected to Artificial Records for Eastern Canada.	310
Fig. 5.84 Drift and Ductility Response for 15-Storey MRF Subjected to Artificial Records for Eastern Canada.	311
Fig. 5.85 Drift Response for 5-Storey SHW Subjected to Artificial Records for Eastern Canada.	312
Fig. 5.86 Drift Response for 10-Storey SHW Subjected to Artificial Records for Eastern Canada.	313
Fig. 5.87 Drift Response for 15-Storey SHW Subjected to Artificial Records for Eastern Canada.	314
Fig. 5.88 Ductility Response for 5-Storey SHW Subjected to Artificial Records for Eastern Canada.	315
Fig. 5.89 Ductility Response for 10-Storey SHW Subjected to Artificial Records for Eastern Canada.	316
Fig. 5.90 Ductility Response for 15-Storey SHW Subjected to Artificial Records for Eastern Canada.	317
Fig. 5.91 Drift and Ductility Response for 5-Storey MRF Subjected to 1988 Saguenay Record for Eastern Canada.	318
Fig. 5.92 Drift and Ductility Response for 10-Storey MRF Subjected to 1988 Saguenay Record for Eastern Canada.	319
Fig. 5.93 Drift and Ductility Response for 15-Storey MRF Subjected to 1988 Saguenay Record for Eastern Canada.	320
Fig. 5.94 Drift Response for 5-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.	321
Fig. 5.95 Drift Response for 10-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.	322
Fig. 5.96 Drift Response for 15-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.	323
Fig. 5.97 Ductility Response for 5-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.	324

Fig. 5.98 Ductility Response for 10-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.	325
Fig. 5.99 Ductility Response for 15-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.	326
Fig. 5.100 Drift and Ductility Response for 5-Storey MRF Subjected to Scaled Artificial Records for Eastern Canada.	327
Fig. 5.101 Drift and Ductility Response for 10-Storey MRF Subjected to Scaled Artificial Records for Eastern Canada.	328
Fig. 5.102 Drift and Ductility Response for 15-Storey MRF Subjected to Scaled Artificial Records for Eastern Canada.	329
Fig. 5.103 Drift Response for 5-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.	330
Fig. 5.104 Drift Response for 10-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.	331
Fig. 5.105 Drift Response for 15-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.	332
Fig. 5.106 Ductility Response for 5-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.	333
Fig. 5.107 Ductility Response for 10-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.	334
Fig. 5.108 Ductility Response for 15-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.	335
Fig. 5.109 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Artificial Records for Eastern Canada.	336
Fig. 5.110 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Selected Record for Eastern Canada.	337
Fig. 5.111 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Scaled Artificial Records for Eastern Canada.	338
Fig. 6.1 Force-Lateral Drift Relationship for 10-Storey MRF building in Vancouver.	339
Fig. 6.2 Drift and Ductility Response at Selected Stages of Static Loading for 10-Storey MRF building in Vancouver.	340
Fig. 6.3 The effect of P- Δ in Push-Over Analysis for	

10-Storey MRF building in Vancouver.	341
Fig. 6.4 Force-Lateral Drift Relationship for 5-Storey MRF building in Vancouver.	342
Fig. 6.5 Drift and Ductility Response at Selected Stages of Static Loading for 5-Storey MRF building in Vancouver.	343
Fig. 6.6 The effect of P- Δ in Push-Over Analysis for 5-Storey MRF building in Vancouver.	344
Fig. 6.7 Force-Lateral Drift Relationship for 15-Storey MRF building in Vancouver.	345
Fig. 6.8 Drift and Ductility Response at Selected Stages of Static Loading for 15-Storey MRF building in Vancouver.	346
Fig. 6.9 The effect of P- Δ in Push-Over Analysis for 15-Storey MRF building in Vancouver.	347
Fig. 6.10 Force-Lateral Drift Relationship for 10-Storey MRF building in Ottawa.	348
Fig. 6.11 Drift and Ductility Response at Selected Stages of Static Loading for 10-Storey MRF building in Ottawa.	349
Fig. 6.12 The Effect of P- Δ in Push-Over Analysis for 10-Storey MRF building in Ottawa.	350
Fig. 6.13 Force-Lateral Drift Relationship for 5-Storey MRF building in Ottawa.	351
Fig. 6.14 Drift and Ductility Response at Selected Stages of Static Loading for 5-Storey MRF building in Ottawa.	352
Fig. 6.15 The Effect of P- Δ in Push-Over Analysis for 5-Storey MRF building in Ottawa.	353
Fig. 6.16 Force-Lateral Drift Relationship for 15-Storey MRF building in Ottawa.	354
Fig. 6.17 Drift and Ductility Response at Selected Stages of Static Loading for 15-Storey MRF building in Ottawa.	355
Fig. 6.18 The Effect of P- Δ in Push-Over Analysis for 15-Storey MRF building in Ottawa.	356
Fig. 6.19 Force-Lateral Drift Relationship for 10-Storey SHW building in Vancouver.	357
Fig. 6.20 Drift Response at Selected Stages of Static Loading for 10-Storey SHW building in Vancouver.	357
Fig. 6.21 Ductility Response at Selected Stages of Static Loading for 10-Storey SHW building in Vancouver.	358

Fig. 6.22 The Effect of P- Δ in Push-Over Analysis for 10-Storey SHW building in Vancouver.	359
Fig. 6.23 Force-Lateral Drift Relationship for 5-Storey SHW building in Vancouver.	360
Fig. 6.24 Ductility Response at Selected Stages of Static Loading for 5-Storey SHW building in Vancouver.	361
Fig. 6.25 Force-Lateral Drift Relationship for 15-Storey SHW building in Vancouver.	362
Fig. 6.26 Ductility Response at Selected Stages of Static Loading for 15-Storey SHW building in Vancouver.	363
Fig. 6.27 Force-Lateral Drift Relationship for 5-Storey SHW building in Ottawa.	364
Fig. 6.28 Force-Lateral Drift Relationship for 10-Storey SHW building in Ottawa.	365
Fig. 6.29 Force-Lateral Drift Relationship for 15-Storey SHW building in Ottawa.	366
Fig. 6.30 Comparison of Static and Dynamic Drift and Ductility Response for 5-Storey MRF building in Vancouver.	367
Fig. 6.31 Comparison of Static and Dynamic Drift and Ductility Response for 10-Storey MRF building in Vancouver.	368
Fig. 6.32 Comparison of Static and Dynamic Drift and Ductility Response for 15-Storey MRF building in Vancouver.	369
Fig. 6.33 Comparison of Static and Dynamic Drift Response for 5-Storey SHW building in Vancouver.	370
Fig. 6.34 Comparison of Static and Dynamic Ductility Response for 5-Storey SHW building in Vancouver.	371
Fig. 6.35 Comparison of Static and Dynamic Drift Response for 10-Storey SHW building in Vancouver.	372
Fig. 6.36 Comparison of Static and Dynamic Ductility Response for 10-Storey SHW building in Vancouver.	373
Fig. 6.37 Comparison of Static and Dynamic Drift Response for 15-Storey SHW building in Vancouver.	374
Fig. 6.38 Comparison of Static and Dynamic Ductility Response for 15-Storey SHW building in Vancouver.	375
Fig. 6.39 Comparison of Static and Dynamic Drift and Ductility Response	

for 5-Storey MRF building in Ottawa.	376
Fig. 6.40 Comparison of Static and Dynamic Drift and Ductility Response for 10-Storey MRF building in Ottawa.	377
Fig. 6.41 Comparison of Static and Dynamic Drift and Ductility Response for 15-Storey MRF building in Ottawa.	378
Fig. 6.42 Comparison of Static and Dynamic Drift Response for 5-Storey SHW building in Ottawa.	379
Fig. 6.43 Comparison of Static and Dynamic Ductility Response for 5-Storey SHW building in Ottawa.	380
Fig. 6.44 Comparison of Static and Dynamic Drift Response for 10-Storey SHW building in Ottawa.	381
Fig. 6.45 Comparison of Static and Dynamic Ductility Response for 10-Storey SHW building in Ottawa.	382
Fig. 6.46 Comparison of Static and Dynamic Drift Response for 15-Storey SHW building in Ottawa.	383
Fig. 6.47 Comparison of Static and Dynamic Ductility Response for 15-Storey SHW building in Ottawa.	384
Fig. 6.48 Yielding Pattern Obtained by Dynamic and Push-Over Analysis of 5-storey MRF in Vancouver.	385

Chapter 1

Introduction

1.1 General

Structures are usually designed for strength and subsequently checked for serviceability. One of the serviceability requirements is to check against maximum allowable deflection. In earthquake resistant structures, inelastic deformability of structural members plays an important role to dissipate seismic induced energy. Structural members are designed to have adequate inelastic deformability so that they can deform without a significant decay in strength. Therefore, in seismic design the consideration of deformation demands becomes a critical design step.

Deformation demands may be expressed in a number of different ways. A commonly accepted quantity is lateral drift. Lateral drift is defined as lateral displacement divided by member length, and is usually expressed in percentage of member length. Lateral drift is often used to express deformation characteristics of entire structure or entire floor, in which case both the displacement and the length terms are of the entire building or the storey height, respectively.

Seismic design provisions usually limit maximum storey drift to 2% as an acceptable

level of deformability. Most design codes recommend a simple procedure for estimating inelastic displacement demands by specifying an amplification factor to be applied to elastic design drifts obtained from design spectra. This simple approach neglects to incorporate the effects of a number of structural and ground motion parameters, and may not reflect realistic drift demands. Furthermore, issues related to different characteristics of hysteretic response of structures and P- Δ deformations are not properly addressed. The problem becomes further complicated for reinforced concrete structures which show substantially different hysteretic behavior depending on structural properties. Hence there is need to establish inelastic deformation demands for reinforced concrete structures.

Dynamic inelastic response, and hence lateral drift demands of structures can be determined through response history analysis. This requires a reliable analytical tool, usually in the form of a computer software, and in depth understanding of the significance of analytical models employed in such a tool. Although significant advances have been made in recent years to establish the characteristics of hysteretic response under seismic excitations, there is still lack of computer software fully equipped with relevant analytical models that account for the primary components of inelastic deformation.

The current research has been undertaken to respond to some of the above needs in terms of developing an analysis tool and establishing the significance of analysis parameters, as well as establishing inelastic drift demands for reinforced concrete structures.

1.2 Objectives and Scope

The overall objective of the research project is to establish seismic deformation

demands for reinforced concrete structures . The objective also includes development of an analytical tool, in the form of a computer software, applicable to seismic analysis of reinforced concrete structures, and investigation of the significance of analysis parameters in conducting response history analysis. The following tasks form the scope of the investigation:

1. Review of previous experimental and analytical research on seismic drift response of reinforced concrete buildings with and without concrete structural walls or masonry infill panels.
2. Develop an improved version of computer software DRAIN-2D, called DRAIN-RC, applicable to analysis of reinforced concrete structures. The new computer program will include features for inelastic flexure with and without M-P interaction effects, inelastic shear, anchorage slip, infill panels, and P- Δ effects for inelastic dynamic analysis of planer RC structures. It will also include a capability to conduct inelastic static "Push-over" analysis.
3. Select and design six reinforced concrete buildings with different heights and seismic resisting systems. The structures will include frames with and without structural walls.
4. Select suitable ground motions, representative of seismicity in different regions of Canada.
5. Conduct dynamic inelastic response history analyses to investigate:
 - i) relationships between structural parameters and drift index;
 - ii) overall structural drift and inter-storey drift demands;
 - iii) credibility of push-over inelastic analysis as an analytical tool for establishing drift demands;
 - iv) effects of infill panels, P-M interaction, P- Δ considerations, inelastic shear and anchorage slip deformations.
 - v) effects of ground motion parameters and expected probability of

occurrences on lateral drift demands.

6. Evaluate the results of analyses and formulate design recommendations in terms of drift requirements for building structures.

1.3 Previous Research

A literature survey has been conducted to gather previously generated information in the area of dynamic response and lateral drift demands of reinforced concrete structures. This was done in several categories, consistent with the scope of the investigation. First, research on drift response of reinforced concrete structures was studied. Subsequently, research on infill panels and M-P interaction effects were compiled since these two effects were introduced into the computer program DRAIN-RC by the author. Literature on static inelastic analysis (Push-over) was reviewed and discussed, as this was another feature that was incorporated in to the computer software. Displacement-based design procedures, proposed by previous researchers, were also reviewed and discussed under separate heading. Finally, a survey of available computer programs relevant to the objectives of the current research was conducted and presented. The literature survey, broken down to the above categories, is presented in the following sections.

1.3.1 Drift Response of RC Buildings

Several researchers performed experimental and analytical research to study drift response of reinforced concrete buildings. Heidebrecht [2] investigated the drift demand of a six storey reinforced concrete frame structures used for office-type occupancy in Vancouver. He used IDARC [3] program to execute inelastic response history analysis and found that the maximum transient roof displacement and inter-storey drift were 0.77% and 1.18%, respectively. He also found that maximum

curvature ductility ratio for columns was 3.15 and for beams 5.59. The analyses reported included inelasticity only in flexure.

Bariola [4] conducted nonlinear seismic analyses of six eight-storey concrete frames to investigate drift response of medium-rise buildings. Analyses were performed using a computer software called, LARZ. The software was intended for planar frames with inelastic flexural action considered in columns and beams. Ground motion could only be specified in the horizontal direction. He found that the response was influenced by strength in low and medium frames but deformations became more important in tall frames. The displacement response was also estimated using linear analysis. Effective period was used for this purpose, computed as twice the natural period based on gross cross-sectional dimensions. Damping was taken as 2% of critical damping. Comparisons of linear and nonlinear analyses indicated reasonably good agreement in displacements for most input ground motions.

Uang and Maarouf [5,6,7] conducted analytical studies on seismic drifts of multi-storey RC frames. The results indicated that the deflection amplification factor specified in seismic design provision of UBC[1], NEHRP Recommended Provisions[8] and National Building Code of Canada [9] were very low and could lead to unconservative drift estimations. The researchers used computer program DRAIN-2D [10] with inelastic flexural springs only.

Bonacci [11] conducted a comprehensive test program to study seismic drift of reinforced concrete structures. He conclude that measured peak displacements displayed a nearly constant relationship to linear spectral estimates for a damping ratio of 2%. By considering displacements and moment response histories which were computed for 4032 individual cases, he found that for modelling inelastic behavior of

concrete, a post-yield slope of 5% of effective elastic slope was marginally more consistent than 2, 10 and 15%. He further concluded that a value of 0.4 for the unloading slope exponent in the Takeda hysteresis model gave the most consistent results.

1.3.2 M-P Interaction Models

Interaction of axial force with flexure during response was investigated both analytically and experimentally. The previous research concentrated on reinforced concrete columns and coupled walls. Mahin and Bertero [12] and Aktan and Bertero [13] introduced M-P interaction effects to elasto-plastic models. These models did not incorporate stiffness degradation of reinforced concrete.

Takayanagi and Schnorbrich [14] modified Takeda's model to consider the effects of M-P interaction. Fig. 1.1 shows the moment curvature curve of this model. The model includes a set of tri-linear primary curves, each relating to a specific level of axial load. Saatcioglu et al.[15] introduced the M-P interaction effects to Powell's version of Takeda's model and used it in investigating non-linear response of coupled wall structures. In this model, the stiffness is updated continuously based on the level of axial force computed at the end of the previous time step. Fig. 1.2 shows the basic characteristic of the model. This model was incorporated into the computer program DRAIN-RC as part of the current investigation. Related details are presented in Chapter 2. Keshavarzian and Schnobrich [16] also performed inelastic analyses of coupled wall structures. They considered the effect of varying axial load on sectional moment curvature relationship. The hysteresis rules adopted were those proposed by Takeda with a bilinear primary curve.

1.3.3 Infill Masonry Panel

Many researchers undertook extensive investigations to determine the effects of masonry infill panels on behavior of buildings. Some also presented analytical models that can be used in dynamic analysis.

Liauw [17] conducted an experimental investigation on dynamic response of four-storey models of steel frames with reinforced concrete infills. He concluded that connectors between infills and frame elements played an important role. The stiffness of each frame increased considerably by installing infill masonry panels.

Bertero and Brokken [18,19] conducted experimental and analytical research on the effects of infills on seismic resistance of buildings. The researchers tested 1/3-scale models of an 11 storey-three bay reinforced concrete frame under quasi-static cyclic and monotonic loads. They found that the addition of either unreinforced or reinforced infills to moment resisting frames increased significantly the lateral stiffness and strength of the frames. They also conclude that hysteretic behavior depended upon the type of infill, the amount and arrangement of reinforcement, the way that the panel is attached to the frame and the loading history. They showed that the initial tangential inter-storey lateral stiffness of infilled frames was more than 10 times the stiffness of bare frames. The addition of infills introduced significant changes to the dynamic characteristic of bare moment resisting frames. In the linear elastic range, the fundamental period decreased more than 54% while the mass increased no more than 10%. In the inelastic range, the major change was in the pattern of lateral deformations. Presence of infill panels decreased displacement demands relative to those corresponding to the bare frame building. The decrease varied between 56% and 85% for cases where all the frames were stiffened by infill walls. This decrease in displacement demand was reported to be a significant advantage for the use of infills.

Klingner and Bertero [20,21] performed an investigation on the hysteretic behavior of infilled masonry panels. After conducting a series of tests, they introduced a pair of equivalent diagonal strut elements which could be used to model each infill panel. These elements would be designed to exhibit strength, stiffness, and deterioration characteristics similar to those observed in the experiments. This model was chosen for implementation into computer software DRAIN-RC as part of the current research, and is discussed in chapter 2.

Altin, Ersoy and Tankut [22] investigated the behavior of cast-in-place reinforced concrete infilled frames under seismic loading. They found that infills increased strength and stiffness of frames significantly under lateral loading, provided that they were properly connected to the frames. They stated that the introduction of infill panels to frames of building structures result in significant changes in dynamic characteristic of buildings. These changes depend on the number of frames that are infilled, as well as the location of these frames in the building. Due to the relative increase in stiffness, the fundamental period of one test specimen decreased by about 80%. It was observed that beyond the ultimate stage, infilled frames were able to dissipate significant amount of energy while providing reduced load resistance.

Mehrabi [23] conducted series of tests consisting of twelve 1/2-scale, single-storey, single-bay RC frames to evaluate the influence of masonry infill panels on seismic performance when the frames were designed in accordance with current code provision. The experimental results indicated that infill panels can considerably improve the performance of RC frames in terms of load resistance and energy-dissipation capability. Lateral loads developed by infilled framed specimens were consistently higher than those of the bare frames. This observation was also true for the least ductile specimen, deforming up to a drift level of 2%. This study indicated

that for frames that are properly designed for strong seismic loads, infill panels will most likely have a beneficial influence on their performances.

Saneinejad and Hobbs [24] proposed a new method of analysis and design for steel frames with concrete or masonry infill walls subjected to in-plane forces, based on results from series of nonlinear finite-element analyses. The infilled frames were modelled as braced frames, replacing the infills by equivalent diagonal struts. This model has been used in computer software IDARC2D, version 4.0 [25].

1.3.4 Inelastic Static analysis

The use of linear elastic analysis for seismic design of structures is often questioned among researchers. Non-linear time history analysis of multi-degree of freedom structures, on the other hand, is not practical for everyday design and may require specialized expertise on the topic. Therefore, non-linear static analysis under monotonically increasing lateral load (push-over analysis) has been gaining momentum as a rational and yet reasonably straight forward procedure. The push-over analysis has been formulated, evaluated, and proposed in several studies in different formats.

Saiidi and Sozen [26] introduced a simple analytical model called the Q-model to estimate the displacement histories of multistory reinforced concrete structures subjected to strong ground motions. In developing this model they made two major simplification. First they replaced a multi-degree-of-freedom (MDOF) model of a structure by a single-degree-of-freedom (SDOF) oscillator. Secondly they approximated the variation of incremental stiffness properties of the whole structure by a single nonlinear spring. The specifications of SDOF oscillator were determined based on a calculated relationship between base moment and lateral displacement under monotonically increasing load. Two researchers compared displacements of

eight small-scale reinforced concrete structures which were tested under strong ground motions with those obtained analytically. The results showed that the Q-model produced good correlations in both high-amplitude and low-amplitude ranges.

Moghadam and Tso [27] conducted nonlinear static push over analysis of asymmetric buildings. They designed two 7-storey reinforced concrete ductile moment resisting frame buildings, one symmetric and the other asymmetric, based on the Canadian practice [9]. A 3-D analysis was carried out using computer program CANNY-C. They investigated displacements, inter-storey drift, ductility and hinging patterns. It was shown that, for the same level of lateral load, the exterior frame of the asymmetrical building experienced significantly larger inter-storey drift and larger ductility demands for both columns and beams. Also the edge frame of the asymmetrical building developed more hinges compare to that of symmetrical building. The damage pattern was very different in two buildings. This was explained by different load redistributions among different frames when members were in the inelastic region.

Biddah, Heideberecht and Naumoski [28] used push-over tests to evaluate damage in RC frame structures subjected to strong seismic ground motions. They used modified version of IDARC[3] computer program to analyse three frames with 4, 10 and 18 stories. A simplified method was suggested, involving two push-over analyses of actual frames and an inelastic dynamic analysis of a transformed equivalent single-degree-of-freedom system. They found that this simplified method provided reasonable results for frames that oscillated mainly in the first mode. They also determined the mean global damage indices for three different frames which were in the acceptable range for structures subjected to strong seismic motions. Furthermore, they introduced dynamic magnification factor to consider the effects of higher modes

which gave good results.

Lawson, Vance and Krawinkler [29] investigated the credibility of nonlinear static push-over analysis. They tried to answer why, when and how push-over analysis can be performed. They analysed four steel buildings with 2 , 5, 10 and 15 stories. The results of nonlinear dynamic analysis for seven different ground motion records were compared with those of nonlinear static push-over analysis. The computer software DRAIN-2DX[30] was used to conducted both dynamic and static analyses. The results were expressed in term of story drift, plastic hinge rotation and hysteretic energy. Some assumptions were made in the analysis to simplify the procedure. The gravity loads consisted of dead load plus fifty percent of live load. The P- Δ effect was not included, and P-M interaction on columns was considered. They concluded that although push-over analysis was a useful tool, it was not always dependable for estimating inelastic strength and deformation demands, or for determining the weaknesses in design. They claimed that the major benefit of push-over analysis was to allow the design engineer to detect important seismic response characteristics, including a good judgment on deformation demands. It was further concluded that while nonlinear static analysis was better than elastic static analysis, it could create a wrong feelings of security for the structure. They found that the correlation in deflection profiles and deformation demands obtained by push-over and inelastic dynamic analyses was good for short period structures which were governed by the first mode of vibrations. However, the correlation for taller structures was not good. This was attributed to higher mode effects. The researchers also investigated lateral load distribution along the height, and concluded that the distribution of shear was in good agreement with that proposed by current building codes. However, the global hysteretic energy dissipated in dynamic analysis was in poor correlation with that indicated by push-over analysis. Hence, it was concluded that the push-over analysis

was not a reliable tool to measure the cumulative damage demand.

1.3.5 Displacement Based Design

A number of researchers conducted several experimental and analytical investigations to establish relationships between the drift response of buildings and behavior of structural members. The results of some experiments on full scale reinforced concrete beams and columns [31,32,33] showed that frames with reasonably detailed and constructed members are expected to perform satisfactorily if the inter-storey drift ratios were kept below 2%. During the last three decades a major analytical effort has been dedicated to understanding structural response under dynamic excitations [34,35,36]. The general response of a structural system to dynamic loading was determined by establishing dynamic characteristics of the load as well as of the structure. It was found that the response of a linear elastic system was particularly sensitive to harmonic loading because of the effects of resonance. In contrast to elastic systems, inelastic systems yielded and experienced changes in period, which kept the system from resonance during harmonic loading.

Newmark, N., M., [37,38,39] showed that for each earthquake motion there were different period ranges in the elastic response spectra which showed almost constant response quantities. He also found that in those period ranges of constant velocity and displacement response, the maximum inelastic displacement was roughly the same as the elastic displacement response if the elastic displacement spectrum was represented by a smooth curve.

Shimazaki and Sozen [40,41] studied displacement and energy response of structures to earthquake ground motions. They found that when the periods of structures were longer than a specific value, T_g , the spectral value of input energy was constant or

decreased slightly, regardless of the strength of the system. The characteristic period T_g of each ground motion was identified as the period corresponding approximately to the period at the intersection of the constant velocity and constant acceleration response regions.

Qi and Moehle [42] conducted an analytical investigation to develop techniques by which the peak inelastic displacement response of structures subjected to design level earthquakes could be easily predicted. The researchers subsequently developed a displacement-based seismic design method. They found that for a single-degree-of-freedom system subjected to earthquake ground motions, the displacement response could be categorized in two period ranges based on the value of characteristic period of ground motion T_g . When the period of system was greater than T_g , the peak displacement response of a system was mainly independent of the strength of the system and the maximum inelastic displacement could be evaluated as being the same as the elastic value. If T_g was greater than the period of system, both strength and stiffness of the system affected the peak inelastic response. The researchers also presented a simple method to estimate the maximum inelastic response of a single-degree-of-freedom of system. They suggested a method to evaluate the displacement response of regular multi-story structures which could be modelled by an equivalent SDOF system. They showed that the displacement-based method provided a direct evaluation of the maximum inter-story drift which the structure may experience during the design earthquake. Hence, based on the assessed displacement or drift limit, the damage of structural and non-structural components as well as the detailing requirements of structural members could be found with greater reliability. They said that the displacement approach focussed on both strength and stiffness of structure to withstand earthquakes, whereas in the conventional strength method the stiffness of a structure was not clearly accounted for. In a particular range of period, the

displacement approach showed the greater importance of structural stiffness. Moehle [43] studied the advantage of displacement-based design method of RC structures subjected to earthquakes. He used displacement as a design parameter and presented several examples in which he designed reinforced concrete structural elements directly by a displacement-based design approach.

1.3.6 Computer Programs for Nonlinear Analysis

Computation of inelastic drift demands of buildings can only be established by means of computer analysis. Drift demands of multiple-degree-of-freedom structures during earthquakes can be established by conducting dynamic analysis. Therefore, the literature survey included review of available computer software that can fulfil this function. This review was limited to those programs that have proper analytical models for analysis of reinforced concrete structures, to be consistent with the scope of the current investigation. Some computer programs were found to be available, with capabilities of analysing reinforced concrete structures under seismic excitations. The following sections provide a brief overview of these programs.

a) IDARC2D: This computer program was originally developed by Park et al. [44] in 1987 for inelastic damage analysis of buildings. It was later modified by Kunnath et al. 1992 [3]. Valles et al. [25] subsequently introduced a number of enhancements in 1996, including:

- 1- Viscoelastic, friction, and hysteretic damper macro elements.
- 2- Macro model for infill panel elements.
- 3- Spread plasticity and yield penetration.
- 4- Hysteresis modules.
- 5- Damage indicators.
- 6- Push-over options.

Although the most recent version of the program appears to be quite powerful, it lacks a number of important hysteretic models for reinforced concrete, including anchorage slip, and P-M interaction effects.

b) DRAIN-2D: General purpose computer program for dynamic analysis of inelastic plane structures was originally developed by Kanaan and Powell [10] in 1973. A reinforced concrete beam element with degrading stiffness characteristics was then introduced by Powell in 1975 [46]. A number of different universities and research organizations developed their versions of the program, including that developed at the University of Ottawa by Alsiwat and Saatcioglu [47] in 1992. This version was developed to run on personal computers and utilized multiple spring beam elements, consisting of an elastic beam and three inelastic springs at each end. It included;

- 1- a hysteretic flexural model for R/C members based on the Takeda model [48],
- 2- a hysteretic shear model developed by Ozcebe and Saatcioglu [49], and
- 3- a hysteretic model for anchorage slip developed by Saatcioglu, Alsiwat and Ozcebe [50,51].

This version was modified by the author to conduct the current research project. The recent modifications are discussed in detail, in Chapter 2.

c) DRAIN-2DX: This is a improved version of DRAIN-2D general purpose computer program for static and dynamic analysis of plane structures, developed by Prakash, Powell and Campbell [30] in 1993. It performs nonlinear static and dynamic analyses, and for dynamic analysis considers ground accelerations (all supports moving in phase), ground displacements (supports may move out of phase), imposed dynamics loads (e.g., wind) and specified initial velocities (e.g., impulse loading). Static and dynamic loads can be applied in any sequence. For example, a dynamic analysis can be performed to damage the structure, and static loads can then be applied to

investigate its behaviour in the damaged state. If a static load follows a dynamic load, a special “restore to static state” analysis is performed to bring the structure to rest before the static loads is applied. The structure state can be saved to the end of any analysis, and the analysis can be restarted from any saved state. The step-by-step integration scheme for dynamic analysis varies the time step during the analysis, on the basis of input error tolerances. Energy balance computations are performed, identifying the static work, the energy absorbed by viscous damping, the kinetic energy, and the input energy. Mode shapes and periods can be calculated from any state. Linear response spectrum analyses can be performed for the unstressed state. Static nonlinear analysis is performed by an event-to-event scheme, where each event corresponds to a significant change in stiffness.

The element library contains Type 01, inelastic truss bar; Type 02, simple inelastic beam column; Type 04, simple inelastic connection, which allows for translational as well as rotational for transfer; Type 06, elastic panel element, which allows vertical, horizontal extensional and flexural stiffnesses to be input; Type 09, inelastic link element , that can act in compression/tension with initial gap or axial force; and Type 15, “ fiber” beam-column element for steel and reinforced concrete members. The element include capabilities for event and internal energy calculations. Inelastic static analysis can be carried out , with the ability to trace sequences of hinge formation and to continue into the post-buckling range.

DRAIN-2DX includes the following analysis options:

- 1- static gravity-linear static analysis for any combination of element and nodal loads;
- 2- other static-nonlinear static analysis for any combination of nodal loads;
- 3- ground acceleration - nonlinear dynamic analysis of motions defined by X, Y and/or R accelerograms;

- 4- mode shapes-calculation of mode shapes and periods, in the initial elastic state or any later state;
 - 5- response spectrum-linear dynamic analysis for specified X and/or Y response spectra, using mode shapes and frequencies in the initial elastic state only;
 - 6- initial velocity (impulse) - nonlinear dynamic analysis for specified initial nodal velocities, an option that can be used to compute response following an impact or to evaluate energy absorption capacity;
 - 7- ground displacement- nonlinear dynamic analysis for ground motions defined by X, Y and/or R displacement records at support points, including different (out of phase) motions at different supports;
 - 8- Dynamic nodal load- nonlinear dynamic analysis for dynamic forces defined by X, Y and/or force records on the nodes.
- Envelope results (maximum effects) and/or time history results can be output to a print file. Time history results are ordered node-by-node and element-by-element.

Chapter 2

Computer Program

2.1 Introduction

A new version of computer program DRAIN-2D was developed and employed to conduct dynamic inelastic response history analysis. This program was originally developed by Kanaan and Powell at the university of California, Berkeley [10]. DRAIN-2D is a general purpose computer program for static and dynamic analysis of planer structures. The original version contains ten different types of elements, one of which is a beam element with degrading stiffness characteristics, suitable for modelling reinforced concrete element. The original DRAIN-2D was modified by Alsiwat at the University of Ottawa [47] who introduced a hysteretic shear model and a hysteretic model for anchorage slip. The program was further modified by the author. The recent modifications include introduction of M-P interaction effects to flexural model and implementation of a hysteretic model for masonry infill panels. Furthermore, push-over inelastic static analysis was added, and the P- Δ effects were introduced. This chapter is devoted to the review of original program, and the modifications introduced to create a new version of DRAIN-2D, called DRAIN-RC.

2.2 Program Concept

The structure to be analysed is idealized as a planar assemblage of discrete elements. Degrees of freedom are considered only at nodes. Each node has up to three degrees of freedom which can be restrained at supports or can be combined with the other

node's degrees of freedom. The global stiffness matrix is formed by assembling individual element stiffnesses, which are determined by the direct stiffness method. This method is a special form of the displacement method. The basis of this method is that the state of deformation of the structure can be define in terms of a finite number of displacement parameters associated with the nodes. The displacements of some nodes are known to be zero or may reasonably be assumed to be zero. For example, because the supports are stiff in comparison with the frame, their displacements may be assumed to be zero. It is a reasonable assumption that the axial deformation of certain members is zero without introducing a significant error. For example to simulate the behavior of a floor as a rigid diaphragm, displacements of all beams in the horizontal direction is assumed to be equal. By this assumption all degrees of freedom of nodes in the same floor in X direction will be identical. The structure mass is assumed to be lumped at the nodes so that the mass matrix is diagonal.

Horizontal and vertical accelerations of ground motion can be input at time intervals. The step-by-step integration method is used to find dynamic response of a structure. It is assumed that during a given time step accelerations remain constant and the tangent stiffness of structure is used for that time step. The following incremental equation of motion is employed for a finite time step Δt .

$$[M](\Delta \ddot{r}) + [C_T](\Delta \dot{r}) + [K_T](\Delta r) = (\Delta F) \quad (2-1)$$

in which $(\Delta \ddot{r})$, $(\Delta \dot{r})$, (Δr) and (ΔF) are small increments of acceleration, velocity, displacement and load, respectively during time step Δt . Referring to Fig. 2.1, the velocity and displacement at time $n+1$ are given by the following expressions:

$$\dot{r}_{n+1} = \dot{r}_n + \frac{\Delta t}{2} (\ddot{r}_n + \ddot{r}_{n+1}) \quad (2-2)$$

$$r_{n+1} = r_n + \Delta t \dot{r}_n + \frac{\Delta t^2}{4} (\ddot{r}_n + \ddot{r}_{n+1}) \quad (2-3)$$

Rewriting Eqs. 2-2 and 2-3 in terms of Δr , gives:

$$\Delta \dot{r} = \frac{2}{\Delta t} \Delta r - 2 \dot{r}_n \quad (2-4)$$

$$\Delta \ddot{r} = \frac{4}{\Delta t^2} \Delta r - \frac{4}{\Delta t} \dot{r}_n - 2 \ddot{r}_n \quad (2-5)$$

Substituting Eqs. 2-4 and 2-5 into Eq. 2-1 results in the following expressions.

$$[K'] (\Delta r) = (F') \quad (2-6)$$

in which :

$$[K'] = \frac{4}{\Delta t^2} [M] + \frac{2}{\Delta t} [C] + [K] \quad (2-7)$$

and

$$(F') = \frac{4}{\Delta t^2} (r'_n) + 2 [M] (r''_n) + 2 [C] (r'_n) + (\Delta F) \quad (2-8)$$

Usually the damping matrix is expressed as a combination of mass and stiffness dependent matrices. This was done in the program as illustrated below.

$$[C] = \alpha [M] + \beta [K] \quad (2-9)$$

Substituting Eq. 2-9 into Eqs. 2-7 and 2-8 leads to the equilibrium equation for each time step.

$$[K^*] (\Delta r) = (F^*) \quad (2-10)$$

in which $[K^*]$ and (F^*) are the effective stiffness and force at the beginning of each time step, respectively.

$$[K^*] = \left(\frac{4}{\Delta t^2} + \frac{2\alpha}{\Delta t} \right) [M] + \left(\frac{2\beta}{\Delta t} + 1 \right) [K] \quad (2-11)$$

$$(F^*) = [M] \left[\frac{4}{\Delta t^2} (r'_n) + 2(r''_n) + 2\alpha (r'_n) \right] + 2\beta [K] (r'_n) + (\Delta F) \quad (2-12)$$

The effective stiffness $[K^*]$ and effective load vector (F^*) are assembled at the beginning of each time step based on Eqs. 2-11 and 2-12. The incremental displacement is obtained by solving Eq. 2-10. The initial velocity and acceleration for

the following time step are determined by from Eqs. 2-4 and 2-5. If nonlinearity develops within a time step, the tangent stiffness is updated at the beginning of the following time step, and unbalanced loads resulting from nonlinearity is added to the load vector and is used in the following time step.

The version of program DRAIN-2D developed as part of this investigation contains only those hysteretic models that are applicable to reinforced-concrete response, and hence is called DRAIN-RC (RC for reinforced concrete). It incorporates three components of inelastic deformations caused by flexure, shear and anchorage slip. Each element is modelled as an elastic line element with three springs at each end to account for inelasticity due to flexure, shear and anchorage slip (see Fig. 2.2). Modelling of these springs is described in the following sections.

2.3 Flexural Modelling

Inelastic behaviour in flexure is introduced by allowing the formation of plastic hinges at the ends of line elements. Therefore, each element is represented by an elastic beam with two nonlinear flexural springs, one at each end. These springs have large initial stiffnesses so that they do not rotate when the element is in the elastic region. Only the elastic beam element deforms within the elastic region. If the force level exceeds the prescribed yield level, then one or more of the springs become active and start rotating to introducing inelasticity.

The element stiffness formulation for flexure is straight forward, and is done in terms of end moments M_A and M_B shown in Fig. 2.3. For a generalized loading, the inflection point need not be located at the center of the beam. While the variation in the location of inflection point is accounted for in computing elastic deformations, the inflection point is assumed to remain at the center for the purpose of modeling post-yield deformations. This is done for simplicity.

Inelastic flexural behaviour is modeled to have two components; the elastic beam and two inelastic flexural springs at two ends. Fig. 2.4 shows moment-curvature relationship for an actual beam to be modeled. The beam is represented by an elastic beam element and flexural springs. When the applied moment M on the actual beam is greater than the yield moment M_y , the relationship between incremental chord-rotation and end moments is expressed as below:

$$\begin{bmatrix} \Delta\theta_A \\ \Delta\theta_B \end{bmatrix} = \begin{bmatrix} \frac{L}{3rEI} & -\frac{L}{6rEI} \\ -\frac{L}{6rEI} & \frac{L}{3rEI} \end{bmatrix} \begin{bmatrix} \Delta M_A \\ \Delta M_B \end{bmatrix} \quad (2-13)$$

where, “ r ” is the strain hardening ratio for the real beam. Incremental chord-rotation at end A can be obtained as:

$$\Delta\theta_A = \frac{L}{3rEI} \left(\Delta M_A - \frac{\Delta M_B}{2} \right) \quad (2-14)$$

A coefficient ξ is introduced as proportion of two end moments:

$$\xi = \frac{\Delta M_B}{\Delta M_A} \quad (2-15)$$

Substituting equation 2-15 into Eq. 2-14 gives

$$(\Delta\theta_A)_{actual} = \frac{L}{3rEI} \left(1 - \frac{\xi}{2} \right) \Delta M_A \quad (2-16)$$

Similarly for the elastic beam element;

$$(\Delta\theta_A)_{elastic} = \frac{L}{3EI} \left(1 - \frac{\xi}{2}\right) \Delta M_A \quad (2-17)$$

Hinge rotation can be obtained as:

$$(\Delta\theta_A)_{hinge} = \frac{\Delta M_A}{K_p} \quad (2-18)$$

in which K_p is the flexural spring stiffness beyond yielding, at end A. As it was illustrated in Fig. 2.4 incremental chord-rotation in actual beam consists of the following two components.

$$(\Delta\theta_A)_{actual} = (\Delta\theta_A)_{elastic} + (\Delta\theta_A)_{hinge} \quad (2-19)$$

Substituting equation 2-16, 2-17 and 2-18 into Eq. 2-19 and solving for K_p gives:

$$K_p = p K_{se} = \frac{3EI}{L \left(1 - \frac{\xi}{2}\right)} \left(\frac{r}{1-r}\right) \quad (2-20)$$

From Eq. 2-20, flexural spring strain hardening ratio, p , can be determined as:

$$p = \frac{3EI}{L \left(1 - \frac{\xi}{2}\right) K_{se}} \left(\frac{r}{1-r}\right) \quad (2-21)$$

For $\xi = 1.0$, the member deforms in double curvature with the inflection point at the

center of the member. Then, “p” for this case can be written as;

$$p = \frac{6EI}{L K_{se}} \left(\frac{r}{1-r} \right) \quad (2-22)$$

Eq. 2-22 is used as default in DRAIN-RC. However, the inflection point is not always in the middle of the beam, and the user needs to enter an “r” value such that when Eq. 2-22 in the program is employed the desired location of the point of inflection is attained. This “r” value is not the actual strain hardening value for the member, but rather a value when used in Eq. 2-22 results in the correct value of “p” for the desired point of inflection. If this r value to be entered is called r_i (input “r”), and the actual strain hardening ratio “r” is denoted by r_a , then an expression can be developed relating these two “r” values. This is illustrated below.

$$P_{actual} = \frac{3EI}{L \left(1 - \frac{\xi}{2}\right) K_{se}} \left(\frac{r_a}{1-r_a} \right) \quad (2-23)$$

Computed “p” by DRAIN-RC;

$$P_{computed} = \frac{6EI}{L K_{se}} \left(\frac{r_i}{1-r_i} \right) \quad (2-24)$$

Since the computed “p” should be equal to the actual “p”, Eqs. 2-23 and 2-24 are set equal to each other. This results in an expression for r_i in terms of r_a and ξ . Consequently, for known quantities of ξ (representing the point of inflection) and

r_a (representing the actual strain hardening ratio of the member that is being modeled), an input value r_i can be computed and entered as input data.

$$r_i = \frac{r_a}{2 \left(1 - \frac{\xi}{2}\right) (1 - r_a) + r_a} \quad (2-25)$$

If $\xi = 1.0$, the point of inflection is in the center and;

$$r_i = r_a \quad (2-26)$$

The flexural flexibility of a spring is computed on the basis of “p” which defines the post yield stiffness of the spring. Eq. 2-24 is employed to compute “p”. Total member flexibility is then established by superimposing elastic and inelastic flexibilities, including those due to shear and anchorage slip. The member stiffness is determined by inverting the flexibility matrix. The total stiffness of the whole structure is assembled using the direct stiffness method.

The hysteretic model for reinforced concrete elements in flexure used in DRAIN-2D was a version of Takeda’s model [48] which was modified by Powell [46]. The model consists of a bilinear primary curve with an initial stiffness and a subsequent strain hardening stiffness which are characteristic of monotonic loading condition. The unloading stiffness depends on the maximum hinge rotation, and is controlled by the input parameter α . Fig. 2.5 illustrates the modified Takeda model.

2.4 Modelling of Inelastic Shear

Shear modelling is done in much the same manner as flexural modelling discussed in

the previous section. A member is modelled for shear to have an elastic beam and two inelastic shear springs at two ends. Two different stages of loading are considered; post-cracking and post-yielding. The elastic beam element models elastic shear deformations, and contributes to post-elastic deformations. The springs rotate upon cracking and produce the remaining and significant portions of inelastic deformations within the post-cracking and post-yielding stages. Fig. 2.6 shows moment-shear distortion (chord angle caused by shear distortion) relationship for an actual beam and the components due to the elastic beam element and an inelastic shear spring. The relationship between incremental chord-rotations and end moments for an actual beam is shown below for $M > M_{cr}$:

$$\begin{bmatrix} \Delta\theta_A \\ \Delta\theta_B \end{bmatrix} = \begin{bmatrix} \frac{1}{s_1 GAL} & \frac{1}{s_1 GAL} \\ \frac{1}{s_1 GAL} & \frac{1}{s_1 GAL} \end{bmatrix} \begin{bmatrix} \Delta M_A \\ \Delta M_B \end{bmatrix} \quad (2-27)$$

in which s_1 is the ratio of post-cracking to pre-yielding stiffnesses of the primary moment-shear distortion curve. Incremental chord-rotation due to shear at end A is given below:

$$\Delta\theta_A = \frac{1}{s_1 GAL} (\Delta M_A + \Delta M_B) \quad (2-28)$$

Substituting coefficient ξ from Eq. 2-15 into Eq. 2-28 results;

$$(\Delta\theta_A)_{actual} = \frac{1}{s_1 GAL} (1 + \xi) \Delta M_A \quad (2-29)$$

Similarly for the elastic beam;

$$(\Delta\theta_A)_{elastic} = \frac{1}{GAL} (1+\xi) \Delta M_A \quad (2-30)$$

As in the case of flexural hinge, the rotation of shear spring can be determined as;

$$(\Delta\theta_A)_{hinge} = \frac{\Delta M_A}{K_{p_1}} \quad (2-31)$$

where, K_{p_1} is the inelastic shear spring stiffness beyond cracking but prior to yielding at end A. The incremental chord-rotation of the actual beam consists of two components as illustrated in Fig. 2.6.

$$(\Delta\theta_A)_{actual} = (\Delta\theta_A)_{elastic} + (\Delta\theta_A)_{hinge} \quad (2-32)$$

Substituting Eqs. 2-29, 2-30 and 2-31 into Eq. 2-32, and solving for K_{p_1} leads to;

$$K_{p_1} = p_1 K_{se} = \frac{GAL}{(1+\xi)} \left(\frac{s_1}{1-s_1} \right) \quad (2-33)$$

p_1 can then be determined from the above expression.

$$p_1 = \frac{GAL}{(1+\xi) K_{se}} \left(\frac{s_1}{1-s_1} \right) \quad (2-34)$$

If $\xi = 1.0$, which means the inflection point is in the middle of the beam, p_1 becomes;

$$p_1 = \frac{GAL}{2 K_{se}} \left(\frac{s_1}{1-s_1} \right) \quad (2-35)$$

Eq. 2-35 is used as default in DRAIN-RC to compute p_1 . However as it was mentioned earlier, the inflection point is not always located in the middle of the beam and the user must find s_1 so that Eq. 2-35 is applicable for a general case. If s_a and s_i are used to denote the actual and input ratios of post-cracking stiffness to elastic stiffness, respectively, the following expression can be derived to compute the input ratio s_i in terms of s_a and ξ . From Eq. 2-34;

$$(p_1)_{actual} = \frac{GAL}{(1+\xi) K_{se}} \left(\frac{s_a}{1-s_a} \right) \quad (2-36)$$

The computed value in DRAIN-RC;

$$(p_1)_{computed} = \frac{GAL}{2 K_{se}} \left(\frac{s_i}{1-s_i} \right) \quad (2-37)$$

Equating the actual and computed values of p_1 , s_i can be obtained as a function of s_a and ξ ;

$$s_i = \frac{2 s_a}{(1+\xi) (1-s_a) + 2 s_a} \quad (2-38)$$

For $\xi = 1.0$;

$$s_i = s_a \quad (2-39)$$

Eq. 2-38 is also applicable to the post-yield region. Once the pre-yield and post-yield flexibilities of each spring are obtained, they are added to the member flexibility matrix, from which the member stiffness matrix is obtained.

The hysteretic model for inelastic shear, developed by Ozcebe and Saatcioglu [49], consists of trilinear primary curve with break points at cracking and yielding, and a set of rules based on experimental data. The model incorporates the effects of axial compression, and simulates pinching of hysteresis loops. Fig. 2.7 depicts the features of the hysteretic model for shear.

2.5 Modelling Anchorage Slip

Inelastic deformations caused by anchorage slip are assigned to the third inelastic spring at each end of an element. Anchorage slip occurs at member ends near beam-column joints and column-foundation interfaces. These deformations develop as concentrated rotations at the ends of members, as rigid body rotations. Elastic and post-yield stiffnesses for anchorage slip can be obtained from a moment-anchorage slip rotation relationship, and can be input into program DRAIN-RC.

The slip hysteretic model implemented in DRAIN-RC was proposed by Saatcioglu, Alsiwat and Ozcebe [50,51]. It consists of a primary curve and a set of hysteretic rules as shown in Fig. 2.8. The primary curve is established from extension and slippage of reinforcement, computed at the end of the member. The strain condition used for this computation is obtained from sectional analysis. The model includes

hysteretic rules established on the basis of experimental data and previous numerical work [47].

2.6 Damping Matrix

Damping matrix, $[C]$, is express as a combination of mass, $[M]$ and stiffness $[K]$ matrices, as expressed in Eq. 2-9. This equation is repeated below, with coefficients α and β obtained from the orthogonality conditions for stiffness and mass matrices.

$$[C] = \alpha [M] + \beta [K] \quad (2-40)$$

$$\alpha = \frac{4\pi (T_i \lambda_i - T_j \lambda_j)}{T_i^2 - T_j^2} \quad (2-41)$$

and

$$\beta = \frac{T_i T_j (T_i \lambda_j - T_j \lambda_i)}{\pi (T_i^2 - T_j^2)} \quad (2-42)$$

in which T_i and T_j are vibration periods for modes i and j and λ_i and λ_j are damping ratios for modes i and j , respectively. The periods of vibration for two first modes of all buildings were computed by software SAP90, and by assuming the damping ratio to be 5%. Coefficients α and β were then computed using Eqs. 2-41 and 2-42, and used in nonlinear dynamic analyses carried out by DRAIN-RC.

2.7 Ductility Ratio

One of the indicators used to express inelastic deformability of members during seismic response is ductility ratio. The ductility ratio can be expressed in the form of displacement, rotation and curvature ductility ratio. It is computed as the ratio of maximum deformation to deformation at first yield. Rotational ductility ratio was used in this investigation as the ratio of maximum chord rotation to chord rotation at initial yield. This is given below, and illustrated in Fig. 2.9.

$$\mu_{\theta} = \frac{\theta_{\max}}{\theta_y} \quad (2-43)$$

The chord rotation at each end of a reinforced concrete member includes three components of inelastic action, consisting of flexure, shear and anchorage slip. Therefore, the ductility ratio can be expressed in terms of each deformation component as flexural ductility ratio, μ_{fl} , shear ductility ratio, μ_{sh} , and slip ductility ratio μ_{sl} . These ductility ratios are explained separately in the following sections.

2.7.1 Flexural Ductility Ratio

The flexural ductility ratio is computed by finding the ratio of maximum flexural rotation to flexural rotation at first yield, as indicated below:

$$\mu_{fl} = \frac{(\theta_{\max})_{fl}}{(\theta_y)_{fl}} \quad (2-44)$$

where,

$$(\theta_y)_{fl} = M_y \left(\frac{L}{6EI} + \frac{1}{K_{e_{fl}}} \right) \quad (2-45)$$

and

$$(\theta_{\max})_{fl} = (\theta_y)_{fl} + (M_{\max} - M_y) \left(\frac{L}{6EI} + \frac{1}{K_{p_{fl}}} \right) \quad (2-46)$$

in which $(K_e)_n$ and $(K_p)_n$ are pre-yield and post-yield stiffnesses of flexural spring, respectively. When axial force-bending moment interaction is considered, the yield rotation corresponds to initial yielding under the concurrent level of axial force and corresponding moment.

2.7.2 Slip Ductility Ratio

The ductility ratio for anchorage slip is computed by finding the ratio of maximum and yield values, as indicated below:

$$\mu_{sl} = \frac{(\theta_{\max})_{sl}}{(\theta_y)_{sl}} \quad (2-47)$$

where,

$$(\theta_y)_{sl} = \frac{M_y}{K_{e_{sl}}} \quad (2-48)$$

and

$$(\theta_{\max})_{sl} = (\theta_y)_{sl} + \frac{(M_{\max} - M_y)}{K_{p_{sl}}} \quad (2-49)$$

in which $(K_e)_{sl}$ and $(K_p)_{sl}$ are pre-yield and post-yield stiffnesses of slip spring, respectively.

2.7.3 Shear Ductility Ratio

The ductility ratio for shear is computed similar to the previous two ductility components, as indicated below:

$$\mu_{sh} = \frac{(\theta_{\max})_{sh}}{(\theta_y)_{sh}} \quad (2-50)$$

where,

$$(\theta_y)_{sh} = M_{cr} \left(\frac{2}{GAL} + \frac{1}{K_{e_{sh}}} \right) + (M_y - M_{cr}) \left(\frac{2}{GAL} + \frac{1}{K_{p1_{sh}}} \right) \quad (2-51)$$

and

$$(\theta_{\max})_{sh} = (\theta_y)_{sh} + (M_{\max} - M_y) \left(\frac{2}{GAL} + \frac{1}{K_{p2_{sh}}} \right) \quad (2-52)$$

in which $(K_e)_{sh}$ is the elastic stiffness, $(K_{p1})_{sh}$ and $(K_{p2})_{sh}$ are pre-yield and post-yield stiffnesses of shear spring, respectively.

2.7.4 Combined Ductility Ratio

A ductility ratio is also computed to reflect combined inelasticity in the member. This ductility ratio consists of all the inelastic components of deformation components superimposed in the general definition of ductility, given as:

$$\mu_{com} = \frac{[(\theta_{max})_{fl} + (\theta_{max})_{sl} + (\theta_{max})_{sh}]}{[(\theta_y)_{fl} + (\theta_y)_{sl} + (\theta_y)_{sh}]} \quad (2-53)$$

where, the yield rotation used in the above expression is the summation of all deformation components at the first yield of any one component. For example, if the flexural spring yields first, then this stage of loading is marked as the initial yield, and rotations due to other components are computed at this stage of loading and added to the yield rotation due to flexure.

DRAIN-RC computes all the above ductility ratios. When the inelastic spring of a deformation component is not included in an analysis, the corresponding rotations are set equal to zero, and do not have any contribution to combined ductility ratio.

2.8 Program Modifications

The original program DRAIN-2D, developed at the University of California, Berkeley [10] was modified by Alsiwat and Saatcioglu at the University of Ottawa [47]. The modifications include incorporation of two new hysteretic models for reinforced-concrete elements, i.e.; inelastic shear and anchorage slip models. Although these

models improved the program significantly, additional modifications were felt necessary to further improve the accuracy of analysis results. These improvements were introduced as part of the current investigation. As a result, a new version of DRAIN-2D, referred to as DRAIN-RC, was created with improved modelling capabilities for application to reinforced concrete structures. These modification can be categorized under four major topics, as indicated below:

- Modification of the flexural hysteretic model to incorporate M-P interaction effects during response.
- Implementation of a new hysteretic model for infill panels.
- Implementation of an option for inelastic “Push-over” analysis.
- Modifications to incorporate P- Δ effects during response.

2.8.1 Axial Force-Moment Interaction Model

The original hysteretic model for reinforced concrete elements in flexure used in DRAIN-2D was a modified version of Takeda model [46,48]. The model with degrading stiffness characteristics was developed for members under constant level of axial force. However, during response to earthquakes, vertical members (columns and walls) generally undergo changes in the amount of axial force. The effect of P-M interaction on strength and stiffness of members during response was modelled by Saatcioglu et. al [15,52,53]. This model was selected for implementation into DRAIN-RC. Fig. 2.10 illustrates a typical P-M interaction surface under reversed loading. The effects on continuously changing yield levels and post-yield stiffnesses are illustrated in Fig. 2.11. Up to balanced loading, the higher the axial compression on member is the higher the yield level becomes. This trend reverses itself above the balance point. It was assumed in the model that the stiffness change due to P-M interaction effects within the elastic region was negligible. However, the yield moment was effected very significantly. Therefore, the yield level was updated continuously

at each time step based on the level of axial force computed during the previous step. The new yield moment is compared at each time step against the imposed action to decide whether yielded has occurred or not. The axial load due to gravity is used for the very first time step of analysis. In post-yield regions, the stiffness of flexural spring changes according to the concurrent level of axial force.

The post-yield branches of curves corresponding to different levels of axial load are assumed to have the same slope. The procedure for considering P-M interaction during initial loading is illustrated in Fig. 2.11. At the beginning of time step i , the rotation of a flexural spring is θ_A , moment is M_A and the stiffness is K_i . Hence at the end of i^{th} time step, the rotation and moment will be θ_B and M_B , respectively. For $(i+1)^{\text{th}}$ time step, the level of axial force changes from P_i to P_{i+1} . Assuming $P_{i+1} > P_i$, the yield moment becomes higher than that for time step i . This increase in yield level translates into an equivalent increase in flexural stiffness. The new stiffness for time step $i+1$, labelled as K_{i+1} , is computed as expressed below and illustrated in Fig. 2.11.

$$K_{i+1} = \text{slope}_{AC} = \frac{M_B - M_A}{(\theta_{equ})_B - \theta_A} \quad (2-54)$$

The spring rotation is computed based on the new stiffness K_{i+1} from point B onward. The path is parallel to line AC until it reaches point D with rotation θ_D and moment M_D , as illustrated in Fig. 2.11. If loading continues during the following time step, the procedure remains the same for $(i+2)^{\text{th}}$ time step. The slope of unloading branch is proportional to the elastic stiffness.

2.8.2 Hysteretic Model for Infill Panels

Another modification that was introduced to DRAIN-RC was the implementation of

an infill panel element. Since the original DRAIN-2D had 10 different types of elements, this new element was numbered as element # 11. The model was developed by R.E. Klingner and V.V. Bertero and was verified extensively by the researchers [20,21]. The infill panel is replaced by two equivalent diagonal struts for the purpose of modelling, one in each direction. The hysteretic behavior of a single strut is illustrated in Fig. 2.12, and the rules are explained below.

1. Elastic loading (path OB or OA, code 0 or 4). This path is defined by;

$$S = \frac{EA}{L}v \quad (2-55)$$

where; S is the axial force in the strut, E is Young's modulus for the infill material, v is the axial deformation in strut with positive values corresponding to extension, L is the length of strut, and A is the area of strut as defined by the product of panel thickness and effective strut width.

2-Strength envelope curve (path AC, code 5). This curve is defined by;

$$S = A f_c (e^{\gamma v}) \quad (2-56)$$

where; A, S and v are as defined above, and f_c is the compressive strength of infill material as determined from prism tests. The strength degradation parameter, γ , is selected based on experience.

3- Elastic unloading (path CD, code 6). The strut is assumed to unload

following a stiffness equal to the elastic loading stiffness of path OA.

4- Tension curve (path DD'E or DD'E', code 6 and 1). Initially, an equivalent diagonal strut is defined with some tensile resistance due to the tensile strength of the panel material (usually very low). The idealized tension curve is defined by;

$$S = A f_t \quad (2-57)$$

where, S and A are as defined previously and f_t is constant nominal resistance in tension. Since the panel tensile resistance is not observed to have any significant effect on the behavior of experimental models, f_t is therefore assigned a zero value in analysis.

5- Elastic unloading (path EF or E F', code 7 or 2). The elastic unloading in tension zone is defined to be identical to the initial elastic stiffness (path OB or OA). Two different codes are assigned to specify the path clearly. Path E F' (code 2) has axial deformation (v) higher than that for path EF (code 7).

6- Reloading in tension zone (path F'O or FO , code 3). Reloading is characterized by zero stiffness and strength until the equivalent strut deformation returns to zero.

7-Reloading in compression zone (path OG or OG',code 8). Reloading is characterized by reduced stiffness and strength compared to the virgin elastic behavior of path OA. For a given strut, reloading curve is a straight line connecting point O with a point on the strength envelope, corresponding to

the maximum (absolute value) deformation - positive or negative- previously experienced by the same strut. Fig. 2.12 shows this behavior. If v_C is greater than v_E , the path becomes OC and the reloading curve connects point O to point C (maximum previous deformation). However, if v_E is greater than v_C , the path becomes OG (v_E is equal to v_G).

2.8.3 Static Inelastic Analysis (Push-Over)

The original DRAIN-2D has the capability of performing elastic static load analysis. The modification introduced to DRAIN-RC enables the program to perform incremental inelastic static load analysis (push-over analysis). In this analysis, the nodal loads can be applied incrementally in several load steps. The global stiffness matrix is updated for each load step, and if yielding is encountered unbalanced forces are computed and added to the load vector in the following load step. Some nodal loads and distributed member loads, like gravity loads, can be applied on the structure prior to the incremental analysis at their full magnitudes, while others can be specified as incremental loads, monotonically increasing to perform the push-over analysis. In this case, the structure is analysed under full static loads in the first load step, followed by the incremental load, effects of which are superimposed to those of the full static load analysis. The lateral load for push-over analysis can be specified either in equal increments or in an arbitrarily increasing manner as defined by the user.

2.8.4 P-Δ Effect

Multistory buildings subjected to lateral deformations are subjected to P-Δ effects. The P-Δ effect generates secondary moments and affects strength and deformability of vertical load carrying members. In structural analysis a convenient way of accounting for the P-Δ effect is to reduce the stiffness of the system such that the

increased displacement associated with the P-Δ moment is obtained [71]. The geometric stiffness can be established for each type of element. The original program DRAIN-2D considers the P-Δ effect by considering the geometric stiffness. However, this version of the program has two major inconsistencies. First, the program uses geometric stiffness properties of a truss element for beam and beam-column elements [10]. Second, the geometric stiffness is incorporated only at the beginning of dynamic analysis (the first time step) and is not calculated and updated during the rest of the analysis. Furthermore, the original DRAIN-2D does not include the P-Δ effect for static analysis. Since this option is particularly important for push-over analysis, it is desirable to introduce this effect to static load analysis, including the push-over analysis. The geometric stiffness matrix for a truss element is shown below.

$$K_G = \frac{N}{L} \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -1 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (2-58)$$

where, L is the length of an element and N is the axial load on the truss element. The above geometric stiffness matrix is used in the original DRAIN-2D for beam-column elements. However, for a beam-column element, the geometric stiffness matrix differs from that of a truss element and can be expressed as shown below [55,56]:

$$K_G = \frac{N}{30L} \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 36 & 3L & 0 & -36 & 3L \\ 0 & 3L & 4L^2 & 0 & -3L & -L^2 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -36 & -3L & 0 & 36 & -3L \\ 0 & 3L & -L^2 & 0 & -3L & 4L^2 \end{bmatrix} \quad (2-59)$$

where, L and N are as defined before. In the modification introduced to DRAIN-RC, K_G is computed for each element as indicated in Eq. 2-59, and subtracted from the flexural stiffness matrix at the beginning of analysis to introduce the effects of geometric stiffness. If the axial force changes in an element, the geometric stiffness is calculated at each time step during dynamic analysis, and at each load step for push-over analysis. This procedure was verified to give accurate results in the elastic range. However, if yielding occurs in an element, a new procedure must be followed as explained in the following paragraphs.

The global stiffness matrix must be computed in the beginning of each analysis. In the original DRAIN-2D the flexibility matrix is first determined and then inverted to obtain the stiffness matrix for each element. The element stiffness matrices are assembled to establish the global stiffness matrix of the whole structure. If the P-Δ effect is to be introduced, the geometric stiffness matrix has to be included in the global stiffness matrix. The element flexibility matrix consists of two different components; elastic flexural flexibility, and geometric flexibility. The new member flexibility matrix, including these two components, has been implemented into DRAIN-RC. The elastic flexural stiffness matrix of a beam element can be obtained from Fig. 2.13, as indicated below.

$$K_e = \frac{E I}{L} \begin{bmatrix} K_{ii} & K_{ij} \\ K_{ij} & K_{jj} \end{bmatrix} \quad (2-60)$$

where, I is the moment of inertia, L is the length of element, and E is the Young's modulus. K_{ii} , K_{jj} and K_{ij} are coefficients which depend on the variation of cross sectional dimensions. For a prismatic element with $K_{ii} = K_{jj} = 4$ and $K_{ij} = 2$, the geometric stiffness matrix is illustrated below.

$$K_G = \frac{N}{30 L} \begin{bmatrix} 4L^2 & -L^2 \\ -L^2 & 4L^2 \end{bmatrix} \quad (2-61)$$

where, N is the axial force (positive if tension) and L is the length of the element. N can be expressed as a function of E , I and L for the purpose of combining the two matrices, as shown below.

$$N = \frac{R E I}{L^2} \quad (2-62)$$

where, R is a coefficient. This leads to;

$$K_t = K_e + K_G = \frac{E I}{L} \begin{bmatrix} K_{ii} + \frac{2 R}{15} & K_{ij} - \frac{R}{30} \\ K_{ij} - \frac{R}{30} & K_{jj} + \frac{2 R}{15} \end{bmatrix} \quad (2-63)$$

If the element is subjected to compression, R becomes negative and K_G is subtracted from K_c . The elastic flexibility matrix of a beam-column element, which also reflects the P- Δ effect, can then be found from Eq. 2-63 as indicated below.

$$F_t = K_t^{-1} = S \begin{bmatrix} 30K_{jj}+4R & -30K_{ij}+R \\ -30K_{ij}+R & 30K_{ii}+4R \end{bmatrix} \quad (2-64)$$

and

$$S = \frac{30 L}{EI[15R^2 + (120K_{ii} + 120K_{jj} + 60K_{IJ})R + 900(K_{ii}K_{jj} - K_{ij}^2)]} \quad (2-65)$$

In the above expressions, if the axial load does not exist, R becomes zero, and the flexibility matrix F_t for a uniform element becomes;

$$F_t = \frac{L}{12EI} \begin{bmatrix} 4 & -2 \\ -2 & 4 \end{bmatrix} \quad (2-66)$$

The total flexibility matrix, including the effects of flexure, anchorage slip and shear, as well as the P- Δ effect, becomes:

$$F_t = \begin{bmatrix} S(30K_{jj}+4R) + \frac{1}{GAL} + (f_{fl} + f_{sl} + f_{sh})_i & S(-30K_{ij}+R) \\ S(-30K_{ij}+R) & S(30K_{ii}+4R) + \frac{1}{GAL} + (f_{fl} + f_{sl} + f_{sh})_j \end{bmatrix} \quad (2-67)$$

Yielding of a member is handled accurately by using F_t .

2.9 Verification of DRAIN-RC

The program DRAIN-RC has been verified against two well-known programs; DRAIN-2DX [30] and SAP90 [57]. Both of these programs have P- Δ considerations. Examples 1 to 4 adopted from verification manual of SAP90 contain a series of columns subjected to end moments and forces as shown in Fig. 2.14. These examples have been analysed with SAP90 and DRAIN-RC and results are presented in Tables 2.1 to 2.4, respectively. As can be seen from these results, there is very good agreement between the computer results and theoretical solutions.

Two additional sample problems were also considered for verification. Example 5 is a column subjected to axial force N and a lateral force V . The properties of the column is shown in Fig. 2.15. If the P- Δ effect is not considered, the lateral deflection of B is given by:

$$\delta_B = \frac{VL^3}{3EI} = \frac{10 \times 5^3}{3 \times 22690} = .01836_m = 18.36_{mm} \quad (2-68)$$

The same analysis results have been obtained by all three programs, as expected. When the P- Δ effect was considered, lateral deflection of the free end of column is expected to develop higher displacements. The results of three analyses are summarized in Table 2.5. As can be seen, the results show excellent agreement between DRAIN-RC and SAP90. It is obvious that the results of DRAIN-2DX are not valid since the shear equilibrium equation is not satisfied. It should be mentioned that the yield level M_Y was assumed to be high enough so that the column remained elastic since program SAP90 could not handle nonlinearity.

Example 6 includes a two bay, two storey frame, as shown in Fig. 2.16. The results of analyses are compared in Table 2.6. The comparison indicates that DRAIN-RC can be used reliably to account for P-Δ effects in structures.

Additional verification was done using the structure in Example 5. This time the column was analysed to find its critical axial load, N_{cr} . The exact solution for a cantilever column is given below:

$$(N_{cr})_{theoretical} = \frac{\Pi^2}{4} \frac{EI}{L^2} = 2.4674011 \frac{EI}{L^2} \quad (2-69)$$

The following results were obtained by DRAIN-RC and DRAIN-2DX:

$$(N_{cr})_{DRAIN-RC} = 2.4859617 \frac{EI}{L^2} \quad (2-70)$$

$$(N_{cr})_{DRAIN-2DX} = 3.0 \frac{EI}{L^2} \quad (2-71)$$

The results indicate excellent agreement between DRAIN-RC and the theoretical solution. To verify the procedure explained in section 2.8.4, Example 5 was further analysed using DRAIN-RC. This time yielding was forced by specifying the yield moment M_Y to be lower than M_A . Because SAP90 can not handle inelasticity, the analysis was performed in two different steps. In the first step, called elastic, a portion of lateral load V was applied so that M_A reached approximately $M_Y = 100$ kN-m. In the second step, called inelastic, the rest of the lateral load V was applied. In this step, however, the end restraint was changed from a fully fixed end to a hinge end with a

rotational spring. The stiffness of this spring was calculated on the basis of Equation 2-20. Fig. 2.17 shows the details of these two steps. The results are summarized in Table 2.7, indicating good correlation between DRAIN-RC and SAP90.

Chapter 3

Selection of Structures and Earthquake Records

3.1 Introduction

One of the main objectives of the current study is to establish drift demands for reinforced concrete structures. This is done by conducting dynamic inelastic response history analysis, as well as inelastic push-over analysis. The first step in such analyses involves selection of realistic structures, representative of design practice. Two types of reinforced concrete structural systems were selected. The first type consisted of moment resisting frames. Moment resisting frames with and without infill masonry walls were selected. The second type consisted of a frame shear-wall interactive system. The structures were designed following the current practice. This chapter describes the properties of reference buildings and the ground input motions intended for dynamic analysis.

3.2 Selection of Reference Structures

Two different types of structural systems, with three different heights were selected for analysis. These include 5-storey, 10-storey and 15-storey frame and frame shear-wall buildings. This results in 6 buildings for a given location. Two locations were selected to represent the two significantly different seismic regions of Canada. These were; Vancouver in the west and Ottawa, in the east. Hence a total of 12 buildings were designed to be analysed. Furthermore, the 10-storey frame building in the west

was analysed either as a bare frame structure, without any non-structural component, or with masonry infill walls. When the infill walls were considered, two different wall thickness, as 100 mm and 200 mm, and three wall widths, covering one, two or all three bays of exterior frames, were used. These different combinations of wall thicknesses and widths resulted in 6 different cases with different floor-to-wall-area ratios and initial fundamental periods. Furthermore, two different grades of masonry were used, as NW with minimum compressive strength of f_m equal 8.6 MPa. and MW with minimum compressive strength f_m equal 15.2 MPa. As a result, a total of 24 buildings with different framing systems, lateral bracing mechanisms, heights and locations were selected for dynamic analysis. The plan views of these buildings are shown in Figs. 3.1 and 3.2. A symmetrical floor plan was selected to minimize the effect of torsion. Elevation views of frames with and without infilled panels, as well as frame-shear wall buildings with different heights are shown in Figs.3.3 to 3.5. The load combinations considered were based on the current National Building Code of Canada (NBCC-1995) [58]. The design data are presented in Table 3-1.

Equivalent static design loads were calculated based on NBCC-1995 [58]. Accordingly, the elastic design base shear, V_e , was calculated from Eq. 3-1.

$$V_e = v S I F W \quad (3-1)$$

where;

- v** : Zonal velocity ratio for the given location.
- S** : Seismic response factor based on the fundamental period of the structure.
- F** : Foundation factor, reflecting soil conditions.
- I** : Importance factor.
- W** : Weight of structure.

Because structures are often designed for certain level of inelasticity, the elastic design base shear is reduced, as permitted by inelastic deformability of the structure, and used as design base shear. The reduction is introduced through “R” factor. The resultant force is modified by coefficient “U” to account for past experience and expected levels of performance.

$$V = \left(\frac{V_e}{R} \right) U \quad (3-2)$$

where, $U = 0.6$. The fundamental period, T , for design purposes was computed based on code specified empirical expressions that account for the presence of non-structural elements. These expressions are shown below:

For concrete frames;

$$T = 0.075 (h_n)^{3/4} \quad (3-3)$$

For shear wall buildings;

$$T = 0.09 h_n / (D_s)^{1/2} \quad (3-4)$$

The building code allows a more accurate determination of the fundamental period provided that the resulting base shear is not less than 80% of the value determined based on the above equations. More accurate estimation of fundamental periods of

structures were obtained by solving the eigen value problem using computer software SAP90. The eigen solution included the effects of potential cracking in members, as well as the lateral bracing provided by infill panels. Diagonal compression struts were used for the latter case, as described later in this chapter.

Building designs were conducted using computer software SAP90. Two-dimensional static analyses were carried out first to determine critical values of axial forces, shear forces, and bending moments at each joint, using preliminary member dimensions. This information was then used to design columns and beams. The presence of non-structural elements, like unreinforced masonry infill panels, were not considered in static analyses. The analyses were repeated with revised member sizes when necessary. The results of this procedure for the ten-story frame and frame-shear-wall buildings for eastern and western Canada are summarized in Tables 3-2 to 3-5.

3.3 Modelling Structures for Inelastic Analysis

There are three stages of modelling that an analyst has to go through to perform static and dynamic inelastic analyses. The first stage involves modelling the entire structure, which consists of individual elements. The second stage involves member modelling. In DRAIN-RC, each member is modelled as a linear line element and three inelastic springs for potential inelasticity at each end, as described in Chapter 2. The inelastic springs simulate the nonlinear portion of deformations by following hysteretic models assigned to them. These include hysteretic models for flexure, shear, anchorage slip; and consist of idealized force-deformation relationships. The analyst has to idealize (or model) these relationships and derive relevant strength and stiffness parameters as input data. This forms the third stage in modelling.

Analysis of structures for earthquake effects is often conducted in two orthogonal

directions, separately and independently. Therefore, a planar analysis is often utilized with computer software developed for plane frame analysis. A similar approach is followed in the current research program. Computer software DRAIN-RC is intended for planar analyses of buildings, either under inelastic dynamic conditions (response history analysis) or inelastic static conditions (push-over analysis). This requires modelling of three-dimensional building structures as series of two dimensional frames linked together to reflect strength and stiffness in each direction.

The buildings considered for analyses consist of four interior and two exterior frames in the short direction. They can be modelled by connecting each frame in series with rigid links so that they deflect the same amount in the plane considered. Implicit in this approach is the assumption of rigid slab diaphragms. The links are modelled in the computer program through rigid truss elements with high axial rigidities. These elements transfer lateral forces without moment connections between the frames. Although the modelling of full structure in each direction can be done as described, it is often not necessary to repeat identical frames in the model. Lumping similar frames into a single frame reduces the number of elements and the required analysis time, and may be necessary for a research project where extensive analysis of structures are to be conducted. Therefore, the model described earlier was condensed into two moment-resisting frames connected by rigid links, where one of the frames represented strength and stiffness of all four interior frames and the other represented the same for the two exterior frames. This is shown in Fig. 3.6 for a 10-storey building. The elements of the interior lumped frame has four times the stiffnesses of beams and columns of a single interior frame. Similarly, the exterior lumped frame has double the stiffness of an actual exterior frame. The stiffnesses mentioned include flexural, anchorage slip and shear stiffnesses. The strength of each member of the lumped frames is also increased proportionately to have the same cracking and

yielding characteristics as those for individual frames. The infill panels are modelled as diagonal ties and struts between the framing members.

Once the structure is modelled as described in the preceding section, consisting of discrete elements, each frame element is then modelled as an elastic line element and inelastic springs. The properties of infill elements require evaluation of effective strut widths. Strength and stiffness properties of each element are then specified. While the elastic stiffness parameters for the elastic line element are readily available and can be specified without any difficulty, the properties of hysteretic models assigned to inelastic springs require further modelling, this time in the form of idealization of force-deformation relationships under monotonic loading. Each hysteretic model consists of a primary curve under monotonically increasing load and a set of built in rules to simulate strength and stiffness characteristics during unloading and reloading branches of cyclic loading. The properties of the primary force-deformation relationships need to be evaluated based on idealized relationships.

The flexural primary curve can be established by conducting a sectional moment-curvature analysis. Computer software, COLA [59], developed at the University of Ottawa, was used for this purpose. The same program also provides the primary moment-rotation relationship for deformations caused by anchorage slip.

3.3.1 Moment-Curvature Relationship

The sectional moment-curvature relationship was determined for each frame member through sectional analysis. The initial linear portion of the relationship, prior to cracking, was determined by beam theory and by considering a transformed section. Strain compatibility analysis was employed for post cracking regions with the usual assumption of plane sections before bending remaining plane after bending. Internal

strain diagram was established for a selected value of extreme fibre strain and an assumed location of neutral axis. The corresponding stress diagram and internal forces were then computed, and equilibrium was checked. The assumption for the neutral axis location was revised if necessary until the internal equilibrium of forces was satisfied. Once sufficient number of strain conditions were considered, the complete moment-curvature relationship was plotted. Although this relationship showed a smooth curve with distinct points for significant changes in slope at cracking and yielding points, it was idealized as a bi-linear relationship where the initial line segment represented the elastic branch and the second line segment represented the post-yield region. This idealization was necessary to be consistent with the flexural hysteretic model incorporated in DRAIN-RC. Fig. 3.7 illustrates computed and idealized moment-curvature relationships for a sample member. From the idealized curve one can find effective flexural rigidity (EI) and strain hardening ratio r (post-yield stiffness ratio) as well as yielding moment (M_Y) to be specified as input for DRAIN-RC. For columns, the effective flexural stiffness (EI) is evaluated as 70% of gross flexural stiffness which is consistent with the reduced value of flexural stiffness recommended by CSA A23.3 Standard for Design of Concrete Structures [60].

The location of inflection point in members is another input item for the purpose of modelling inelastic springs. As it was discussed in Chapter 2, this is done by specifying the ratio of bending moment at far end to that at near end (ξ). The ξ ratio was computed based on the load combination that included earthquake loading dead load and 50% of live load. Substituting ξ and strain hardening ratio “ r ” for the actual member, into Eq. 2-21, the strain hardening ratio “ p ” for flexural spring was determined. This procedure was done for both ends of each member in preparing the input data file for DRAIN-RC.

3.3.2 Moment -Anchorage Slip Rotation Relationship

A similar procedure was followed to model force-deformation characteristics of members due to anchorage slip. A sectional analysis was performed to find the properties of anchorage slip spring used to model inelastic deformations due to extension and/or slippage of reinforcement in the adjoining member. The moment-anchorage slip rotation relationship at member end can be idealized as in the case of moment-curvature relationship. Fig. 3.8 shows actual and idealized curves for a beam, from which one can obtain the elastic stiffness and yield moment for the slip spring, as well as the strain hardening ratio. It should be mention that the stiffness of the slip spring is independent of the location of inflection point.

3.3.3 Moment-Shear Distortion Relationship

Determination of shear response of reinforced concrete members is not an easy task. Therefore, most researchers rely on experimentally established empirical expressions. A similar approach has been adopted here to establish the primary shear force-shear deformation relationship. Ozcebe, G. and Balci, H. [61] evaluated a large volume of experimental data which was obtained from full scale tests of beams and columns, and suggested an analytical procedure to establish a relationship between moment and shear deformations. Accordingly, moment-shear distortion relationship can be obtained, knowing the ultimate shear strength of a member and the initial tangent stiffness. This is illustrated in Fig.3.9. The intersection of the horizontal line drawn at ultimate shear level and the initial tangent stiffness, K_{s0} , determines Δ_{rs} . For an arbitrary value of shear displacement, Δ_s , one can find K_s based on the following expression, defining a point on the curve.

$$\frac{K_s}{K_{so}} = \frac{1.0}{1.0 + .85 \left(\frac{\Delta_s}{\Delta_{rs}} \right)} \quad (3-5)$$

Once the relationship of shear force and shear displacement is obtained, one can establish the moment-shear distortion relationship which can then be idealized by a tri-linear curve from zero to cracking moment, post cracking to yielding and post yielding to ultimate. The properties of these branches are input into DRAIN-RC for the purpose of modelling the hysteretic relationship for shear.

3.3.4 Modelling Effective Widths of Panel Struts

Various approximate methods have been proposed by previous researchers to determine the lateral stiffness of wall elements. A simple and highly developed method was developed by Stafford-Smith [62] based on the concept of equivalent diagonal struts. Accordingly, the geometric properties of diagonal struts are defined as a function of the contact lengths between the wall and the columns, α_h , and between the wall and the beams, α_L . This is illustrated in Fig. 3.10. Holmes [63] recommended that the diagonal strut width should be equal to one-third of the diagonal length of the panel. New Zealand Code [64] specifies a width equal to one quarter of the wall length. The following expressions were developed, based on the beams on elastic foundation theory, for α_h and α_L [65]:

$$\alpha_h = \frac{\pi}{2} \left[\frac{4 E_f I_c h}{E_m t \sin 2\theta} \right]^{.25} \quad (3-6)$$

and

$$\alpha_L = \pi \left[\frac{4 E_f I_b L}{E_m t \sin 2\theta} \right]^{.25} \quad (3-7)$$

where; E_m and E_f are elastic moduli of masonry and frame material, respectively; and t , h and L are thickness, height and length of infill wall, respectively. The angle is defined as $\theta = \tan^{-1} (h/L)$. If a triangular stress distribution is assumed along the strut width w , as illustrated in Fig. 3.10, the effective strut width for uniform stress distribution can be obtained as shown below:

$$w = \frac{1}{2} \sqrt{\alpha_h^2 + \alpha_L^2} \quad (3-8)$$

The modulus of elasticity of masonry, E_m , can be obtained from the following expression as traditionally done [66,67]:

$$E_m = k f_m \quad (3-9)$$

where f_m is the compressive strength of masonry, and k is a coefficient equal to 750 for concrete block, and 500 for clay brick.

3.4 Selection of Earthquake Records

It is essential to use realistic ground motions to establish realistic drift demands, which forms the main objective of the current investigation. An inventory of earthquake

records was selected for this purpose, representing two distinctly different seismic regions of Canada; the east and the west. The earthquake records consisted of actual ground motions recorded during past earthquakes and artificially generated records based on seismological history and geological make up of the region. The actual records of significance in eastern Canada are limited in numbers. Two such records were selected for this region, consisting of 1988 Saguenay, Quebec Earthquake and 1982 Miramichi, New Brunswick Earthquake. The artificial records were generated by Atkinson and Beresnev [68] for 10% probability of exceedance in 50 years to match with the Uniform Hazard Spectra developed for firm ground sites by the Geological Survey of Canada (Adams et al. [69]). More than one earthquake record was needed to match the spectra. While a moderate nearby earthquake could be critical for short period structures, it could not produce sufficient long-period energy to be critical in the long period range. Similarly, a high magnitude distant earthquake may be critical for the long period structures, while it loses too much high-frequency energy, due to attenuation, to be critical in the short period range. It was assumed that representative earthquake motions can be generated for eastern Canadian sites using an event of Magnitude 5.5 for short period structures, and an event of Magnitude 7.0 for long period structures. These events were assumed to occur at certain distances for cities under consideration. In this study, Ottawa was selected as a location in Eastern Canada. Accordingly, the Magnitude 5.5 event was assumed to occur at a hypocentral distance of 30 km, and the Magnitude 7.0 event was assumed to occur at 150 km. The records obtained were then scaled by a “fine tuning factor” to match with the uniform hazard spectra for the probability level considered. A total of four artificial records were considered for the east, two for short period structures and two for long period structures, with two different distances from the hypocenter.

There was no record available for western Canada. Therefore, previous earthquake

motions recorded in western U.S.A. were adopted for dynamic analysis, although it may be argued that the seismological activity of western U.S.A. may not be the same as that of western Canada. A total of seven records were considered, consisting of 1940 El-Centro Earthquake, 1952 Taft Record, 1971 San Fernando Earthquake, and four different records of the 1994 Northridge Earthquake. In addition, four artificial records generated by Atkinson and Beresnev [68] were considered for short and long period structures for 10% probability of exceedance in 50 years. The records were based on magnitude 6.0 event for short-period hazard, and magnitude 7.2 event for long-period hazard. These records were generated following the same approach as that for eastern Canada. Table 3-6 provides a list of all the records.

The artificial records were verified by computing spectral values and comparing them with the Uniform Hazard Spectra developed by the Geological Survey Canada [70]. A single-degree-of-freedom system was selected, consisting of an elastic cantilever column, fixed at the base with a lumped mass at the free end. By changing the value of the mass, a range of fundamental periods between 0.2 and 5.0 seconds was obtained. Response spectra was calculated for a city in the east and another city in the west using computer software SAP90. The results are shown in Figs. 3.11 and 3.12 for Vancouver and Ottawa. The response spectra are then compared with the uniform hazard response spectra (UHS) generated by the Geological Survey Canada in Fig. 3.13 and show good agreement.

Chapter 4

Dynamic Inelastic Analysis

4.1 Introduction

One of the major stages of the current research program involved investigation of modelling and ground motion parameters on structural response, prior to studying drift demands on different types of concrete buildings. This was necessary to identify and select parameters that play significant roles on structural response in the subsequent phase of the investigation. It would also provide insight into the significance of the modelling features considered for dynamic inelastic analysis.

The parametric investigation was conducted in two stages. Stage one consisted of ground motion characteristics. In this stage an inventory of previously recorded and artificially generated earthquake records were used for the seismological regions considered. The results helped in identifying critical ground motions for the structures and locations selected. Stage two consisted of structural parameters that addressed modelling features. The information generated in this stage helped identify the modelling features that had to be considered for improved accuracy of analysis.

The parametric investigation was conducted by analysing the buildings designed in Chapter 3 for eastern and western Canada. All the parameters, except the one that

was being investigated, were kept constant during the investigation.

4.2 Ground Motion Parameters

Two parameters were considered to identify the characteristics of ground motion. These were; the frequency content as reflected in different earthquake records, and intensity as reflected in the magnitude of ground accelerations.

4.2.1 Frequency Content

The earthquake records selected for the parametric study consisted of actual ground motions recorded during past earthquakes and artificially generated records based on seismological history and geological make up of the region. These earthquake records are described in Chapter 3.

The previously recorded earthquake motions were first normalized with respect to the peak ground acceleration to eliminate intensity as a parameter. This was done by scaling all the records for western Canada to 23% g and those for eastern Canada to 20% g. These levels of g correspond to the probability of exceedance of 10% in 50 years as per the National Building Code of Canada (NBCC 1995[58]). The artificial records were used as they were, without any scaling factor.

The 10-storey moment resisting frame structure, designed for western Canada (Vancouver), was analysed first under 7 actual earthquake records and 4 artificially generated records. The properties of the structure are given in Table 3.2. Maximum drifts obtained from each analysis along the height of the structure are plotted in Figs. 4.1 to 4.3. The results indicate that, among the artificial records, Long Event No. 1 generated the highest drift demand, showing approximately 0.65% drift at the fourth

floor level. Under actual ground records, the Northridge Record No. 1 showed the highest drift demand and developed approximately 1.8% drift at the fourth floor level. While this record showed highest drift demand in the negative direction, it was not critical in the positive direction. The 1940 El-Centro record, with approximately 1.5% drift demands in both positive and negative directions was the second most critical record.

The 10-storey frame-shear wall structure was analysed next under the same eleven records. The properties of the structure are summarised in Table 3.3. Maximum drift demands obtained are plotted in Figs 4.4 to 4.6. Artificial Short Event No.2, and Northridge Record No.1 produced 0.47% and 0.60% drifts, respectively, as highest drift demands, which occurred at the roof level. The 1940 El-Centro Record was also as critical as the Northridge Record, showing approximately the same maximum drift demand at the roof level. Comparisons of drift demands of the two 10-storey reinforced concrete structures indicate that the drift demand for the structure with shear walls was lower than that for the moment resisting frame structure, as expected. However, the displacement distribution along the height was different, with the frame structure showing the highest drift demands at approximately 1/3 the height from foundation, whereas, the shear wall structure showing the highest drift demand at the top of the building.

The same analyses were repeated for frame and frame-shear wall structures designed for eastern Canada (Ottawa). The properties of structures are given in Tables 3.4 and 3.5. The structures were subjected to two actual records and four artificial records. Maximum drift demands for frames with and without shear walls in eastern Canada are presented in Figs. 4.7 to 4.10. The results indicate that, among the artificial records both Long Events No.1 and 2 generated the highest drift demands, showing

approximately 0.11% lateral drift for the frame building. Long Event No.2 was critical for the frame-shear wall building, with 0.12% drift at the roof level. Among the actual earthquake records, Saguenay earthquake required highest drift demands with 0.09% and 0.06% maximum drifts for the frame building and frame-shear wall building, respectively. Comparisons of drift demands of these two types of structures reveal little difference in maximum response, although the fundamental periods of structures were significantly different. The results also indicate that the structures in the west showed approximately 3 to 6 times the displacement demands revealed for the corresponding structures in the east.

4.2.2 Earthquake Intensity

The intensity of earthquake record, as represented by peak ground acceleration, was investigated as the second ground motion parameter. The actual recorded earthquake motions used in the preceding section were scaled to have peak ground accelerations for a specific probability of occurrence, i.e., 10% in 50 years. The artificially generated records were also derived for the same probability of occurrence. However, earthquakes of different levels of probability of occurrences with different intensities do occur rather randomly. Therefore, the significance of ground motion intensity on lateral drift demand was investigated. The earthquake records that were found to be critical in terms of frequency characteristics were selected for this purpose. The intensity of each record was increased to investigate the effects of intensity. The 1940 El Centro and the 1988 Saguenay records were intensified by 50% and 100%. The artificial records were intensified to match with the peak ground accelerations used for the actual records. This resulted in an increase of 170%, 300% and 440% for the record in the west; and 127%, 240%, and 450% for the record in the east. The 10-storey frame and 10-storey frame-shear wall buildings designed for eastern and western Canada were analysed under intensified earthquake records. The results were

compared to get insight into the effect of intensity of ground motion.

Figures 4.11 to 4.14 show maximum drift, inter-storey drift, and maximum ductility demands for beams and columns of the 10-storey frame building in eastern Canada when subjected to the critical artificial record (Long Event No.2) and the Saguenay Earthquake. In all cases the response remained elastic except for the first storey column. When the artificial record was intensified to have 40% g as the peak acceleration, limited inelasticity was developed with a ductility ratio of 1.25. The lateral drift increased approximately linearly with maximum ground acceleration.

Drift and ductility response of the frame-shear wall building is shown in Figs. 4.15 to 4.18. The results indicate a similar trend as for the frame building. The building remained elastic under the Saguenay Earthquake even when the intensity was doubled relative to the intensity corresponding to 10% probability of exceedance in 50 years. Some yielding was observed when the intensity of the artificial record was magnified to give peak ground acceleration of 40% g. In this case the second storey columns and first storey walls yielded, showing ductility ratios of approximately 2.0. The beams showed limited yielding, only near the top of the building. The drift demand increased approximately linearly with the increase in earthquake intensity.

The records for western Canada were more intense. These records were further intensified and used to investigate the significance of intensity. The results provided valuable information on the variation of deformation demands with intensity when significant inelasticity was experienced. Drift and ductility response of frame and frame-shear wall structures are illustrated in Figs. 4.19 to 4.26. Although the distributions of lateral drift, inter-storey drift and ductility demands along the height of the structure show the same trend as those observed previously for structures in

the east, deformation demands did not show linear variation with earthquake intensity. This is attributed to significantly higher levels of inelasticity developed in these structures. While the ductility ratio in vertical elements was limited to approximately 2.0, it ranged between 10.0 and 15.0 in the beams.

For example when moment resisting frame subjected to El-Centro ground motion with 23% and 34.5%g intensity, maximum drift increases from 1.5% to 3% (see Fig.4.20) or maximum beam ductility ratio at roof level of frame with shear wall becomes four times and increases from 2 to 8 when the intensity has just become twice (see Fig. 4.26). It should be noted than when the intensity of ground motion is 46%g moment resisting frame can not sustain and it collapsed therefor in Figs. 4.19 to 4.22 only two cases are illustrated.

4.3 Effects of Structural Parameters

The significance of selected structural parameters, consisting mostly of the modeling features available in computer software DRAIN-RC, was studied prior to generating data for drift demands. These parameters are discussed in the following sections with examples of structural response obtained for each case.

4.3.1 P-M Interaction

It was mentioned in Chapter 2 that building response could be influenced significantly by axial forces that change continuously during response. Therefore, a hysteretic model incorporating P-M interaction effects was implemented in DRAIN-RC. While this effect was shown to be very significant in coupled wall structures [15,52,53], due to significant variation in coupling axial forces in walls, the same may not be true for the frame and frame shear wall structures considered in this investigation. If this were the case, then the P-M interaction feature of the software need not be considered in

analyses of these types of structures, generating drift data in Chapter 5.

Both 10-storey frame and 10-storey frame-shear wall buildings were analyzed twice, with and without the P-M interaction effects. P-M interaction diagrams for the columns were established first using computer software COLA [59]. These relationships were specified as input data into DRAIN-RC. Initial moment capacities under static gravity loads were also computed and updated continuously during response based on the concurrent level of axial force. The earthquake records, that were found to be critical in the earlier part of the parametric investigation, were used as input motions. The results for the two types of structural systems considered are shown in Figs. 4.27 to 4.34 for western and eastern Canada. The comparisons with and without P-M interaction effects indicate that there was no sizeable effect of this parameter on overall building response, as shown by lateral drift. This conclusion can easily be justified since the variation in axial force during response results from shear forces generated in the attached beams and these beams, with long shear spans, do not attract high shear forces. In fact, the results showed that the maximum axial force in columns is limited to 20% of column concentric capacity. Another reason for the insignificant effect of axial force is the limited inelasticity observed in vertical members where axial forces develop. The effect of axial force is primarily on the yield strength of member. If there was no yielding, or limited yielding, the P-M interactive effects tend to be very small. Since the structures were designed following the “strong-column weak-beam” concept of the current building code, inelasticity in columns was observed to be very limited. Additional analyses were conducted under increased ground motion intensity to obtain increased inelasticity. This was done by scaling the ground motion by a factor of 1.5. Furthermore, the yield moment of all beams were increased by a factor of 2.0 to make them attract higher shear forces, which would generate higher axial forces in columns. The ductility ratios obtained are

shown in Figs. 4.35 and 4.36. They show increased inelasticity, as intended. When the earthquake intensity was increased the column and beam ductility ratios increased to 2.0 and 14.0, respectively. When the beam yield level was increased, the column ductility ratio increased up to approximately 9.0 due to the increase in column axial tension. It was clear that the P-M interaction effect was very significant on ductility demands. However, when the overall structural response was plotted in terms of lateral drift and ductility demands, there was no significant effect. This is illustrated in Figs. 4.37 to 4.42.

4.3.2 Anchorage Slip

Research on inelastic behaviour of reinforced concrete elements show that anchorage slip, derived from the slippage and/or extension of longitudinal reinforcement in the adjoining member, can play an important role on building response [47,50,51]. Hysteretic behaviour of inelastic deformations due to anchorage slip was modeled in DRAIN-RC, as explained in Chapter 2.

Ten-storey frame and frame-shear wall structures in the east and west were analyzed with and without the anchorage slip effect. The structures were subjected to the same critical ground motions as before. Figs. 4.43 to 4.50 show comparison of maximum drift and inter-story drift response. The results show some differences in drift response caused by softening in structures due to anchorage slip. The change in structural stiffness translated into a change in frequencies of structures, and hence differences in response. Allowing for anchorage slip rotation did not necessarily produce increased deformations in all cases. The structures analyzed did not develop high inelasticity which would magnify deformations due to yield penetration and resulting anchorage slip. Therefore, further analyses were conducted under magnified earthquake intensity. Increased yielding in these cases are evident in Fig. 4.51, which

shown beam and column ductility demands. The drift response is illustrated in Fig. 4.52, and indicates a higher drift demand when the anchorage slip was allowed to occur in analysis.

4.3.3 Inelastic Shear Deformation

In multi-storey buildings, structural members often behave predominantly in the flexure mode. Therefore, in computing deformations, shear effects are conveniently ignored. However, deep structural members, including structural walls, may develop significant shear deformations. Although consideration of shear within the elastic range can easily be done, evaluation of inelastic shear deformations under reversed cyclic loading may become a challenging task. Computer program DRAIN-RC does include a hysteretic model for inelastic shear deformations. This program was used in the parametric investigation, with and without inelastic shear feature, to study the significance of inelastic shear deformations on building response.

Ten-storey frame and frame-shear wall structures were analyzed under eastern and western Canadian seismic conditions. The buildings were modeled such that premature shear failure prior to flexural yielding was prevented, since they were designed based on the current design requirements. In the analyses, shear yielding was triggered whenever there was flexural yielding detected. This was a phenomenon observed experimentally [49] even if the design shear capacity was higher than that corresponding to flexural yielding. Analysis results are plotted in Figs. 4.53 to 4.60 in terms of maximum drift and maximum inter-storey drift. The frame structure showed little or no effect of inelastic shear action. However, the shear wall structure did show some effect, especially in the west where the ground motion intensity led to significant yielding. The results are further evaluated in Table 4.1, where ratios between maximum drifts with and without the inelastic shear effect, are summarized. Accordingly, inelastic shear effects in the shear wall structure produced 18% increase

in lateral drift and 16% in inter-storey drift.

Additional analyses were conducted to acquire a better insight into the effect of inelastic shear on building response. This time shear yielding was permitted prior to flexural yielding. The yield level in shear was reduced to 75% and 50% of the value corresponding to flexural yielding. The results are presented in Figs. 4.61 and 4.62 for the shear wall building subjected to the western artificial record. Table 4.2 also provides a summary of drift response. The results indicate that inelastic shear effects can be very significant in shear wall buildings, especially if the shear strength is lower than that corresponding to flexural strength. For the structure and the ground motion record considered in the analysis, inelastic shear contributed 53% to total lateral drift, and 61% to inter-storey drift.

4.3.4 P- Δ Effect

Structures subjected to earthquakes may experience significant lateral drift. In tall buildings and buildings undergoing significant inelasticity, the lateral drift can be very significant, leading to secondary deformations and stresses (P- Δ effect). Neglecting this effect can give erroneous results and may mislead structural engineers. DRAIN-RC has the capability of accounting for the secondary stresses due to P- Δ effect, as explained in Chapter 2. The program was used to investigate the significance of this parameter on seismic response.

Ten-storey buildings were analyzed with and without P- Δ effect under eastern and western seismic conditions. The buildings were subjected to critical ground motions for the two regions. Maximum drift and inter-storey drift response for each building are illustrated in Figs. 4.63 to 4.70. The results are also summarized in Table 4.3. They indicate a significant increase in drift demand due to the P- Δ effect. The effect

becomes magnified with increased inelasticity and lateral deflection. While the shear wall structures showed limited P- Δ effect due to lateral stiffness and reduced deflection, the frame structures exhibited increased lateral drift due to the P- Δ effect. This point was further investigated under increased intensity of ground motions, which would aggravate the P- Δ effect. The ten-storey moment resisting frame was re-analyzed under Long Event No.2 and El-Centro 1940 record, with peak accelerations scaled up to 46% g and 34.5% g, respectively. These records resulted in increased yielding and hence high ductility demands as illustrated in Fig.4.71. The drift response obtained is shown in Fig. 4.72 and indicates an increase of as much as 35% in drift response and 40% in inter-storey drift. The building analyzed under the intensified artificial record developed excessive lateral drift and collapsed when the P- Δ effect was considered, whereas the same building survived the earthquake with approximately 5% maximum drift when the secondary deformations due to P- Δ were not considered (see Fig. 4.73). The analyses provide sufficient data to conclude that P- Δ effects play a crucial role on building response and that they can not be ignored in response history analysis for earthquake effects.

4.3.5 Effects of Structural Stiffness, Mass and Period

The frequency characteristics of a building interact with those of the exciting force. Therefore, the fundamental period of a building plays a major role on dynamic response. The period is a function of structural mass and stiffness. It is conceivable that the variation in these properties may not have significant effects on drift response, so long as the fundamental period remains constant. If it is shown through dynamic analysis that the period alone is sufficient to represent the mass and stiffness, then the results for building response can be generalized on the basis of fundamental period.

The 10-storey frame structure designed for western Canada was considered for a

series of analyses with different stiffnesses and mass. Figure 4.74 shows drift response of the structure with 4 different values of flexural rigidity (EI) and corresponding changes in structural mass so that the fundamental period remained the same. The results indicate that the response changed in each case significantly. Furthermore, there was no pattern observed among building responses. Further analyses were conducted for structures with different flexural rigidities but constant mass. The results shown in Fig. 4.75 indicate that maximum drift decreased with increasing rigidity. A similar set of analyses was conducted using four structures with four different mass, but the same flexural rigidity. In these structures, while a different response was obtained in each case, there was no pattern among structural responses as shown in Fig. 4.76. The analyses conducted in this part of the parametric investigation indicate that both the stiffness and mass need to be considered separately, in describing structural characteristics of a building, and that their effects can not be lumped in the fundamental period.

4.3.6 Effect of Flexural Strength

The effect of strength on response was analyzed by considering the same ten-storey moment resisting frame designed for western Canada, by changing the flexural yield level. The original building was analyzed four more times with four different levels of flexural yield moment for beams. The four cases consisted of $0.50 M_y$, $0.75 M_y$, $1.25 M_y$, and $1.50 M_y$ assigned to the beams. The results are plotted in Fig. 4.77. The structure with the lowest beam strength, i.e., with $0.50 M_y$ could not sustain the artificial record, and collapsed due to the P- Δ effect.

Chapter 5

Drift Demands

5.1 Introduction

One of the main objectives of the current study was to establish seismic drift demands for reinforced concrete structures. Several steps were taken to achieve this objective. First a powerful software was developed for nonlinear response history analysis of reinforced concrete structures by modifying and enhancing the capabilities of an existing software. Six frame and six frame-shear wall structures were designed for two different regions of Canada, reflecting eastern and western Canadian seismic conditions, while covering three different building heights and two different structural systems. Twelve additional buildings were considered by including masonry infill panels in the 10-storey frame building. These consisted of three different wall widths filling one-bay, two-bays, and three bays of exterior frames, two different wall thicknesses consisting of 100 mm and 200 mm, and two different masonry grades with 8.6 MPa and 15.2 MPa compressive strengths. Some of these structures were used to conduct a parametric, study with the computer software developed, to identify the significance of modeling features and to determine the critical ground motions for each structure in each region. A comprehensive investigation of seismic drift demands was then conducted through response history analysis. The details of these analyses

and resulting drift demands are presented in this chapter.

5.2 Ground Motion Records Used

Drift demands of structures vary substantially with characteristics of ground motions. Effects of frequency content and intensity of earthquake records were investigated earlier, and are discussed in Chapter 4. These parameters reflect seismological and soil conditions of the region, as well as the probability of occurrence of the ground motion. It is extremely difficult to predict the characteristics of future earthquakes. Seismologists often consider the history of earthquake activity in a region and the geological formation of each region to employ a statistical approach in establishing main features of seismic motions. Artificial earthquake records may be generated for each region, capturing the salient features of probable earthquake records.

Two different regions with two sets of earthquake records were considered in this investigation. The city of Ottawa was selected to represent eastern Canada, while the city of Vancouver was selected to represent western Canada. Previously recorded actual earthquake records, as well as artificially generated records were used in each region. The probability level considered was consistent with that adopted by the National Building Code of Canada (NBCC-95[58]) for designing earthquake resistant buildings. This corresponded to the probability of occurrence of 10 % in 50 years. The current building code uses peak ground acceleration (PGA) as a measure of intensity. Accordingly, 20 % of the gravitational acceleration (g) is used as PGA for Ottawa and 23% g is used for Vancouver, in computing total design base shears. The relationship used in the current NBCC between PGA and probability level was established based on the studies of the Geological Survey Canada [72,73,74]. This relationship was used to scale previously recorded earthquake motions that were selected for this investigation so that they would represent the same probability level.

Two of the seven records for western Canada, and one of the two records for eastern Canada were found to be critical for drift response earlier in the parametric study. These three records were scaled to give the same probability level as that suggested in NBCC-95 [58] before they were used in generating drift data. This produced the first set of data.

Recently, the Geological Survey Canada (GSC) generated uniform hazard spectra (UHS) for use in year 2001 NBCC with 10 % probability of occurrence in 50 years [70]. The UHS was obtained on the basis of increased seismic data that has become available in recent years and new knowledge attained on seismological characteristics of Canada. Artificial records were generated by Atkinson and Beresnev [68] to match the UHS. These records were used in the current investigation to represent the expected seismic activity in Ottawa and Vancouver, corresponding to UHS. This formed the second set of data.

Although the UHS was based on the same probability level that was used in NBCC-95 [58], the resulting design base shear was not necessarily the same. A third set of analyses was conducted using the artificial records generated to match UHS after scaling them to yield the same base shear as that specified in NBCC-95[58]. The scaling was done through a trial and error process in which each record was scaled until the spectral acceleration corresponding to the fundamental period of the building under consideration resulted in the same base shear as that specified in NBCC-95 [58]. The scaling factors are summarized in Tables 5.1 to 5.4. Fig. 5.1 shows comparisons of NBCC-95 design response spectra, UHS, and the spectra for artificial records. Figs. 5.2 and 5.3 give sample response spectra for the scaled artificial records used for the most stiff (5-storey frame-shear wall) and the most flexible (15-storey frame) structures considered in this investigation, respectively.

In summary, a total of three sets of analyses were conducted under; i) critical earthquake records scaled to NBCC-95 PGA, ii) artificial records that match UHS, and iii) artificial records scaled to give the same design base shears as those obtained using NBCC-95 [58].

5.3 Computation of Drift Demands

Two different types of concrete structures were analyzed to compute drift response. These included frame structures and frame-shear wall interactive systems. The frame structures were analyzed with and without masonry infill panels. The masonry infills had two different masonry grades and 6 different combinations of wall thicknesses and wall widths. Three different building heights were considered as 5-storey, 10-storey, and 15-storey. This resulted in a total of 24 buildings with different structural systems, lateral bracing characteristics, and different seismic regions. Each building was subjected to 5 to 10 different earthquake records, depending on structural period and seismic zone. The three different types of earthquake record described in the previous section were employed as ground motions. Computer software DRAIN-RC was used to conduct the analyses. Structural properties determined on the basis of building designs performed in Chapter 3 were used in each case. Among the hysteretic models available in the computer software, flexure, shear, and anchorage slip models were used since these features were shown in Chapter 4 to have potentially significant effects on building response. The hysteretic model for infill panels was used when applicable. However, P-M interaction effects during response were found to be negligibly small in computing drift response, for the structures considered in this investigation. Therefore, this feature was not included in the analyses.

5.3.1 Western Canada

5.3.1.1 Artificial Records

A set of artificial records were proposed by Atkinson and Beresnev [68] to match the Uniform Hazard Spectra generated by the Geological Survey Canada [70], with a 10% probability of occurrence in 50 years. The set for each region includes two records for structures with long periods and two others for short periods. The records proposed for Vancouver were used first. Figs. 5.4 to 5.6 show the maximum lateral drift and inter-storey drift, as well as the maximum ductility ratios along the height of 5, 10, and 15 storey frame structures with fundamental periods of 1.73 sec., 3.70 sec., and 5.24 sec., respectively. The structures were designed based on NBCC-95 [58] without any consideration of possible contributions coming from non-structural masonry panels. Therefore, these structures have somewhat longer periods than those expected in practice for buildings of similar height and structural system. The results indicate that the maximum lateral drift is limited to approximately 0.7 % in all cases. The maximum inter-storey drift is approximately 0.8 % in 5-storey and 10-storey buildings, and approaches 1.2 % in 15-storey building. In all cases the columns remained elastic during response with maximum moments ranging approximately between 50 % and 75% of yield moment. The beams, however, experienced inelasticity with maximum ductility ratios ranging between 2.0 and 5.5.

Masonry infill panels were provided in the 10-storey frame structure to have more realistic representations of frame structures in practice. Different panel-to-floor area ratios and masonry strengths were considered. This was done by providing masonry walls of either 100 mm or 200 mm thickness, covering one, two or all three bays of exterior frames. The compressive strength of masonry was either of grade NW with a minimum compressive strength $f_m = 8.6$ MPa, or grade MW with $f_m = 15.2$ MPa. Fig. 5.7 shows drift and ductility response of a frame structure with single bays in

both exterior frames filled with 100 mm thick grade NW masonry wall. This resulted in a wall-to-floor area ratio of 0.21% and a fundamental period of 2.39 sec., reduced from 3.70 sec. for the bare frame structure. The results indicate that the maximum lateral drift in this case reduced to approximately 0.5 % and the maximum inter-storey drift was reduced to approximately 0.55 %. The columns remained elastic, with maximum moment remaining below 60% of the yield level. The maximum beam ductility ratio was reduced to 1.7, showing almost elastic response. The infill walls remained elastic, with maximum compression in diagonal struts approximately equal to 80 % of their capacity. When the infill panel thickness was increased to 200 mm, the wall-to-floor area ratio was increased to 0.42 % and the fundamental period showed a slight reduction to 2.28 sec. However, the response was not affected appreciably as indicated in Fig. 5.8. The maximum drift and maximum inter-storey drift remained below 0.45% and 0.5%, respectively. When the infill panel thickness was increased from 100 mm to 200 mm, the maximum diagonal strut force in masonry infill walls decreased from 80% to 53% of capacity. Therefore, there was no crushing of masonry walls in either case.

The 10-storey frame structure was next analyzed with two bays of exterior frames filled with masonry walls of grade NW. When a wall thickness of 100 mm was used, the wall-to-floor area ratio was 0.42 % and the fundamental period was reduced to 1.75 sec. Because of the reduced period, the building could potentially be critical under the artificial records generated for short-period structures. Therefore, all four artificial records were used to analyze the structure. The response, illustrated in Fig. 5.9, shows approximately the same drift demands as those for the previous case where walls with twice the thickness were used in a single bay, with the same wall-to-floor area ratio. However, the governing earthquake record was different. The maximum lateral drift and inter-storey drift were limited to 0.46% and 0.5%, respectively. The

masonry walls remained elastic, although one of the first-storey walls approached crushing, with a diagonal compressive strut force equal to 97 % of its capacity. The same structure was subsequently analyzed with masonry walls twice the thickness as the previous structure. The wall-to-floor area ratio was increased to 0.83% and the period was reduced to 1.62 sec. Drift and ductility responses, depicted in Fig. 5.10, indicate somewhat reduced lateral drift with a maximum value of 0.36 %. The maximum inter-storey drift for the same structure was 0.43%. The columns remained elastic and the beams experienced limited inelasticity, with a maximum ductility ratio of 1.6. The masonry walls remained elastic with maximum compression strut force equal to 67% of its capacity.

Another set of analyses was conducted using the same 10-storey frame structure with all three exterior bays filled with masonry walls of either 100 mm or 200 mm thickness. The buildings had wall-to-floor area ratios of 0.62% and 1.24%, respectively, with fundamental periods reduced to 1.41 sec. and 1.27 sec., respectively. The results are shown in Figs. 5.11 and 5.12, and indicate a consistent reduction in drift values with increased lateral stiffness. Accordingly, the maximum drift was 0.3% and 0.23% when the wall-to-floor area ratio was 0.62% and 1.24%, respectively. The reinforced concrete frame members remained elastic in both cases with beams and columns developing maximum moments of approximately 75% and 40% of their respective yield moments. The masonry walls remained elastic, with maximum compressive strut forces equal to 71% and 41% of their capacity for 100 mm and 200 mm walls, respectively.

The drift response of 10-storey frame building was investigated in terms the degree of lateral bracing provided by infill panels. Fig. 5.13 shows maximum lateral drift and inter-storey drift as a function of fundamental period reflecting overall structural

stiffness. The results indicate a consistent increase in drift demands with increasing period. This relationship is approximately linear, as can be seen in Fig. 5.13. The results further indicate that the masonry walls remained elastic in all of these buildings under the artificial records. This was true even in the case of single bay walls with as little as 100 mm thickness of the lowest grade masonry considered. This observation suggests that the contribution of masonry walls to lateral bracing of reinforced concrete frame walls could be quite substantial and should not be underestimated.

Additional analyses were conducted using the same infill panels as before, except with stronger masonry. Increasing masonry strength would directly translate into increased strength of the wall diagonal struts. It will also increase the rigidity of walls, since the increase in strength would also result in an increase in elastic modulus. Although the former effect would not be realized in these structures, since the elastic strength level in the previous set of walls with a lower strength was never exceeded, the latter effect would have some contribution to structural response. The same six buildings with either 100 mm or 200 mm wall thickness, providing lateral bracing in one, two or three bays, were analyzed using grade MW masonry with $f_m = 15.2$ MPa. The results are presented in Figs. 5.14 through 5.19. They indicate that drift demands are reduced in buildings with stronger (stiffer) masonry walls, approximately in proportion to the reduction in the fundamental period. Fig. 5.20 illustrates the variation of drift and storey drift demands with the fundamental period of structures. In all cases masonry walls remained elastic and the ratio of maximum strut force remained below 62% of its crushing capacity. Table 5.5 summarizes maximum strut forces recorded in each structure, expressed in percentage of strut capacity.

Three frame-shear wall buildings with different building heights were analyzed next. These buildings did not contain any masonry walls, and the lateral bracing was

provided by reinforced concrete walls. Figs. 5.21 to 5.23 include maximum lateral drift and inter-storey drift responses along the height of 5, 10, and 15-storey buildings with fundamental periods of 0.59 sec., 1.77 sec., and 3.38 sec., respectively. The results indicate that the maximum lateral drift is equal to 0.2 %, 0.47 % and 0.33 % in 5, 10 and 15-storey buildings, respectively. The maximum inter-storey drift is equal to 0.23 %, 0.55 % and 0.47 % in 5, 10, and 15-storey shear wall buildings, respectively. The beams of the frames remained essentially elastic, with minor yielding at the roof level, while the columns remained elastic, with maximum moments ranging between 50 % and 85 % of yield moment. The walls of 5 and 10-storey buildings yielded at the base, developing maximum ductility ratios of 2.1 and 3.3, respectively, whereas, the walls of 15-storey building remained elastic with maximum moment almost equal to the yield value. The maximum ductility ratios are plotted in Figs. 5.24 to 5.26.

5.3.1.2 Previously Recorded Actual Earthquakes

The same 18 frame buildings analyzed under artificial earthquake records were also analyzed under the previously recorded actual ground motion records. The two earthquake records, previously shown to be critical in Chapter 4, were selected for this purpose. These were the 1940 El Centro and 1994 Northridge Record No. 1. It should be mentioned that these two records were found to be critical for the 10-storey frame and shear wall frame buildings, earlier in the parametric study, reported in Chapter 4. Additional analyses were conducted to find the critical ground motion for 5 and 15-storey buildings with and without shear wall. Figs. 5.27 and 5.28 show the drift response of these buildings under 7 earthquake records that had been selected as the inventory of earthquake records. The comparison shows that the same two records that were critical for the 10-storey buildings, i.e., the 1940 El Centro and 1994 Northridge Record No. 1, were also critical for these buildings.

The earthquake records were scaled to give PGA of 23 % g, which was the intensity associated with 10 % probability of occurrence in 50 years, based on NBCC-95 Building Code [58]. Figs. 5.29 to 5.31 show the maximum lateral drift and inter-storey drift, as well as the maximum ductility ratio along the height of 5, 10, and 15 storey frame structures. The results indicate that the maximum lateral drift demand ranges between approximately 1.0 % and 1.8 %, whereas the maximum inter-storey drift ranges between 1.6 % and 2.5 %. These drift demands are approximately a factor of 2 higher than those indicated by artificial records. In all cases the columns developed limited inelasticity either at the base or where there was a strength taper introduced along the height, with ductility ratios below 2.0. The beam ductility ratio ranged between 5.0 and 12.0, indicating significant yielding.

Masonry infill panels were provided in the 10-storey frame structure to have more realistic representations of frame structures in practice. This was done by providing masonry walls of either 100 mm or 200 mm thickness, covering one, two or all three bays of exterior frames. The compressive strength of masonry was either of grade NW with a minimum compressive strength f_m equal to 8.6 MPa, or grade MW with f_m equal 15.2 MPa. These buildings were exactly the same as those analyzed previously under the artificial earthquake records. Figs. 5.32 through 5.43 show drift and ductility responses of frames laterally braced with infill panels. The results indicate a significant increase in drift demands, relative to those obtained under the artificial earthquake records. The maximum lateral drift demand increased to approximately 0.88 %, 0.67 % and 0.47 % for buildings with infill walls in one, two and three bays of exterior frames, respectively. Similarly, the inter-storey drift demands increased by approximately 70 % relative to those indicated by the artificial records, showing maximum values of 1.1 %, 0.8 % and 0.5 % storey drift for buildings with infill walls in one, two and three bays of exterior frames, respectively. Although the columns

remained elastic, the beams developed some inelasticity, ranging between elastic response when all three bays of exterior frames were braced with panels, to a ductility ratio of approximately 4.6 when only single bays were braced with walls. The majority of masonry walls remained elastic, although some in lower stories showed post elastic behaviour. The structure braced with two masonry walls, having a minimum compressive strength of $f_m = 8.6$ MPa and a thickness of 100 mm, developed inelasticity in the walls, experiencing post yield deformations in compression struts. Three masonry panels in the lowest three stories of this building were all subjected to post-yield compressive deformations in both directions, while three others deformed inelastically in one direction. This is illustrated in Fig. 5.44, where the 2nd floor time history of inter-storey displacement is shown with progressive inelasticity in masonry walls indicated. The progression of hysteretic relationship for a diagonal strut in the second storey interior panel is depicted in Fig. 5.45, illustrating changes in response within selected time increments. All the buildings with 100 mm Grade NW masonry walls experienced inelasticity in diagonal struts. The building braced with single bay masonry walls of higher strength (Grade MW) also developed inelasticity in panels when the wall thickness was 100 mm. Table 5-5 summarizes maximum strut forces recorded in masonry walls and patterns of inelasticity when applicable. Figs. 5.46 and 5.47 illustrate the variation of drift demands with fundamental period of frame structures, having different degrees of lateral bracing by masonry walls. The results show approximately linear relationship, irrespective of the inelasticity observed in some of the masonry panels.

The three frame-shear wall buildings were also analyzed using the previously recorded critical records, i.e. 1940 El Centro and 1994 Northridge No. 1 records. These records were found to be critical for the 10-storey frame shear wall building earlier in the parametric study. The 5-storey and 15-storey frame shear wall buildings were

analyzed under the records in the inventory, and were found to be also critical when subjected to the same two records, as shown in Figs. 5.48 and 5.49. Figs. 5.50 to 5.52 show the maximum lateral drift and maximum storey drift along the height of 5, 10, and 15 storey frame-shear wall structures. The results indicate that the maximum lateral drift vary between 0.33 % for 5-storey building and 1.0 % for 15-storey building. These values are approximately twice the values computed using the artificial records. The maximum storey drift is approximately 0.38 % in 5-storey, 0.72% in 10-storey , and 1.3 % in 15-storey frame-shear wall building. These are substantially higher than those found using the artificial records. The beams remained elastic in the 5-storey building but experienced inelasticity in others, especially in the 15-storey building, experiencing a maximum ductility ratio of 3.4. The columns in 5-storey building remained elastic, while they experienced inelasticity in the others two buildings, with maximum ductility ratio approaching 2.2. The walls developed some inelasticity at the base, with maximum ductility ratio equal to 5.5. Maximum beam, column and wall ductility ratios for the three frame-shear buildings are plotted in Figs. 5.53 to 5.55.

5.3.1.3 Scaled Artificial Records to Produce NBCC-95 Base Shears

The same 18 buildings analyzed previously were subjected to the same artificial records as before, except this time the records were scaled to produce the same base shear as that specified by NBCC-95 [58]. The results for the frame building are presented in Figs. 5.56 to 5.70 for drift and ductility responses. When there was no masonry bracing elements, the 5, 10 and 15-storey buildings developed maximum drifts of 0.75 %, 1.4 %, and 1.5 %, respectively. The storey drifts demands are higher, showing 0.9 %, 1.8 %, and 3.1 % for 5-storey, 10-storey, and 15-storey buildings, respectively. These drift demands are higher than those computed earlier under the artificial records without scaling. The columns remained elastic, with maximum

moments approaching 80 % of the yield moment, except at the 10th floor level in 15-storey building where there was a strength taper and the ductility ratio was 1.6. The beams experienced yielding, showing ductility demands of 3 to 5.5, except in 15-storey building, where the ductility demand at the 10th floor level was as high as 14.

Significant reductions occurred in drift and ductility demands when the 10-storey building was laterally braced by masonry walls. The maximum drift demand ranged between 0.17 % to 0.64 %; and the storey drift between 0.19 % to 0.77 %. The masonry remained elastic in all cases, with maximum strut forces as summarized in Table 5.5. All columns and some beams remained elastic, while the remaining beams developed some inelasticity with a maximum ductility ratio of 3.2. Figs. 5.71 and 5.72 illustrate the variation of drift demands with fundamental period of frame structures having different degrees of lateral bracing.

Figs. 5.73 to 5.75 show maximum drift response for frame-shear wall structures under scaled artificial records. The results indicate that the maximum lateral drift are 0.13%, 0.39%, and 0.53% for 5-storey, 10-storey and 15-storey buildings. The corresponding maximum storey drifts are 0.16%, 0.47% and 0.81%, respectively. The beams remained elastic in 5-storey building but experienced limited inelasticity in the other two buildings, with a maximum ductility ratio of 2.0 occurring in the 15-storey building. The columns showed a similar behaviour, remaining essentially elastic in all cases except for the 15-storey building where the maximum ductility ratio was limited to 1.2. The walls experienced some inelasticity at the base with a maximum ductility ratio of 3.2. The ductility response for these structures is plotted in Figs. 5.76 to 5.78.

5.3.1.4 Conclusions on Drift Demands in the West

Maximum drift demands for all 18 structures designed for Western Canada were re-evaluated in an attempt to generalize some of the earlier observations made. Figs. 5.79 through 5.81 show the drift demands obtained from analyses of structures under the three different sets of earthquake records considered. The results were plotted in terms of maximum drift and inter-storey drift as a function of the fundamental period of structures. Although the data for certain cases is scarce, both drift quantities show approximately linear variation with fundamental period for a given building height. Drift demands increase with increasing fundamental period, irrespective of the structural type. Therefore, frames with and without reinforced concrete shear walls, and frames with and without non-structural masonry infill panels could all be combined in the same plot, while the structural differences in these buildings were reflected by their initial fundamental periods. When the results for all buildings, including those for 5, 10, and 15-storey frame and frame-shear wall buildings are shown in the same plot, a significant scatter in data is observed, though the same trend remains to be evident. The inter-storey drift demand is consistently higher than the overall drift. Since the inter-storey drift quantities are of more significance to structural engineers and are also more critical, these values are summarized below.

The storey drift demand under artificial records, with 10 % probability of occurrence in 50 years, vary between approximately 0.2 % and 1.2 %. The storey drift demand under previously recorded earthquakes, with intensities adjusted to reflect a similar probability level, show a variation of 0.4 % and 2.5 %. When the artificial records are scaled to yield the same base shear as that recommended in NBCC-95[58], this variation becomes 0.2 % to 3.1 %. Although the intensity of the latter set of earthquakes may be questionable, they show the highest drift demands. The values summarized above are indicative of the range for drift demands in Western Canada.

A more accurate estimate of drift demands can be obtained with due considerations given to the building height, as suggested in Figs. 5.79 to 5.81.

5.3.2 Eastern Canada

5.3.2.1 Artificial Records

Frame buildings, as well as buildings with a frame-shear wall interactive system were analyzed under the artificial records generated by Atkinson and Beresnev [68] for Ottawa, representing seismic characteristics of Eastern Canada. The buildings were either 5, 10 or 15 stories high. The frame buildings had fundamental periods of 1.73 sec., 3.73 sec., and 5.24 sec. for 5, 10 and 15 storey buildings, respectively. Figs. 5.82 to 5.84 show maximum lateral drift, inter-storey drift, and maximum ductility responses for the frame buildings. The results indicate that the maximum lateral drift in frames was limited to approximately 0.13 % and the maximum inter storey drift was limited to 0.2 %. Because of the low intensity of eastern earthquake records, the frame buildings remained elastic in most cases, with the exception of the 15-storey building, which experienced some yielding in beams with a maximum ductility ratio of 1.3. The moment response in columns remained within 40% of yield moment.

Figures 5.85 to 5.87 show maximum lateral drift and inter-storey drift along the height of 5, 10, and 15 storey frame-shear wall structures, with fundamental periods of 0.63 sec., 1.76 sec., and 3.52 sec., respectively. The results indicate that the maximum lateral drift was limited to approximately 0.12 % in all cases, which is about one forth of that computed in the west. The maximum storey drift was limited to approximately 0.16 %. There was little difference between responses of frame and frame-shear wall

buildings. The buildings remained elastic in all cases, with maximum moments in beams and columns approaching about 60% and 40 % their respective yield values. The wall response indicated maximum moments ranging between 30 % to 60 % of yield at the base. Figs. 5.88 to 5.90 illustrate ductility ratios of less than 1.0 in all members, signifying elastic response.

5.3.2.2 Previously Recorded Actual Earthquakes

The same 3 frame buildings and 3 frame-shear wall buildings designed for Ottawa were also analyzed under the 1988 Saguenay record, which was found to be the critical earthquake record for Eastern Canada in Chapter 4. The record was scaled to give a PGA of 20 % g, which was the intensity associated with 10% probability of occurrence in 50 years, based on the NBCC-95 Building Code. Figs. 5.91 to 5.93 show maximum lateral drift, inter-storey drift, and maximum ductility ratio along the height of 5, 10, and 15 storey frame structures, respectively. The results indicate that maximum lateral and inter-storey drift demands were both limited to approximately 0.12 %. This is a drift level which is significantly lower than those computed for corresponding buildings in the west, under previously recorded western earthquakes. It is also 40% lower than those found under eastern artificial records. All three frame buildings remained elastic during response to the 1988 Saguenay Earthquake. The maximum column moment computed was approximately equal to 40% of the yield moment, while the beams approached yielding in at least one building, developing displacement response in the range of 0.6 % to 0.9 % of yield.

The frame-shear wall buildings were also analyzed under the 1988 Saguenay

Earthquake. Figures 5.94 to 5.96 show maximum lateral drift and inter-storey drift along the height of 5, 10, and 15 storey frame-shear wall structures. The results indicate maximum drift response of approximately 0.1 % in all cases. This is a very small drift demand, smaller than that computed for the artificial records. With this small value, all members in all three buildings remained elastic as illustrated in Figs. 5.97 to 5.99.

5.3.2.3 Scaled Artificial Records to Produce NBCC-95 Base Shears

The same 3 frame and 3 frame-shear wall buildings were analyzed one more time under a different intensity level. The artificial records proposed for Eastern Canada produced lower base shears than those required by NBCC-95 [58]. Therefore, in this set of analyses the artificial records were first scaled to give the same base shear values as those specified by NBCC-95. Since the difference in base shear was higher in the east than in the west, the scale factors employed were considerably higher. The results for the 3 frame buildings are presented in Figs. 5.100 to 5.102 for drift and ductility response. These buildings showed a maximum drift demand of about 0.63 %. The storey drifts demands were higher, showing 0.56 %, 0.74 %, and 1.2 % for 5, 10, and 15-storey buildings, respectively. These amounts are the highest drift response computed for frame buildings in the east. While the columns remained elastic in all cases with a maximum moment response equal to about 70 % of the yield moment, the beams experienced yielding, showing ductility demands of 2 to 6.

The drift response of frame-shear wall buildings are shown in Figs. 5.103 to 5.105, and indicate that the maximum lateral drift was limited to approximately 0.33 % in all

cases. The maximum storey drift was 0.39 %, which was higher than that indicated by previous analyses. The beams remained elastic in 5-storey and 10-storey buildings, but experienced inelasticity in the 15-storey building with a maximum ductility ratio of 1.5. Similarly the shear walls remained elastic in all buildings, except in the 15-storey building, where the maximum ductility ratio was computed to be 1.8. The columns remained elastic, with maximum moments ranging approximately between 40% and 80% of the yield value. The ductility response is depicted in Figs. 5.106 to 5.108.

5.3.2.4 Conclusions on Drift Demands in the East

The drift response was re-evaluated in this section in an attempt to draw conclusions that can be generalized. Maximum drift demands of all 6 buildings, with and without shear walls, are plotted together in Figs. 5.109 to 5.111. The results indicate a clear trend of increase in lateral drift with fundamental period. This trend becomes more evident when the data for each building height is evaluated separately. It was found that while the drift demand was very small under the artificial and Saguenay records, a correlation existed between storey drifts in Vancouver and those for Ottawa when scaled artificial records were used to give the same base shear as that recommended by NBCC-95.

The results indicate that inter-storey drift response is higher than lateral drift. The maximum inter-storey drift remains below 0.2 % under both artificial and 1988 Saguenay Earthquakes with 10 % probability of occurrence in 50 years. The same drift demand increases up to approximately 0.5 % for 5-storey buildings and 1.2 %

for 15-storey buildings, respectively, when the artificial records were scaled up to give the same base shear as NBCC-95 [58]. In general the buildings remain elastic during earthquakes, while developing limited inelasticity in the beams of 15-storey buildings.

Chapter 6

Static Inelastic (Push Over) Analysis

6.1 Introduction

An alternative method for seismic analysis of structures is push-over analysis, in which lateral static loads are gradually imposed on the structure. This requires consideration of inelasticity in members under increasing lateral loading. Static inelastic analysis was introduced into computer program DRAIN-RC as part of the current investigation. This was explained in Chapter 2. The program was used to analyze previously designed buildings under incrementally increasing static loads. The results are presented in this chapter.

6.2 Push-Over Analysis of Buildings

Push-over analyses of all the buildings designed were conducted by applying incrementally increasing equivalent seismic forces as lateral loads, while keeping gravity loads on the structure. The distribution of the lateral load was the same as that recommended in the National Building Code of Canada (NBCC-95)[58]. The gravity loads contributed to structural response, including secondary effects caused by P- Δ effect.

The distribution of lateral force along the height of the structure is established from

the following expression:

$$F_x = \frac{(V - F_t) W_x h_x}{(\sum W_i h_i)} \quad (6-1)$$

where;

F_x : Lateral force applied at level x .

V : Lateral seismic force at the base of the structure (base shear).

F_t : The portion of V to be concentrated at the top of the structure.

W_x, W_i : Those portions of dead load which are located at or are assigned to level x or i , respectively.

h_x, h_i : Height to level x or i , respectively, relative to the base where the base is the level at which horizontal earthquake motions are considered to be imported to the structure.

Inelasticity in static analysis was considered through the same inelastic springs used for dynamic analysis. The same hysteretic models for flexure, shear and anchorage slip were used, except this time only the primary curves under monotonic loading were utilized. The axial force-flexure interaction during response was not included in analyses, as in the case of dynamic analyses. All other features of DRAIN-RC, including the $P-\Delta$ effect, were included.

6.2.1 Moment Resisting Frames

Ten-storey moment resisting frame in western Canada was analyzed first. The structure was analyzed twice; first, without any consideration of $P-\Delta$ effects, and next with the $P-\Delta$ effects incorporated into the analysis. Force-lateral drift relationships for

both cases are illustrated in Fig. 6.1 for roof, as well as 5th floor where the drift was the highest. The results indicate that the P- Δ effect was very significant. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. The maximum inter-storey drift was in excess of 4.0% and the maximum ductility ratio computed in beams was in excess of 15. The maximum load resistance was equal to 200 % of the design base shear when the analysis was terminated. Fig. 6.2 shows variations in drift, inter-storey drift, and ductility demands, along the height of the building under increasing lateral loads. The same building showed a more realistic behaviour when secondary deformations due to the P- Δ effect were considered. The building experienced instability failure at a maximum inter-storey drift of about 1 %. The over-strength ratio, relative to the design base shear, was approximately 1.2 when the structure collapsed. At this load stage, the maximum beam ductility ratio was 3.4, while the columns remained elastic. Fig. 6.3 illustrates the comparison of drift and ductility demands between the cases with and without the P- Δ effect, just before instability.

Five-storey moment resisting frame in western Canada was analyzed next. The structure was analyzed twice; with and without the P- Δ effects. Force-lateral drift relationships for both cases are illustrated in Fig. 6.4 for roof, as well as the 3rd floor where the drift was the highest. The results indicate that the P- Δ effect was significant, although less significant than that observed for the 10-storey building. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. The maximum inter-storey drift was in excess of 4.5 % and the maximum ductility ratio computed in beams was in excess of 15.0 when the analysis was terminated. The maximum load resistance at this stage of loading was equal to 300 % of the design base shear. Fig. 6.5 shows variations in drift, inter-storey drift, and ductility demands along the height of the building. The

same building showed a more realistic behaviour when secondary deformations due to the P- Δ effect were considered. The initial yield occurred in the beams when the applied load exceeded the design base shear by about 10 %. The building experienced instability failure at a maximum inter-storey drift of approximately 3.0 %. The over-strength ratio, relative to the design base shear, was approximately 2.1 when the structure collapsed. At this load stage, the maximum beam and column ductility ratios were 10.4 and 4.4, respectively. Fig. 6.6 illustrates the comparison of drift and ductility demands between the cases with and without the P- Δ effect, just before instability.

Fifteen-storey moment resisting frame in western Canada was analyzed next. The structure was analyzed twice, like the previous frame structures; with and without the P- Δ effects. Force-lateral drift relationships for both cases are illustrated in Fig. 6.7 for roof, as well as 9th floor where the drift was the highest. The results indicate that the P- Δ effect was as significant as it was for the 10-storey building. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. The maximum inter-storey drift was in excess of 6.0 % and the maximum ductility ratio computed in beams was in excess of 15.0 when the analysis was terminated. The maximum load resistance at this stage of loading was equal to 200 % of the design base shear. Fig. 6.8 shows variations in drift, inter-storey drift, and ductility demands along the height of the building. The same building showed a more realistic behaviour when secondary deformations due to the P- Δ effect were considered. The initial yield occurred in the beams when the applied load exceeded the design base shear by about 6 %. The building experienced instability failure at a maximum inter-storey drift of approximately 1.2 %. The over-strength ratio, relative to the design base shear, was approximately 1.2 when the structure collapsed. At this load stage, the maximum beam ductility ratios was 2.4 while the

columns remained elastic. Fig. 6.9 illustrates the comparison of drift and ductility demands between the cases with and without the P- Δ effect, just before instability.

The three frame buildings designed for Ottawa, representing the seismic conditions of eastern Canada, were also subjected to inelastic push-over analysis. The 10-storey building was first to be analyzed with and without the P- Δ effects. Force-lateral drift relationships for both cases are illustrated in Fig. 6.10 for roof, as well as 5th floor where the drift was the highest. The results indicate that the P- Δ effect was as significant as the companion building in the west. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. The maximum inter-storey drift was in excess of 4.5 % and the maximum ductility ratio computed in beams was in excess of 15.0 when the analysis was terminated. The maximum load resistance at this stage of loading was equal to 200% of the design base shear. Fig. 6.11 shows variations in drift, inter-storey drift, and ductility demands along the height of the building. The same building showed a more realistic behaviour when secondary deformations due to the P- Δ effect were considered. The initial yield occurred in the beams when the applied load was almost exactly equal to the design base shear. The building experienced instability failure at a maximum inter-storey drift of approximately 1.2 %. At this load stage, the maximum beam ductility ratios was 3.7, while the columns remained elastic. Fig. 6.12 illustrates the comparison of drift and ductility demands between the cases with and without the P- Δ effect, just before instability.

Five-storey moment resisting frame designed for eastern Canada was analyzed by imposing incrementally increasing lateral load. The structure was analyzed twice; with and without the P- Δ effects. Force-lateral drift relationships for both cases are illustrated in Fig. 6.13 for roof, as well as the 3rd floor where the drift was the highest.

The results show that the P- Δ effect was significant and should be considered in analysis. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. The maximum inter-storey drift was in excess of 5.0 % and the maximum ductility ratio computed in beams was in excess of 20.0 when the analysis was terminated which are unrealistically high values. The maximum load resistance at this stage of loading was equal to 300 % of the design base shear. Fig. 6.14 shows variations in drift, inter-storey drift, and ductility demands along the height of the building. The same building showed a more realistic behaviour when secondary deformations due to the P- Δ effect were considered. The initial yield occurred in the beams when the applied load exceeded the design base shear by about 7 %. The building experienced instability failure at a maximum inter-storey drift of approximately 1.8 %. The over-strength ratio, relative to the design base shear, was approximately 2.0 when the structure collapsed. At this load stage, the maximum beam and column ductility ratios were 6.0 and 1.5, respectively. Fig. 6.15 illustrates the comparison of drift and ductility demands between the cases with and without the P- Δ effect, just before instability.

The fifteen-storey building designed for eastern Canada was the last moment resisting frame which was analyzed. The structure was analyzed twice; with and without the P- Δ effects. Force-lateral drift relationships for both cases are illustrated in Fig. 6.16 for roof, as well as 9th floor where the drift was the highest. The results indicate that the P- Δ effect was as significant as it was for the 15-storey building in the west. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. When the analysis was terminated; the maximum inter-storey drift was about 6.0 % and the maximum beam and column ductility ratios were about 15.0 and 1.5, respectively. The maximum load resistance at this stage of loading was equal to 200 % of the design base shear. Fig. 6.17 shows

variations in drift, inter-storey drift, and ductility demands along the height of the building. The same building, similar to the others, showed a more realistic behaviour when secondary deformations due to the $P-\Delta$ effect were considered. The initial yield occurred in the beams when the applied load exceeded the design base shear by about 6 %. The building experienced instability failure at a maximum inter-storey drift of approximately 1.2 %. The over-strength ratio, relative to the design base shear, was approximately 1.2 when the structure showed instability. At this load stage, the maximum beam ductility ratios was 2.4, while the columns remained elastic. Fig. 6.18 illustrates the comparison of drift and ductility demands between the cases with and without the $P-\Delta$ effect, just before instability.

6.2.2 Frame-Shear Wall Buildings

The frame-shear wall structures designed for Vancouver and Ottawa, representing western and eastern Canadian seismic conditions were subjected to static inelastic analysis, similar to the frame buildings analyzed earlier. Western Canada was considered first. Fig. 6.19 illustrates the results of analyses for the 10-storey building in Vancouver, with and without the $P-\Delta$ effects. The results indicate that the $P-\Delta$ effect was also significant in shear wall structures. Without this effect, the building continued resisting increased lateral loads beyond realistic values of drift and ductility ratio. The maximum inter-storey drift was in excess of 6.5 % and the maximum ductility ratio in beams and shear walls was in excess of 20.0 when the analysis was terminated. The maximum load resistance at this stage of loading was approximately equal to 350 % of the design base shear. Fig. 6.20 and 6.21 show variations in drift, inter-storey drift, and ductility demands along the height of the building. The same building showed a more realistic behaviour when secondary deformations due to the $P-\Delta$ effect were considered. The initial yield occurred in the beams when the applied load exceeded the design base shear by about 30 %. The building experienced

instability failure at a maximum inter-storey drift of approximately 3.0 %. The over-strength ratio, relative to the design base shear, was approximately 2.2 when the structure collapsed. At this load stage, the maximum beam, column, and wall ductility ratios were 9.6, 6.3, and 11.6, respectively. Fig. 6.22 illustrates the comparison of ductility demands between the cases with and without the P- Δ effect, just before instability.

Force-drift relationships obtained from push-over analyses of five-storey frame-shear wall buildings are illustrated in Fig. 6.23. The results indicate reduced influence of the P- Δ effect, since absolute displacements of a 5-storey building, laterally stiffened by a shear wall are small. The building did not suffer from instability failure. It continued resisting lateral loads, beyond realistic values of drift and ductility ratio. This is explained by the characteristics of inelastic springs used in modeling non-linear response. As indicated earlier, the hysteretic models used in dynamic analysis were employed in the push-over analysis, except this time only the primary curves (the back bone curve) were utilized. However, the primary curves for the models do not recognize any strength decay in member, and hence must be used with care with due limitations imposed on ductility demands. Certainly, a ductility ratio of 35 computed for walls at the termination of static analysis is not realistic, and the corresponding point on the force-drift relationship should not be used as true response. The variation of ductility demands for beams, columns, and walls, under incrementally increasing static lateral loads is shown in Fig. 6.24.

The results of 15-storey frame shear wall structure in Vancouver are illustrated in Fig. 6.25. They indicate that the P- Δ effect once again became a significant parameter on building behaviour. While the building resistance continued increasing without the P- Δ effect, the building failed at about 1.0 % storey-drift due to instability, when the P-

Δ effect was considered. The over-strength ratio, relative to static design base shear in this case, was equal to 1.6. At failure the ductility ratios were computed to be 2.6, 1.5, and 3.4 for beams, columns, and the wall, respectively. Fig. 6.26 depicts ductility demands along the height, when the P- Δ effect was ignored.

Figs. 6.27 through 6.29 illustrate the behaviour of frame-shear wall buildings designed for Ottawa, representing eastern Canada. The trends observed earlier for western Canada also apply to the building behaviour observed in these analyses. The P- Δ effect was significant in 10-storey and 15-storey buildings, but showed a reduced influence on the 5-storey building. Building resistance continued increasing beyond realistic values of drift and ductility demands when secondary deformations due to P- Δ effects were ignored. The same trend was also observed in the 5-storey shear-wall building even when the secondary effects were considered. The other two structures, i.e., 10 and 15-storey buildings failed due to instability when the P- Δ effect was included in analyses. The 10-storey building developed its first yield at a 60 % higher lateral load than the design base shear, and showed an over-strength ratio of about 2.5, relative to the same design value. The 15-storey building developed its first yield at a lateral load of about 15 % higher than the design base shear, and failed at about 180 % of its design base shear.

6.3 Significance of Push-Over Analysis

Static inelastic analysis (Push-over analysis) of structures for seismic evaluation of buildings has been gaining recognition as an alternative to dynamic time history analysis. While there is sufficient incentive to pursue static analysis, in view of its simplicity and the uncertainties associated with earthquake records needed for dynamic analysis, the usefulness of push-over analysis is often questioned by researchers.

The push-over analysis may provide the following information for seismic evaluation of structures:

1. Overall strength and available over-strength (strength relative to design base shear) under a given lateral load distribution.
2. Overall drift and inter-storey drift capacities.
3. Distribution of plastification within the structure and identification of potentially critical regions for improved design and detailing.
4. Ductility demands.

The 12 buildings designed for Ottawa and Vancouver, representing eastern and western Canadian seismic regions, were analyzed under incrementally increasing static lateral loads, as presented in the preceding sections. The results indicate that push-over analysis produces critical response in terms of stability of structures as compared to dynamic analysis. All but one building analyzed under incrementally increasing lateral static loads failed due to instability. This was not the case, however, when the P- Δ effect was ignored in analyses, illustrating the importance of P- Δ effects on building behaviour. Similarly, the same buildings did not show any failure when subjected to dynamic loading of similar magnitude. One set of ground motion records considered in the dynamic analysis was scaled to give the same base shear as static design base shear. Though some buildings experienced higher lateral displacements under dynamic loading, they did not develop instability.

The over-strength ratio (ratio of maximum lateral load capacity to design base shear) in 10 and 15-storey frame buildings was approximately 1.2. Five-storey frame buildings showed higher over-strength ratios with values of approximately 2.0. The over-strength ratio for shear wall buildings ranged between 1.6 and 4.0. Initial yielding in all structures occurred shortly after exceeding the design base shear. Most

buildings showed the first yielding at approximately 5 % to 10 % above the design base shear. Table 6-1 summarizes maximum roof displacements computed under dynamic load condition.

Ductility demands and yield patterns under static loading were investigated by comparing ductility ratios for beams, columns and walls obtained from push-over analyses with those obtained from dynamic response history analyses. The comparison was made at a performance level corresponding to the same roof displacement. Hence, the ductility ratios are compared when the push-over analysis gave the same roof displacement as dynamic analysis. The roof displacement under governing artificial record was used for buildings located in Vancouver. For buildings in Ottawa, however, because the buildings remained elastic under artificial records, as well the Saguenay record, the comparison was made for the roof displacement under the governing scaled artificial record. The static lateral loads required to attain these roof displacements are summarized in Table 6.2. The same table also illustrates the value of total static load at initial yielding, as well as the maximum value when the building collapsed due to instability. Figs. 6.30 Through 6.47 show these comparisons for 5, 10 and 15-storey buildings in Vancouver and Ottawa. The results indicate very good correlations of drift and ductility demands when the first mode response was dominant. Five-storey and ten-storey buildings consistently showed approximately the same ductility demands under static and dynamic loading conditions. However, the correlation was not good for 15-storey buildings. This clearly illustrates the importance of static load distribution on the structure for push-over analysis. When higher mode effects were significant, as they would be in 15-storey buildings, the lateral load distribution used to simulate first mode deformations did not produce good agreement with dynamic values.

The yield pattern obtained under static and dynamic loads also produced a good correlation when the first mode response was dominant. This can be seen in Fig. 6.48 where yield patterns for a 5-storey building are compared.

The above comparative study indicates that push-over analysis may be used to assess ductility demands in structures. The locations of critical regions can be identified for improved design and detailing. It may also be used to establish over-strength in structures. The overall strength of structure, established by a push-over analysis, tends to be lower than those that can be sustained under dynamic earthquake loading.

Chapter 7

Summary and Conclusion

7.1 Summary

A total of 12 reinforced concrete buildings were designed to investigate inelastic drift demands of buildings under earthquake loading. The buildings were designed based on the current Canadian practice. An enhanced version of computer software DRAIN-2D was developed and labelled as DRAIN-RC, incorporating relevant features of reinforced concrete hysteretic response. The enhancement also included introduction of P- Δ effects and a capability for static inelastic push-over analysis. A parametric investigation was conducted to identify the significance of modelling features and ground motion characteristics. Three sets of earthquake records were selected for dynamic inelastic analysis. The first set consisted of artificial records consistent with recently proposed uniform hazard spectra. The second set included previously recorded strong motion records during actual earthquakes. The third set was obtained by scaling the records in the first set so that they produce the same design base shear as that recommended by NBCC-95.

Over 350 analyses were conducted to establish inelastic seismic drift demands of

reinforced concrete structures. The results were evaluated in terms of drift, story drift, and ductility ratio. The following section summarizes the conclusions obtained in this investigation.

7.2 Conclusion

The following conclusions can be derived from the analytical research conducted in this project:

- Computer software DRAIN-RC, developed as part of this investigation, incorporates the relevant features of hysteretic reinforced concrete response, and produces accurate results. The accuracy of the software was verified using available analytical tools, manual calculations, and by examining hysteretic member behaviour relative to expected response.
- Softening in reinforced concrete structures due to anchorage slip has an effect on dynamic characteristics of structures, leading to some differences in response. When inelasticity was limited, the difference in response was limited to approximately 10 %. However, when the intensity of earthquake motion was increased, the effect of anchorage slip on drift response was in excess of 15 %. It is therefore recommended that inelastic deformations due to anchorage slip should be considered in response history analysis, for improved accuracy.
- P-M interaction during response does not appear to have a significant influence on drift response of frame and frame-shear wall buildings. However, some effect was observed on ductility demands, as yield level changes with concurrent level of axial force. For drift and displacement based design approaches, P-M interaction effects during response may be ignored without significantly affecting results.
- Inelastic shear effects are negligibly small in a typical reinforced concrete frame

structure. However, shear wall structures exhibit significantly different response when these effects are included in analysis. The significance of inelastic shear effects in these structures increases with increasing inelasticity. The shear wall building designed for Vancouver produced an 18 % increase in lateral drift when inelasticity in shear was permitted at a shear force corresponding to flexural yielding. When the shear capacity was below that corresponding to flexural yielding, the difference in drift and inter-storey drift, due to the consideration of inelastic shear effects, was as high as approximately 50 % and 60 %, respectively.

- Secondary deformations due to P- Δ effect play a significant role on dynamic and static inelastic analyses. This effect is magnified with inelasticity and associated increase in lateral deflections. While the shear wall structures analyzed showed limited P- Δ effect due to their increased lateral stiffnesses and correspondingly reduced deflections, the frame structures exhibited increased lateral drift due to the P- Δ effect. The results obtained indicate an increase of as much as 35% in drift response and 40% in inter-storey drifts. In one case, the building analyzed with the P- Δ effect collapsed during a given earthquake, whereas the same building survived the earthquake with approximately 5% maximum drift when the same earthquake record was used without any P- Δ effect. Similar observations were made relative to static inelastic (push-over) analyses. The analyses conducted in this investigation provide sufficient data to conclude that P- Δ effects play a crucial role on building response and that they can not be ignored in response history analysis for earthquake effects.
- Non-structural masonry infill walls play a very significant role on building response to seismic forces. They stiffen structures and reduce the period. This puts the structure in a completely different seismic risk category. Consideration of masonry walls, in establishing the period of structure is vitally important in establishing seismic design base shear. In the frame buildings considered in this

investigation, masonry walls covering only two bays with a wall-to-floor-area ratio of 0.2 %, was sufficient to reduce the period and lateral drift by a factor of approximately 2.0. The same building remained mostly elastic, with little or no damage to the masonry. When the wall-to-floor area ratio was increased, elastic behaviour of the entire structure was ensured under earthquake records with 10% probability of occurrence in 50 years. The effect of masonry walls could be to change building response from inelastic to elastic. It was further found that drift demands in frames braced with unreinforced masonry walls change almost linearly with initial fundamental period. Hence, the use of masonry walls can reduce drift demands substantially.

- Displacement response increases with the intensity of earthquake. When structural response remains elastic, or have limited inelasticity, the variation of drift demand with intensity is approximately linear. This was observed in buildings located in Ottawa. However, this linear relationship ceases to exist when significant inelasticity is developed. Another factor contributing to change in linearity is the presence of secondary deformations caused by P- Δ effects.
- Storey drift demands in western Canada are below 2.5 % and 1.0% for frame and frame-shear wall buildings, respectively, under earthquake records believed to have 10 % probability of occurrence in 50 years. When this probability level is reduced, as in the case of scaled artificial records, the storey drift can be as high as approximately 3.0 %.
- Storey drift demands in eastern Canada are below 0.2 % for both frame and frame-shear wall buildings, under earthquake records believed to have 10 % probability of occurrence in 50 years. When this probability level is reduced, as in the case of scaled artificial records, the storey drift can be as high as approximately 1.0 %.
- Push-over analysis may be a powerful tool for seismic analysis of reinforced

concrete buildings. However, its usefulness may be limited to structures behaving predominantly in the first mode, if the building code recommended equivalent static load distribution is to be used. Buildings of up to 10 stories, considered in the current investigation, showed good correlations between static and dynamic ductility demands, as well as static and dynamic drift demands when the comparison is made for a load stage producing the same roof displacement. Furthermore, the yield patterns observed under dynamic and static loads were very similar, signifying that push-over analysis may be used to identify potentially critical regions of structures during seismic response.

7.3 Recommendations for Further Research

The research program undertaken in this dissertation covered a large number of reinforced concrete buildings. However, a number of cases were not studied, including buildings with different heights, stiffened by infill panels. It is recommended that the current investigation be extended to cover additional frame buildings with different heights, stiffened by non-structural elements. Furthermore, the floor plan considered was selected to be regular to avoid torsional effects. It is recommended that buildings with irregular floor plans be considered in future research.

One of the significant limitations of the current study is the earthquake records considered. Although every effort was made to generalize the conclusions by considering three types of records, additional dynamic analyses under record of different probability of occurrences would be beneficial to structural engineers. Further research is recommended to expand the results of the current investigation to cover seismic risks at other probability levels.

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Table 2.1 Comparison of results for a single column in pure sway with P- Δ and without rigid end offset (example 1)

Quantity	SAP90			DRAIN-RC			Theoretical
	1 Element model	2 Element model	10 Element model	1 Element model	2 Element model	10 Element model	
Δ_B	6.805	6.823	6.847	6.802	6.820	6.843	6.849
M_A	9054.2	9064.8	9079.0	9052.2	9062.8	9076.8	9084.0

Table 2.2 Comparison of results for a single column in pure sway with P- Δ and with rigid end offset (example 2)

Quantity	SAP90			DRAIN-RC			Theoretical
	1 Element model	2 Element model	10 Element model	1 Element model	2 Element model	10 Element model	
Δ_B	2.335	2.341	2.350	2.333	2.339	2.347	2.349
M_A	8339.2	8341.7	8356.8	8336.2	8338.7	8353.4	8358.8
M_B	7339.2	7351.0	7356.8	7336.2	7348.0	7353.4	7358.8
M_C	6339.2	6371.7	6356.8	6336.2	6338.7	6353.4	6358.8
M_D	6339.2	6351.0	6356.0	6336.2	6348.0	6353.4	6358.8

Table 2.3 Comparison of results for a single column with no sway with P- Δ and without rigid end offset (example 3)

Quantity	SAP90			DRAIN-RC			Theoretical
	1 Element model	2 Element model	10 Element model	1 Element model	2 Element model	10 Element model	
θ_B	.00156	.001680	.00170	.001557	.00168	.00171	.00170
M_A	87.84	101.20	102.0	87.85	101.18	101.98	102.0
V_B	1.878	2.012	2.02	1.88	2.01	2.02	2.02

Table 2.4 Comparison of results for a single column with no shear with P- Δ and without rigid end offset (example 4)

Quantity	SAP90			DRAIN-RC			Theoretical
	1 Element model	2 Element model	10 Element model	1 Element model	2 Element model	10 Element model	
θ_B	.00749	.007513	.00752	.007485	.007513	.0075146	.00752
Δ_B	.4181	.4197	.41981	.4181	.4197	.4198	.42
M_A	224.60	225.07	225.10	224.60	225.06	225.10	225.2

Table 2.5 Comparison of results for a single column. (example 5)

Quantity	DRAIN-2DX	DRAIN-RC	SAP90
Δ_B	55.09	93.48	93.49
M_A	150	219.7	219.7
V	30	10	10

Table 2.6 Comparison of results for two bay- two storey frame analysis (example 6)

Quantity	DRAIN-2DX	DRAIN-RC	SAP90	Consideration
Δ_7	0.0557869	0.05602	.056012	with P- Δ effect $L_L=6.0$ m $L_R=6.0$ m $H_1=5.0$ m $H_2=5.0$ m $W=100$ kN/m $P=200$ kN $(EI)_{beam}=43200$ kN-m ² $(EI)_{Column}=65000$ kN-m ²
Δ_4	0.0235203	0.02365	.023646	
M_{4-1}	-4.012	-5.18	-5.415	
M_{4-5}	-46.19	-45.15	-45.074	
M_{4-7}	50.2	50.33	50.489	
$\sum M$	0	0	0	
V_{1-4}	37.90	35.32	35.94	
V_{2-5}	83.91	77.78	77.78	
V_{3-6}	89.48	86.90	86.28	
$\sum V$	211.29	200	200	
P_1	453.7	453.33	453.28	
P_2	1263.3	1263.32	1263.45	
P_3	683.00	683.35	683.27	
$\sum P$	2400	2400	2400	

Table 2.7 Comparison of results for a single column with P- Δ and inelastic behaviour

Quantity	SAP90			DRAIN-RC
	Elastic	Inelastic	Total	Total
θ_B	0.00501	0.025389	0.030403	0.029177
Δ_B	0.017222	0.126155	0.143377	0.137
M_A	99.98	10.0558	110.0358	109.57

Table 3.1 Data for design of buildings**Dead Loads:**Floor.....5.0 KN/m²Roof..... 3.5 KN/m²**Live Loads:**Floor.....2.4 KN/m²Roof.....2.2 KN/m²**Material strength:**

Reinforcement Yield Strength.....400 MPa

Concrete Compressive Strength.....30 MPa

Table 3.2 Reinforcement arrangements and sectional size of ten-storey frame building for western Canada

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 25	450 x 450 12 # 20
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 20	450 x 450 8 # 20
	10	300 x 400 3 # 20 top, 2 # 15 bottom	500 x 500 12 # 20	450 x 450 8 # 20
Exterior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	400 x 400 8 # 25	350 x 350 8 # 25
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	400 x 400 8 # 20	350 x 350 4 # 25
	10	300 x 400 3 # 20 top, 2 # 15 bottom	400 x 400 8 # 20	350 x 350 4 # 25

Table 3.3 Reinforcement arrangements and sectional size of ten-storey frame-shear wall building for western Canada

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 25	350 x 350 8 # 25
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 20	350 x 350 4 # 25
	10	300 x 400 3 # 20 top, 2 # 15 bottom	500 x 500 12 # 20	350 x 350 4 # 25
Exterior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	N.A	300 x 300 8 # 20
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	N.A	300 x 300 4 # 20
	10	300 x 400 3 # 20 top, 2 # 15 bottom	N.A	300 x 300 4 # 20

Table 3.4 Reinforcement arrangements and sectional size of ten-storey frame building for eastern Canada

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 25	400 x 400 8 # 25
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 20	400 x 400 8 # 20
	10	300 x 400 3 # 20 top, 2 # 15 bottom	500 x 500 12 # 20	400 x 400 8 # 20
Exterior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	400 x 400 8 # 25	350 x 350 8 # 25
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	400 x 400 8 # 20	350 x 350 4 # 25
	10	300 x 400 3 # 20 top, 2 # 15 bottom	400 x 400 8 # 20	350 x 350 4 # 25

Table 3.5 Reinforcement arrangements and sectional size of ten-storey frame-shear wall building for eastern Canada

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 25	350 x 350 8 # 25
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	500 x 500 12 # 20	350 x 350 4 # 25
	10	300 x 400 3 # 20 top, 2 # 15 bottom	500 x 500 12 # 20	350 x 350 4 # 25
Exterior	1-5	300 x 450 3 # 25 top, 2 # 20 bottom	N.A	300 x 300 8 # 20
	6-9	300 x 450 3 # 25 top, 2 # 20 bottom	N.A	300 x 300 4 # 20
	10	300 x 400 3 # 20 top, 2 # 15 bottom	N.A	300 x 300 4 # 20

Table 3.6 Artificial and selected ground motion records used in dynamic analysis.

Location	Ground Motion Record	Max. Acceleration. (cm/s²)	PGA (% g)	ΔT (sec)	Duration (sec)
West	Short Event No.1	180.4	18.4	.01	8.0
	Short Event No.2	220.0	22.4	.01	8.0
	Long Event No.1	85.6	8.7	.01	20.0
	Long Event No.2	83.2	8.5	.01	20.0
	El Centro (1940)	182.1	18.6	.02	10.0
	Taft (1952)	152.7	15.6	.02	31.0
	San Fernando (1971)	147.6	15.1	.02	37.0
	Northridge No.1 (1994)	428.1	43.6	.02	13.45
	Northridge No.2 (1994)	570.0	58.1	.02	13.45
	Northridge No.3 (1994)	420.4	42.9	.02	13.45
	Northridge No.4 (1994)	547.5	55.8	.02	13.45
East	Short Event No.1	140.0	14.3	.01	10.0
	Short Event No.2	152.0	15.5	.01	10.0
	Long Event No.1	91.7	9.4	.01	26.0
	Long Event No.2	86.1	8.8	.01	26.0
	Saguenay, Quebec (1988)	123.1	12.6	.005	44.0
	New Brunswick (1982)	244.7	24.9	.005	20.0

Table 4.1. Comparison of drift response with and without inelastic shear deformation.

Location	Frame Type	No. of Storey	Ground Motion Record	Maximum Drift (%)				Maximum Inter-Storey Drift (%)		
				without Shear	with Shear	with Shear / without Shear	without Shear	with Shear	with Shear / without Shear	without Shear
West	MRF	10	Long Event No. 2	2.542	2.543	1.00	3.275	3.275	1.00	1.00
	MRF	10	1940 El Centro	1.493	1.490	1.00	1.923	1.923	1.00	1.00
	SHW	10	Long Event No. 2	0.724	0.852	1.18	0.808	0.935	1.16	1.16
	SHW	10	1940 El Centro	0.537	0.583	1.09	0.613	0.690	1.13	1.13
East	MRF	10	Long Event No. 2	0.259	0.256	0.99	0.300	0.298	0.99	0.99
	MRF	10	1988 Saguenay	0.092	0.092	1.00	0.108	0.108	1.00	1.00
	SHW	10	Long Event No. 2	0.271	0.250	0.92	0.365	0.328	0.90	0.90
	SHW	10	1988 Saguenay	0.056	0.056	1.00	0.114	0.114	1.00	1.00

Table 4.2. Comparison of drift response with and without inelastic shear deformation for different level of yield in shear.

Location	Frame Type	No. of Storey	$(M_y)_{shear} / (M_y)_{flexure}$	Maximum Drift (%)			Maximum Inter-Storey Drift (%)		
				without Shear	with Shear	with Shear / without Shear	without Shear	with Shear	with Shear / without Shear
West	SHW	10	0.75	0.724	.915	1.26	0.808	1.042	1.29
	SHW	10	0.50	0.724	1.111	1.53	0.808	1.300	1.61

Table 4.3. Comparison of drift response with and without P- Δ .

Location	Frame Type	No. of Storey	Ground Motion Record	Maximum Drift (%)				Maximum Inter-Storey Drift (%)		
				without P- Δ	with P- Δ	with P- Δ / without P- Δ	without P- Δ	with P- Δ	with P- Δ / without P- Δ	
West	MRF	10	Long Event No. 2	2.256	2.543	1.13	2.825	3.275	1.16	
	MRF	10	1940 El Centro	1.714	1.490	0.87	2.260	1.923	0.85	
	SHW	10	Long Event No. 2	0.777	0.852	1.10	0.858	0.935	1.09	
	SHW	10	1940 El Centro	0.594	0.583	0.98	0.715	0.690	0.97	
East	MRF	10	Long Event No. 2	0.327	0.256	0.78	0.368	0.298	0.81	
	MRF	10	1988 Saguenay	0.088	0.092	1.05	0.105	0.108	1.03	
	SHW	10	Long Event No. 2	0.262	0.250	0.95	0.345	0.328	0.95	
	SHW	10	1988 Saguenay	0.056	0.056	1.00	0.116	0.114	0.98	

Table 5-1. Scale factor of artificial records for buildings in western Canada.

Location	Frame Type	No. of Storey	Period (sec)	Record	Scale	Intensity (%g)
West	Moment Resisting Frame	5	1.73	Long # 1	1.01	8.9
				Long # 2	1.08	9.2
				Short # 1	1.70	31.2
				Short # 2	0.79	17.8
		10	3.70	Long # 1	2.01	17.6
				Long # 2	1.60	13.6
		15	5.24	Long # 1	2.39	20.9
				Long # 2	1.60	13.6
	Frame Shear Wall	5	0.59	Long # 1	1.34	11.7
				Long # 2	1.91	16.2
				Short # 1	0.63	11.5
				Short # 2	0.84	18.9
		10	1.77	Long # 1	0.96	8.4
				Long # 2	1.13	9.6
				Short # 1	1.65	30.4
				Short # 2	0.81	18.2
		15	3.38	Long # 1	1.34	11.7
				Long # 2	1.37	11.6
				Short # 1	2.06	37.9
				Short # 2	2.35	52.8

Table 5-2. Scale factor of artificial records for buildings in eastern Canada.

Location	Frame Type	No. of Storey	Period (sec)	Record	Scale	Intensity (%g)
East	Moment Resisting Frame	5	1.78	Long # 1	3.42	32.0
				Long # 2	1.67	14.7
				Short # 1	3.95	56.3
				Short # 2	5.44	84.2
		10	3.73	Long # 1	4.78	44.7
				Long # 2	5.29	46.5
		15	5.30	Long # 1	7.98	74.6
				Long # 2	4.61	40.5
	Frame Shear Wall	5	0.63	Long # 1	1.11	10.4
				Long # 2	0.89	7.8
				Short # 1	2.26	32.2
				Short # 2	1.55	24.0
		10	1.76	Long # 1	3.09	28.9
				Long # 2	1.15	10.1
				Short # 1	3.70	52.8
				Short # 2	5.15	79.8
		15	3.52	Long # 1	5.03	47.0
				Long # 2	5.22	45.8

Table 5-3. Scale factor of artificial records for 10-storey infilled frame, $f_m=8.6$ MPa

Location	Frame Type	No. of Storey	Infill	Period (sec)	Record	Scale	Intensity (%g)
West	Moment Resisting Frame	10	One Bay R=0.21% t=100 mm	2.39	Long # 1	1.75	15.3
					Long # 2	1.29	10.9
			Two Bay R=0.42% t=100 mm	1.75	Long # 1	0.99	8.6
					Long # 2	1.11	9.4
					Short # 1	1.67	30.8
					Short # 2	0.80	18.0
			Three Bay R=0.62% t=100 mm	1.41	Long # 1	1.87	16.3
					Long # 2	1.29	10.9
					Short # 1	1.78	32.7
					Short # 2	0.74	16.7
			One Bay R=0.42% t=200 mm	2.28	Long # 1	1.87	16.3
					Long # 2	1.33	11.3
			Two Bay R=0.83% t=200 mm	1.62	Long # 1	1.45	12.7
					Long # 2	1.11	9.40
					Short # 1	1.89	34.7
					Short # 2	0.75	16.9
			Three Bay R=1.24% t=200 mm	1.27	Long # 1	1.70	14.8
					Long # 2	1.30	11.0
					Short # 1	1.34	24.6
					Short # 2	0.79	17.8

Table 5-4. Scale factor of artificial records for 10-storey infilled frame, $f_m=15.2$ Mpa

Location	Frame Type	No. of Storey	Infill	Period (sec)	Record	Scale	Intensity (%g)
West	Moment Resisting Frame	10	One Bay R=0.21% t=100 mm	2.30	Long # 1	1.87	16.3
					Long # 2	1.32	11.2
			Two Bay R=0.42% t=100 mm	1.64	Long # 1	1.43	12.5
					Long # 2	1.06	9.0
					Short # 1	1.85	34.0
					Short # 2	0.76	17.0
			Three Bay R=0.62% t=100 mm	1.29	Long # 1	1.77	15.4
					Long # 2	1.36	11.6
					Short # 1	1.39	25.5
					Short # 2	0.77	17.3
			One Bay R=0.42% t=200 mm	2.14	Long # 1	1.64	14.3
					Long # 2	1.17	9.9
			Two Bay R=0.83% t=200 mm	1.54	Long # 1	1.53	13.3
					Long # 2	1.33	11.3
					Short # 1	2.00	36.8
					Short # 2	0.74	16.5
			Three Bay R=1.24% t=200 mm	1.19	Long # 1	1.72	15.0
					Long # 2	1.02	8.6
					Short # 1	1.23	22.7
					Short # 2	.77	17.2

Table 5-5. Maximum strut forces recorded in masonry walls, expressed in percentage of compressive capacity.

Bay	Thickness (mm)	Artificial Records		Selected Records		Scaled Artificial Records	
		$f_m=8.6$	$f_m=15.2$	$f_m=8.6$	$f_m=15.2$	$f_m=8.6$	$f_m=15.2$
1	100	80%	58%	Inel ¹	Inel ²	97%	85%
1	200	53%	41%	97%	70%	81%	54%
2	100	97%	62%	Inel ³	Inel ⁴	79%	55%
2	200	67%	49%	Inel ⁵	69%	52%	44%
3	100	71%	44%	Inel ⁶	67%	82%	50%
3	200	41%	32%	61%	42%	46%	31%

Superscript indicate the progression of inelasticity in masonry walls as indicated in figures below:

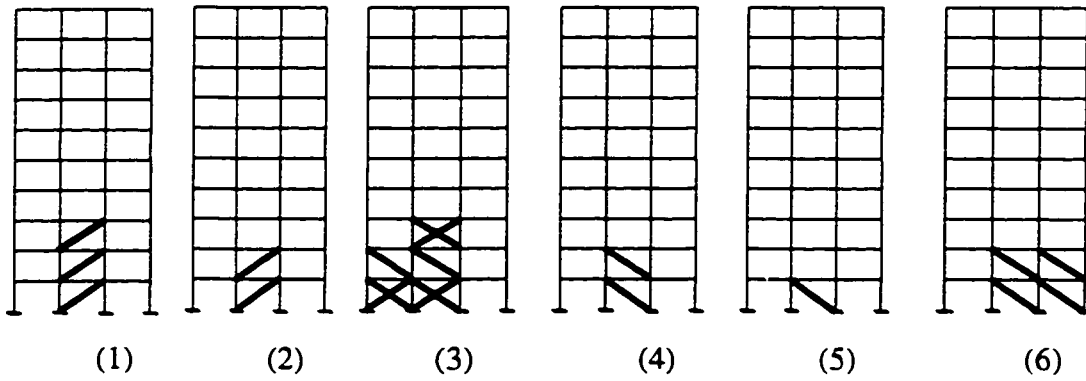


Table 6.1 Maximum roof displacement of buildings under dynamic load condition

Location	Frame Type	No. Of Storey	Maximum Roof Lateral Displacement (mm)		
			Artificial Records	Actual Records	Scaled Artificial Records
West	MRF	5	139	218	119
		10	169	400	313
		15	368	585	856
	SHW	5	40	67	26
		10	187	237	157
		15	141	614	317
East	MRF	5	20	16	66
		10	38	24	184
		15	52	37	292
	SHW	5	14	11	16
		10	49	22	99
		15	54	24	200

Table 6.2 The static load stage at first initial yielding, collapsed due to instability and required to attain the same roof displacements due dynamic analysis.

Location	Frame Type	No. Of Storey	Static Lateral Load		
			Same Roof Displacement	First Yielding	Instability (with P-Δ)
West	MRF	5	1.928 V_d	1.10 V_d	2.12 V_d
		10	1.102 V_d	1.05 V_d	1.195 V_d
		15	1.170 V_d	1.06 V_d	1.18 V_d
	SHW	5	1.221 V_d	1.07 V_d	3.93 V_d
		10	1.803 V_d	1.32 V_d	2.25 V_d
		15	0.774 V_d	1.04 V_d	1.61 V_d
East	MRF	5	1.357 V_d	1.07 V_d	2.03 V_d
		10	1.144 V_d	1.01 V_d	1.192 V_d
		15	1.000 V_d	1.03 V_d	1.17 V_d
	SHW	5	0.750 V_d	1.28 V_d	N/A
		10	1.348 V_d	1.65 V_d	2.46 V_d
		15	1.123 V_d	1.13 V_d	1.80 V_d

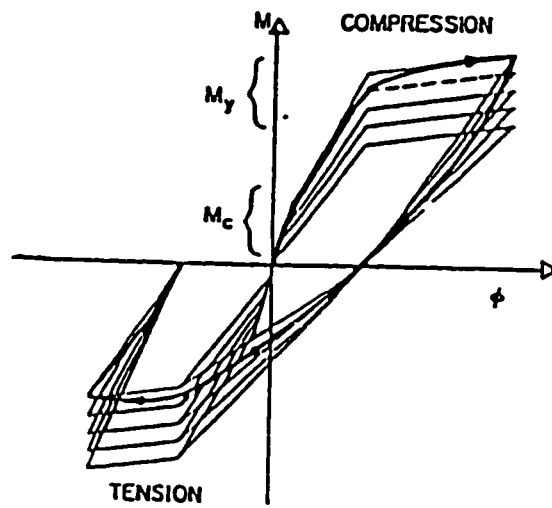


Fig. 1.1 Axial Force-Moment Interaction Model by Takayanagi and Schnobrich [14].

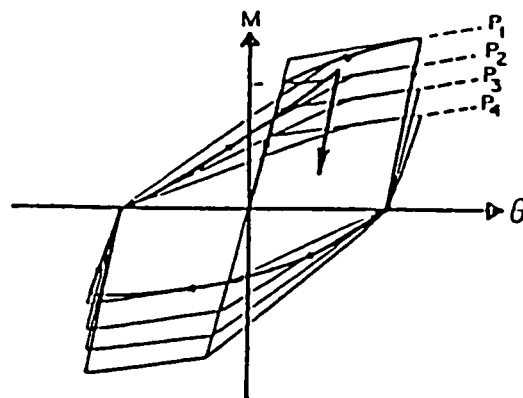
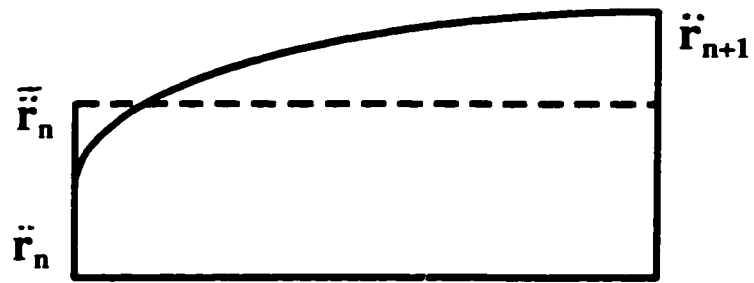
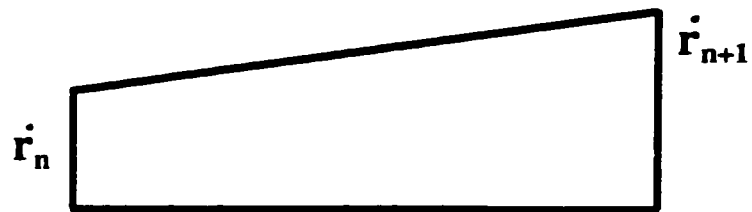


Fig. 1.2 Axial Force-Moment Interaction Model by Sattcioglu [15].



a) Acceleration



b) Velocity



c) Displacement

Fig. 2.1 Average Acceleration Method.

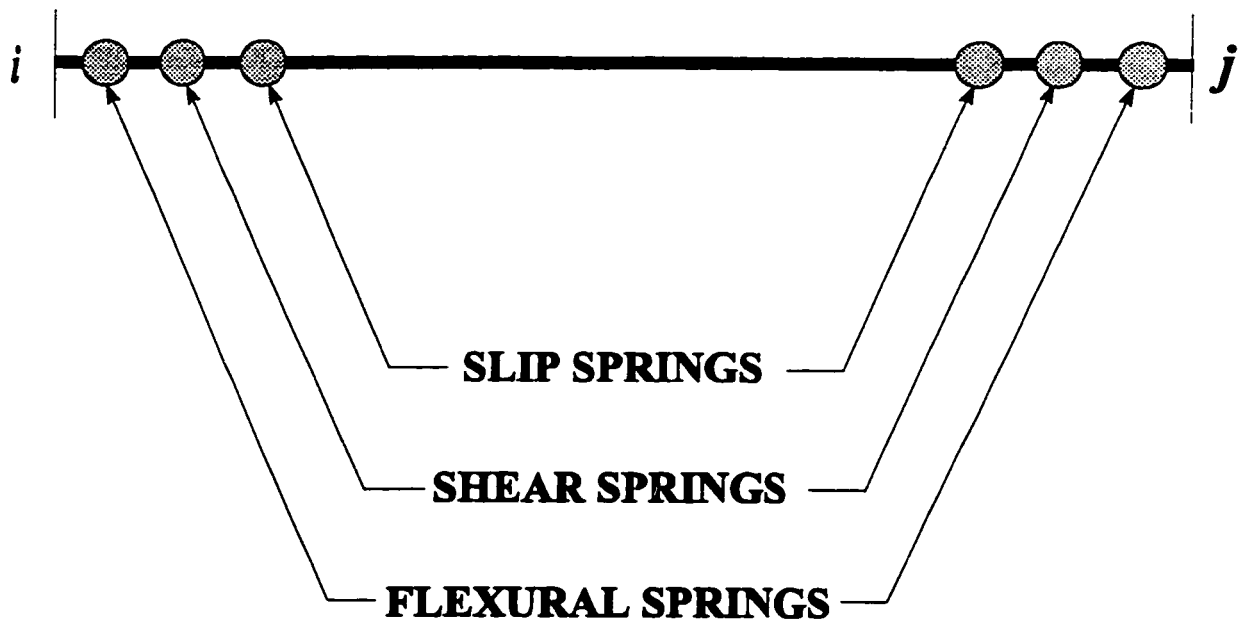


Fig. 2.2 Elastic Element with Three Springs.

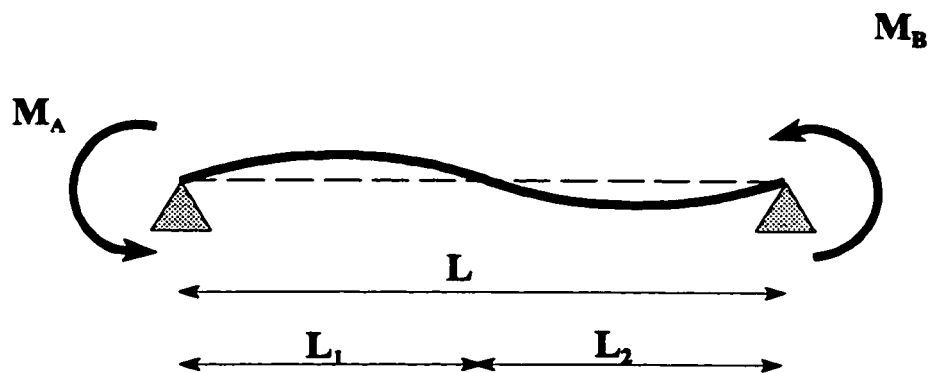
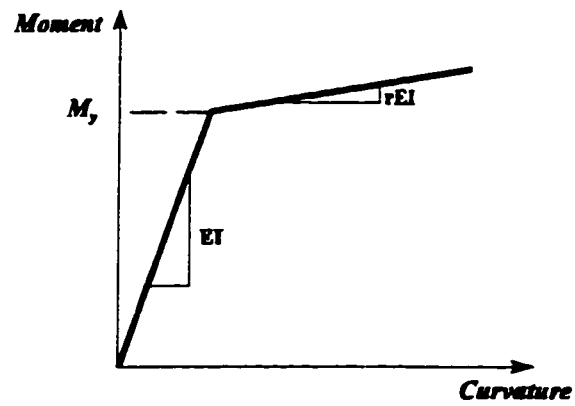
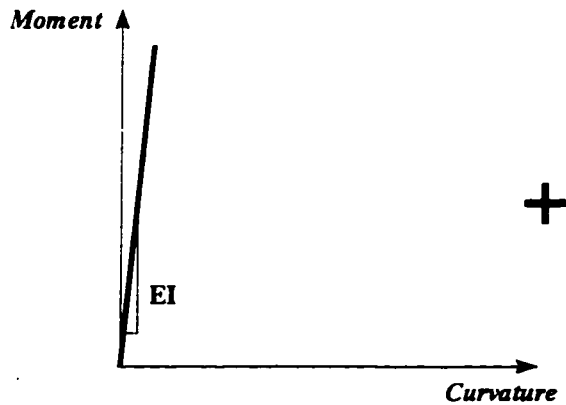


Fig. 2.3 Actual Beam Subjected to Different End Moments



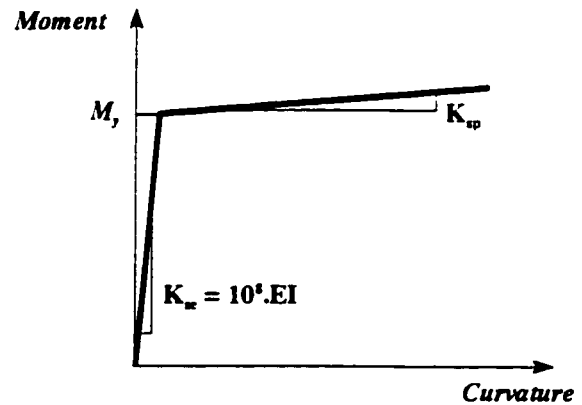
M- ϕ Relationship for Actual Beam

=



M- ϕ Relationship for Elastic Beam

+



M- ϕ Relationship for Flexural Spring

Fig. 2.4 Moment Curvature Relationship for Actual and Idealized Beam

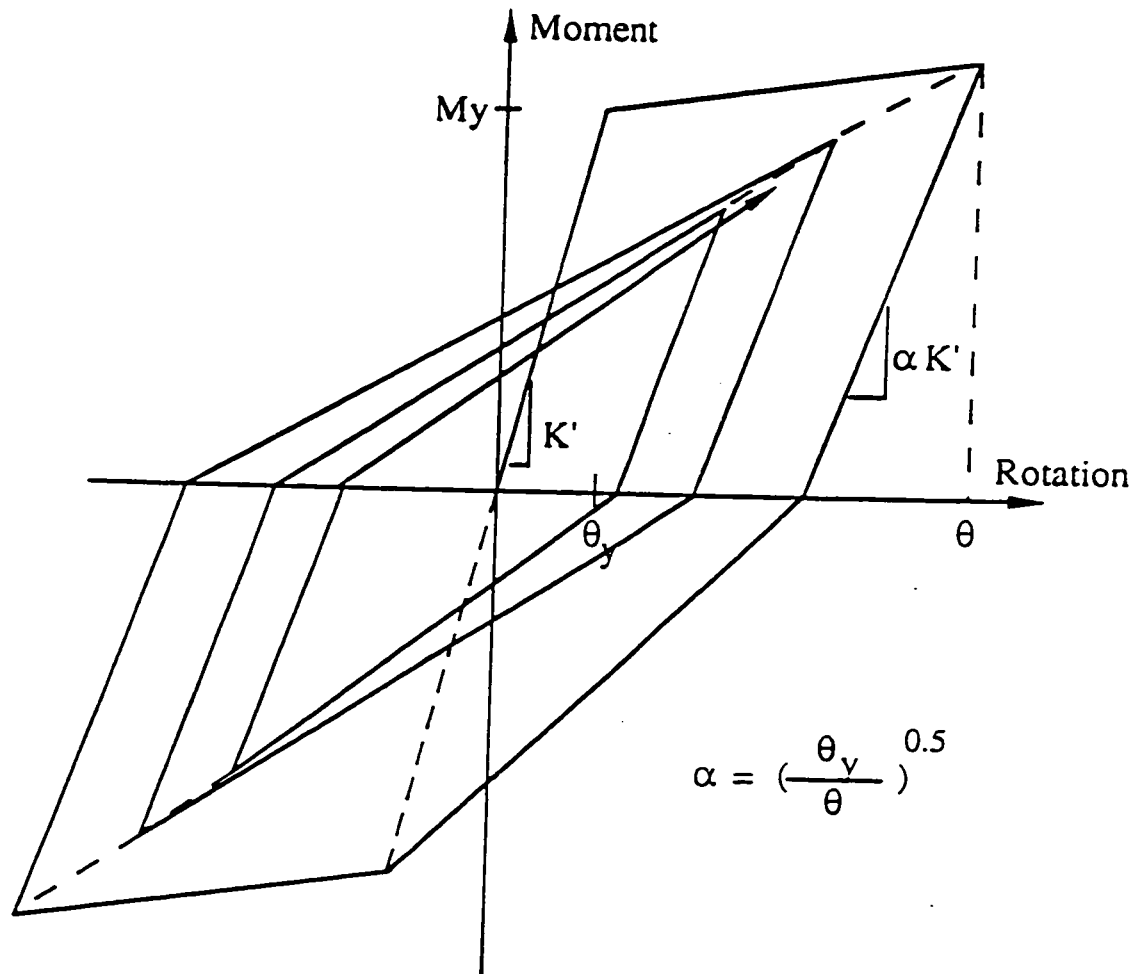
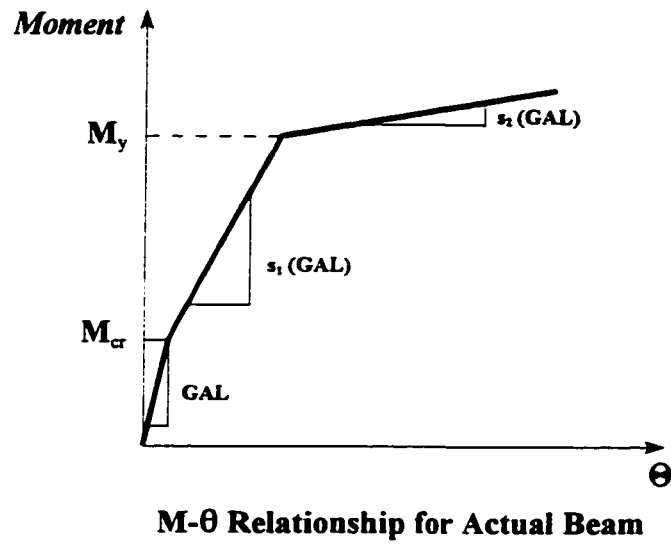
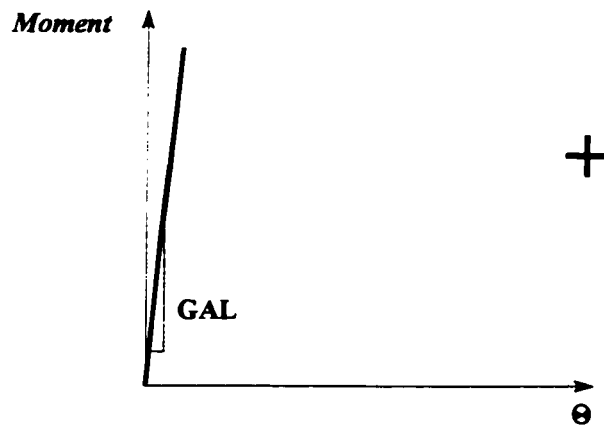


Fig. 2.5 Modified Takeda's Hysteretic Model [46]



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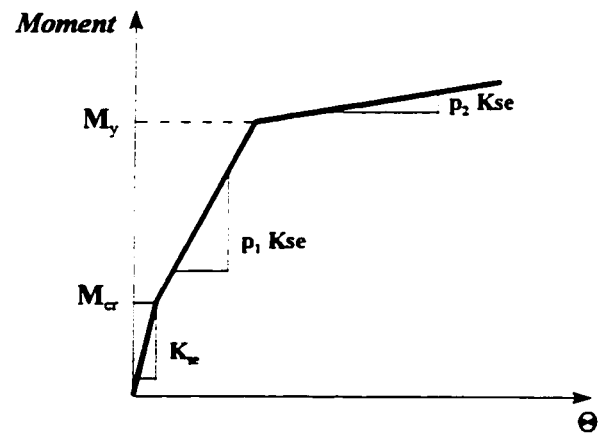


Fig. 2.6 Moment-Chord Rotation(due to shear) Relationship for Actual and Idealized Beam

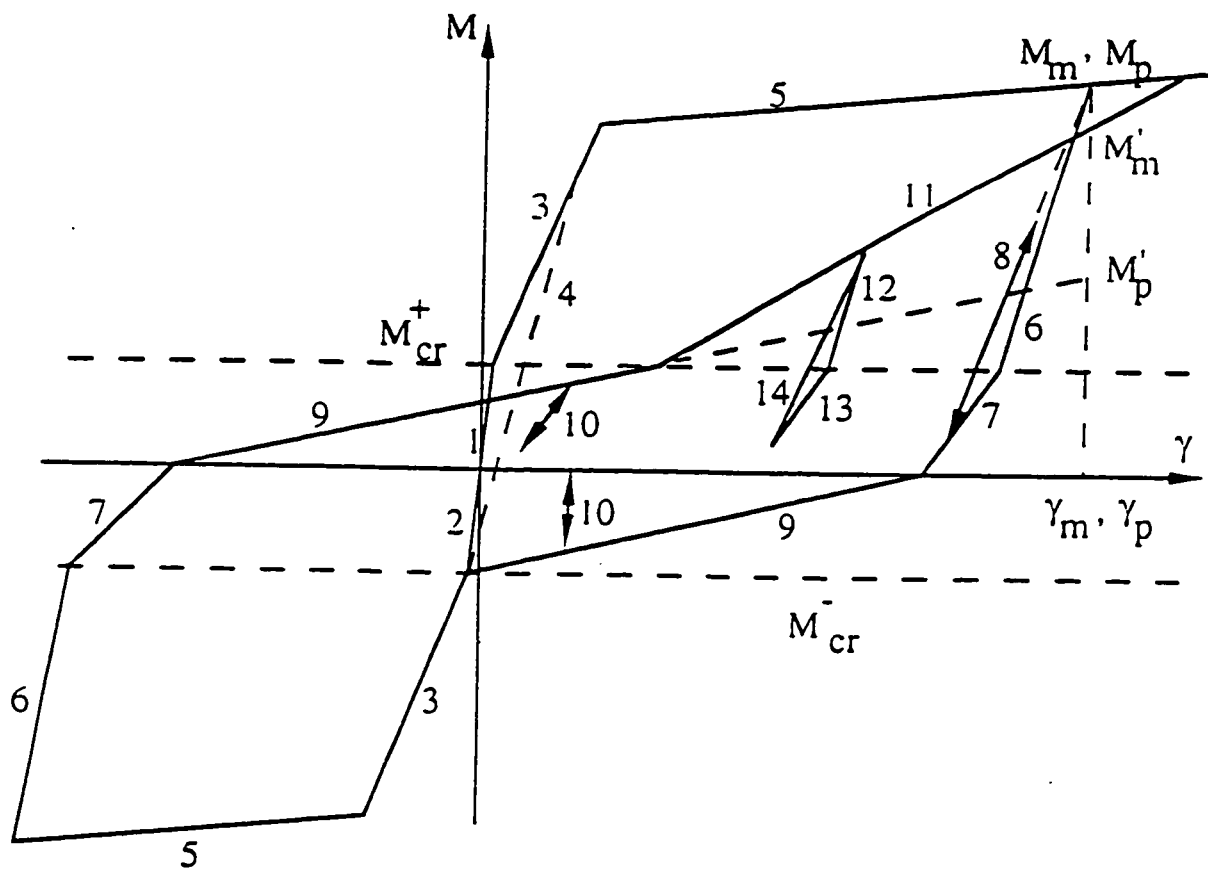


Fig. 2.7 Hysteretic Model for Shear [49]

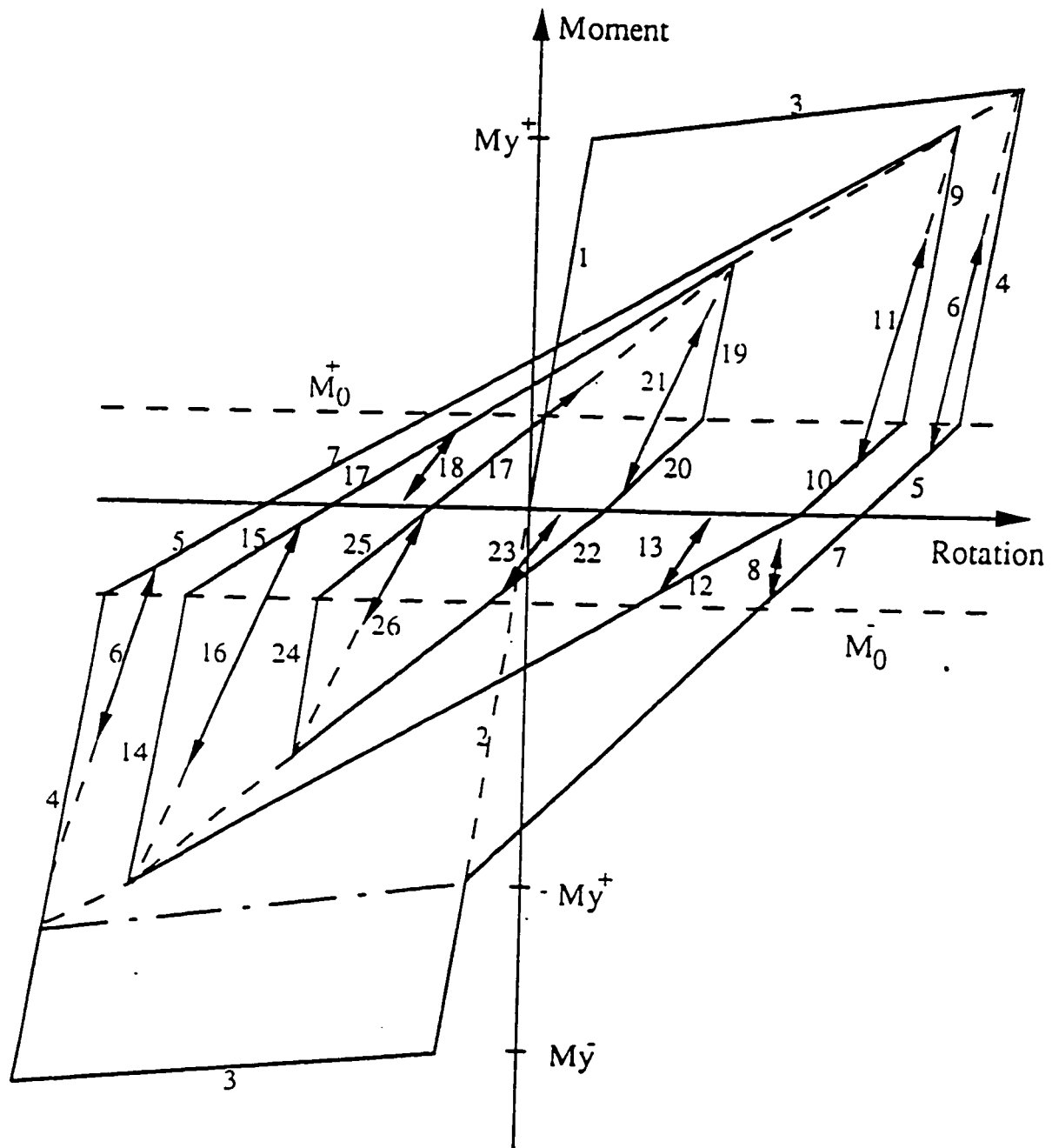


Fig. 2.8 Hysteretic Anchorage Slip Model [47]

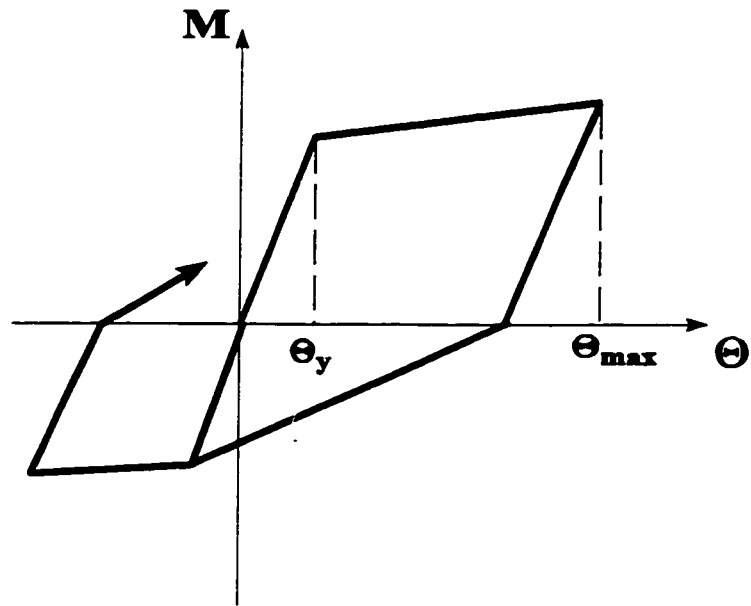


Fig. 2.9 Ductility Definition

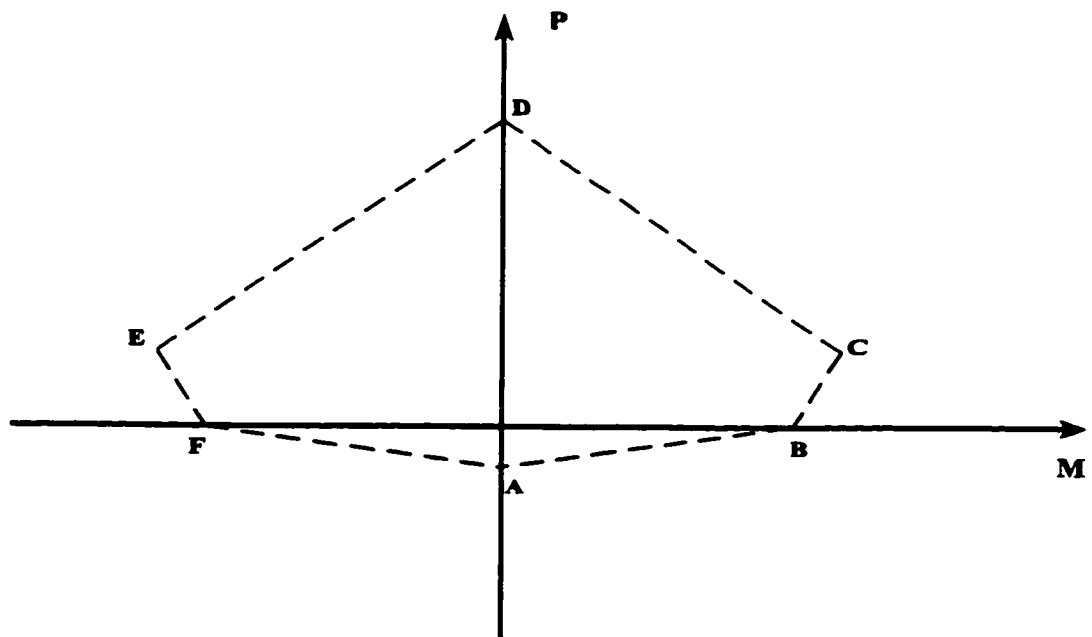


Fig. 2.10 Axial Force-Moment Interaction Diagram

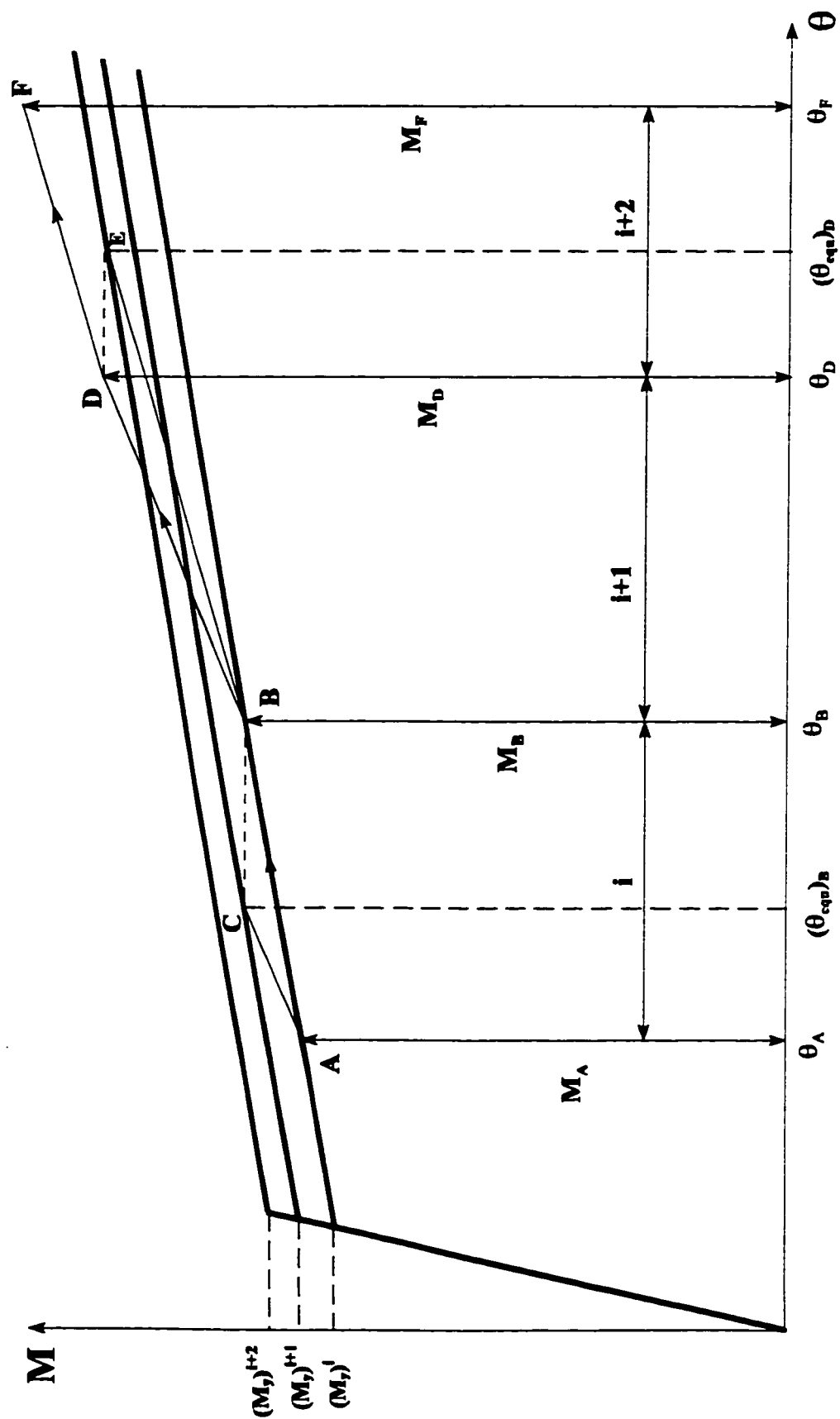


Fig. 2.11 Procedure of Re-Loading with P-M Interaction

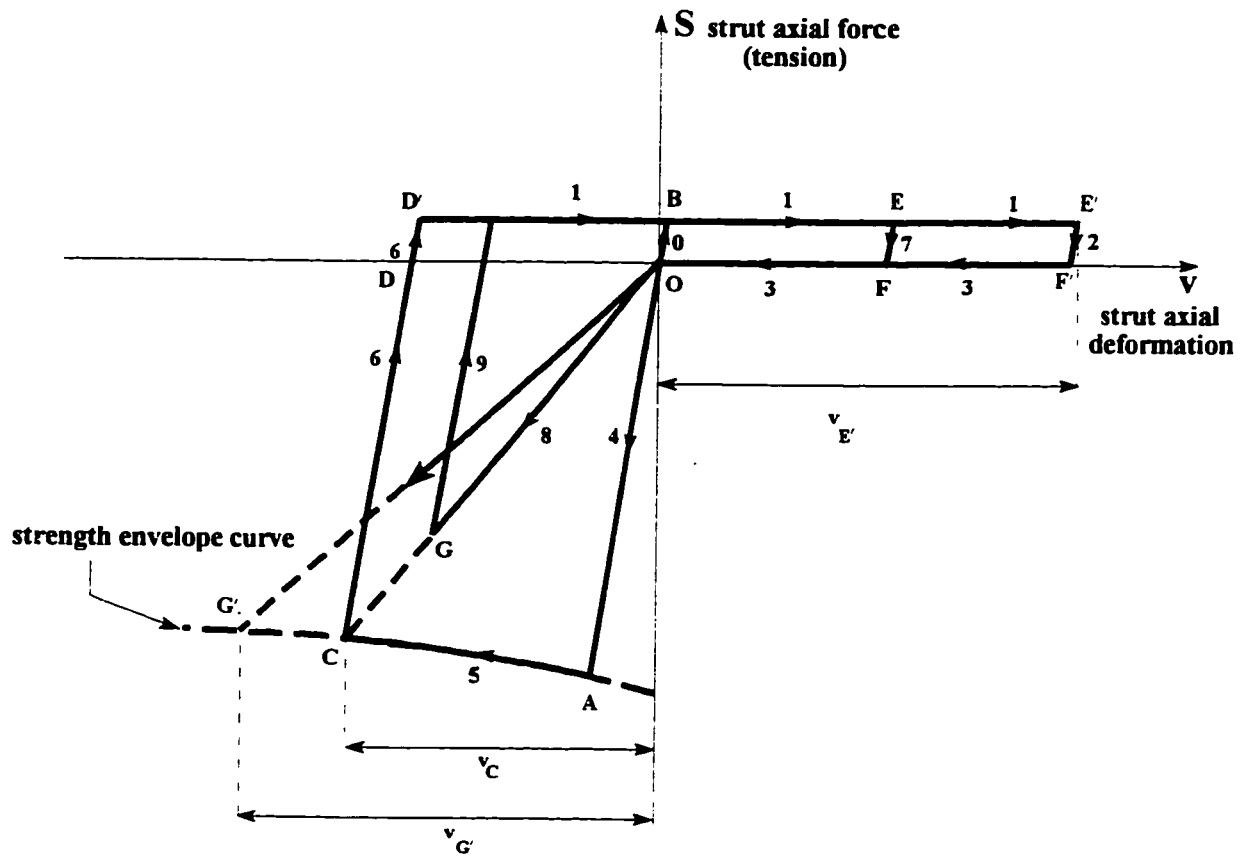


Fig. 2.12 Hysteretic Behavior of a Equivalent Trust Element (Infill Panel) [20,21]

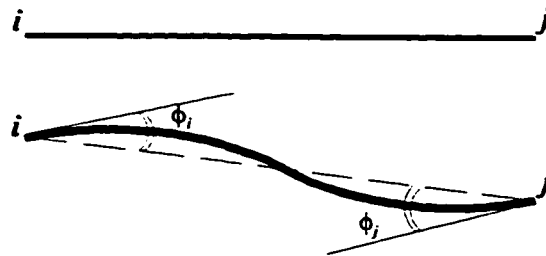
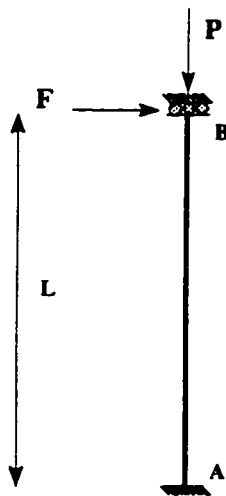
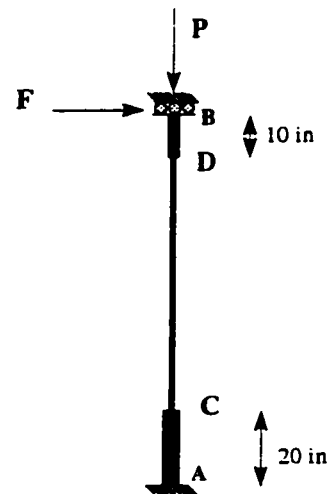


Fig. 2.13 Rotation at Ends of a Beam Element



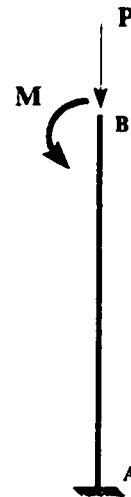
F=100 lb, P=1191.5 lb
Example 1.



F=100 lb, P=2431.5 lb
Example 2.



M=100 lb-in, P=298 lb
Example 3.



M=100 lb-in, P=608 lb
Example 4.

$L=100$ in , $A=1$ in² , $I=1/12$ in⁴ , $E=29 \times 10^6$ psi

Fig. 2.14 Verification Examples 1 to 4

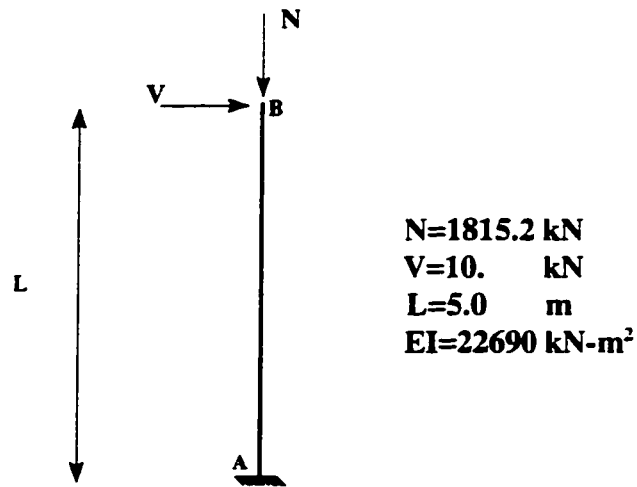


Fig. 2.15 Verification Example 5

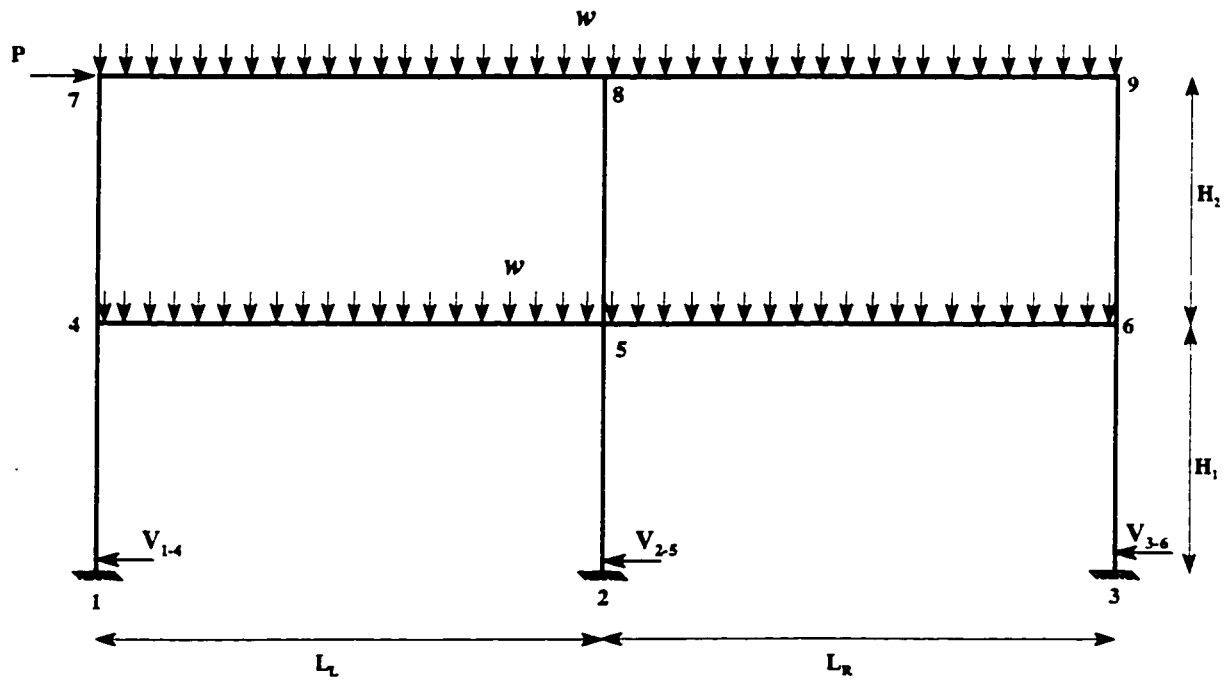
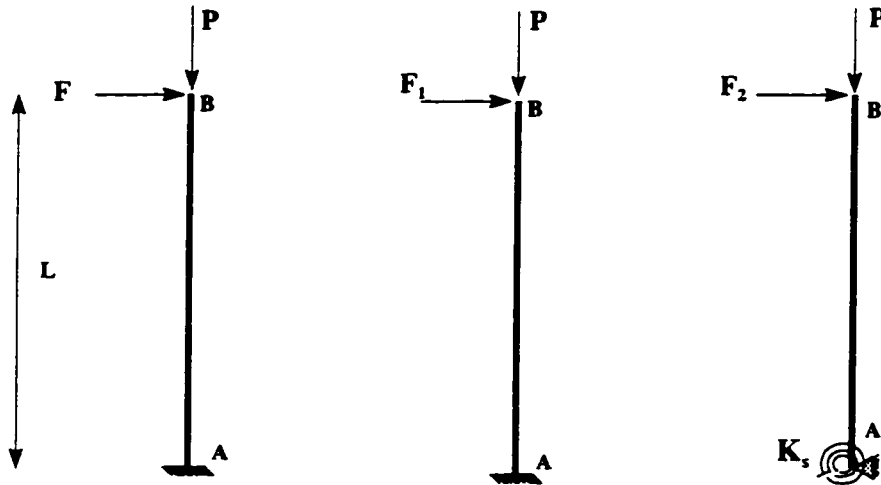


Fig. 2.16 Verification Example 6



$P=70$ kN
 $F=20$ kN
 $F_1=19.755$ kN
 $F_2=0.245$ kN
 $L=5.0$ m
 $EI=50000$ kN-m²
 $K_s=404.1$ kN-m

Fig.2.17 Verification Example. Single Column with Inelastic Behaviour

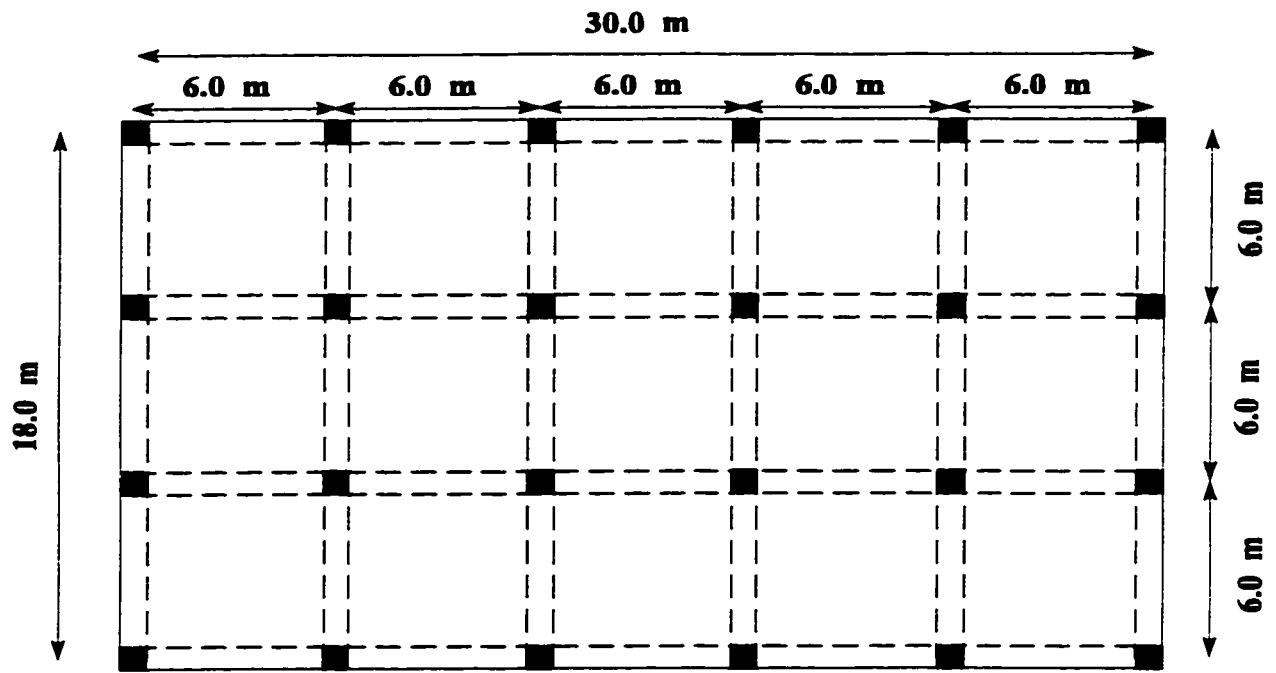


Fig. 3.1 Plan of Moment Resisting Frame Building

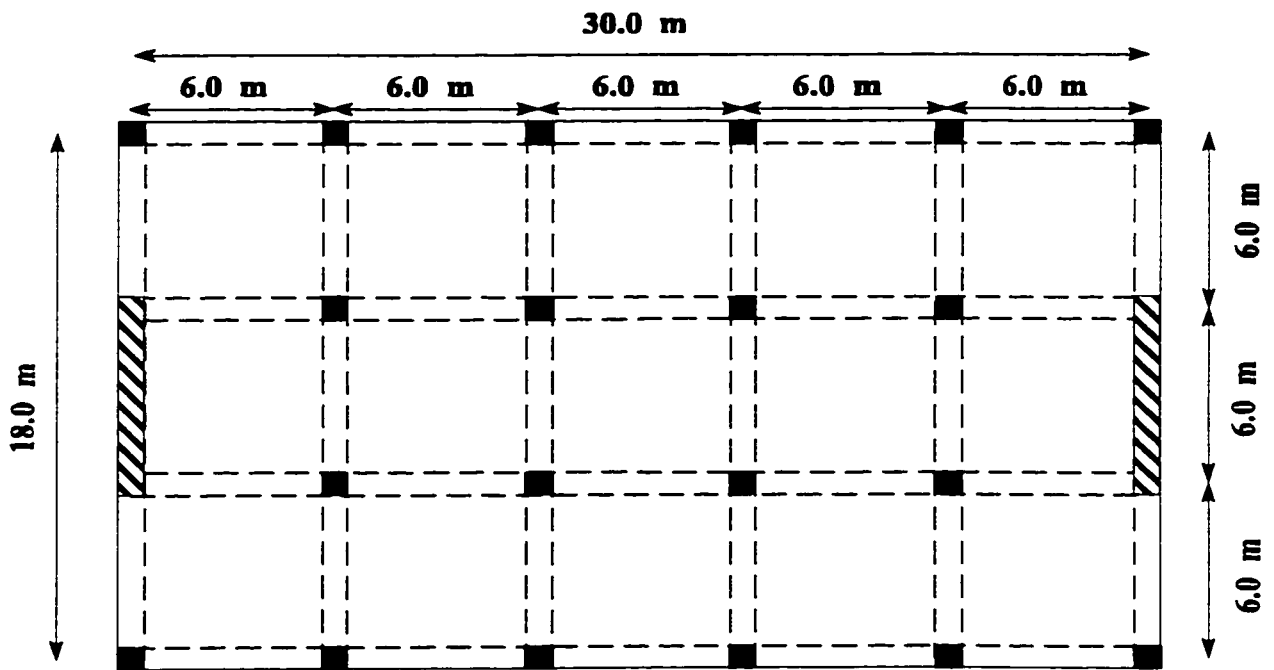


Fig. 3.2 Plan of Frame-Shear Wall Building

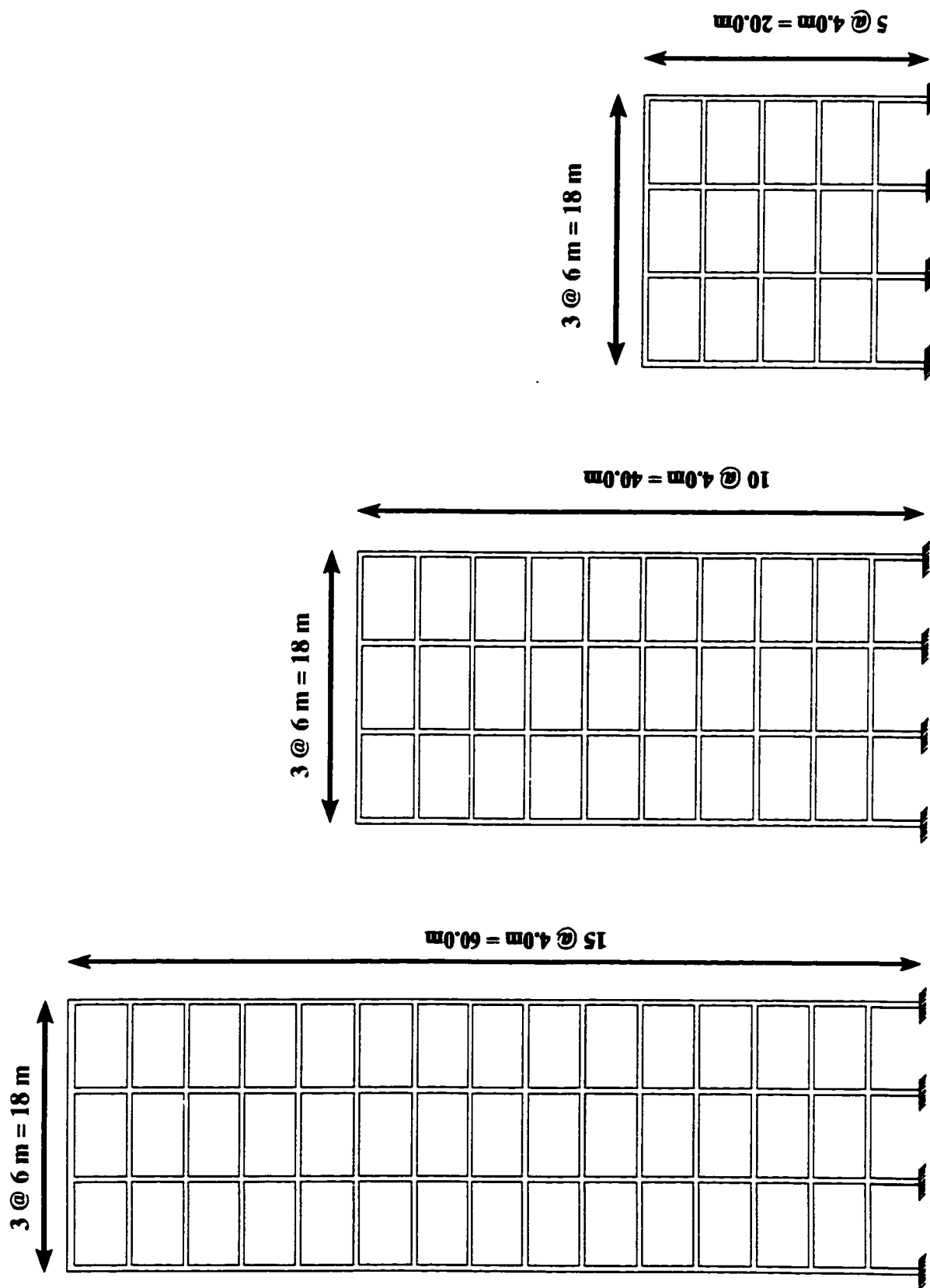


Fig. 3.3 Elevation View of 15, 10 and 5-Storey Frame Buildings without Infilled Masonry Panel

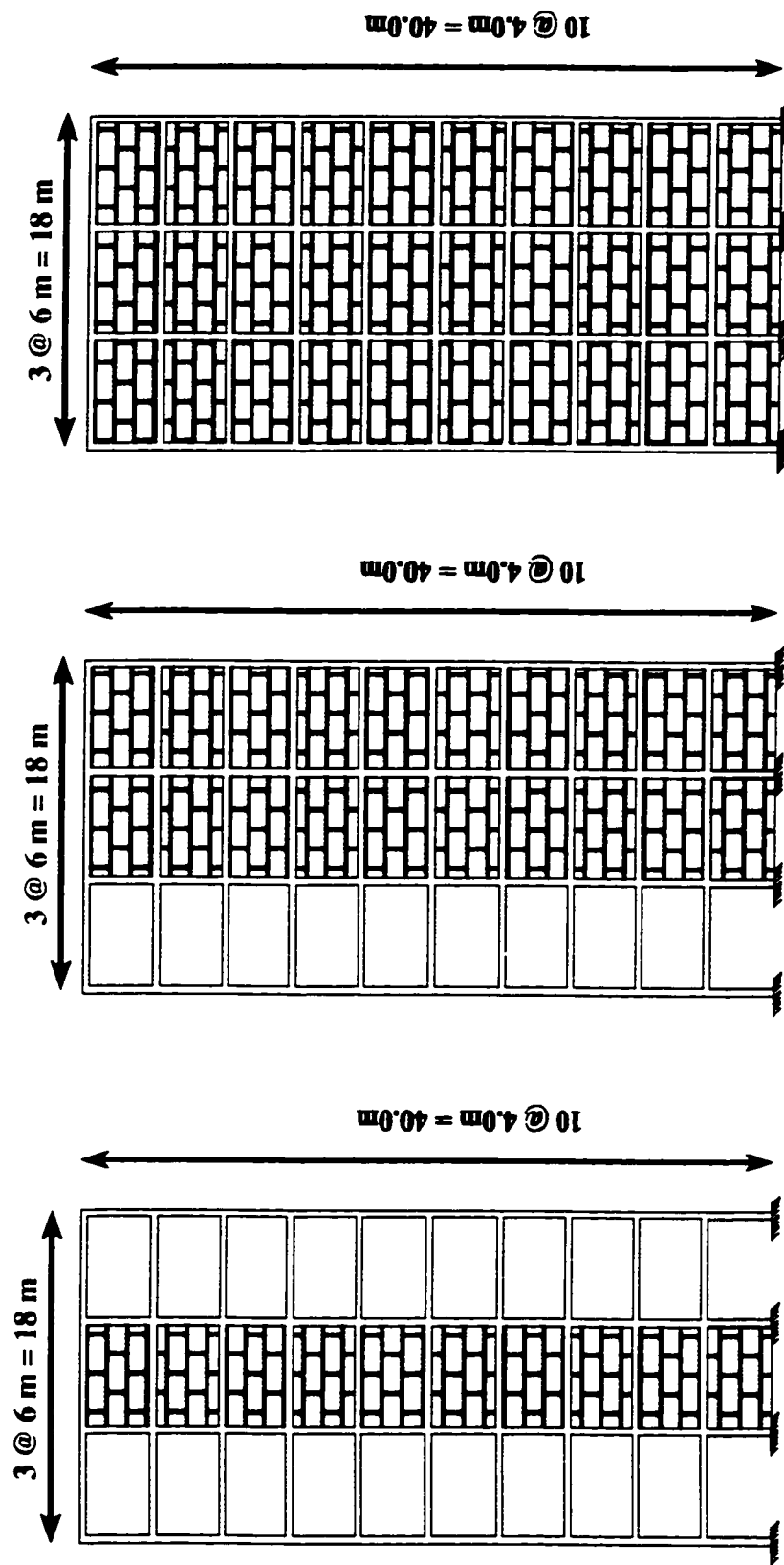


Fig. 3.4 Elevation View of 10-Storey Frame Building with 1,2 and 3 Bays Covered by Infilled Masonry Panel

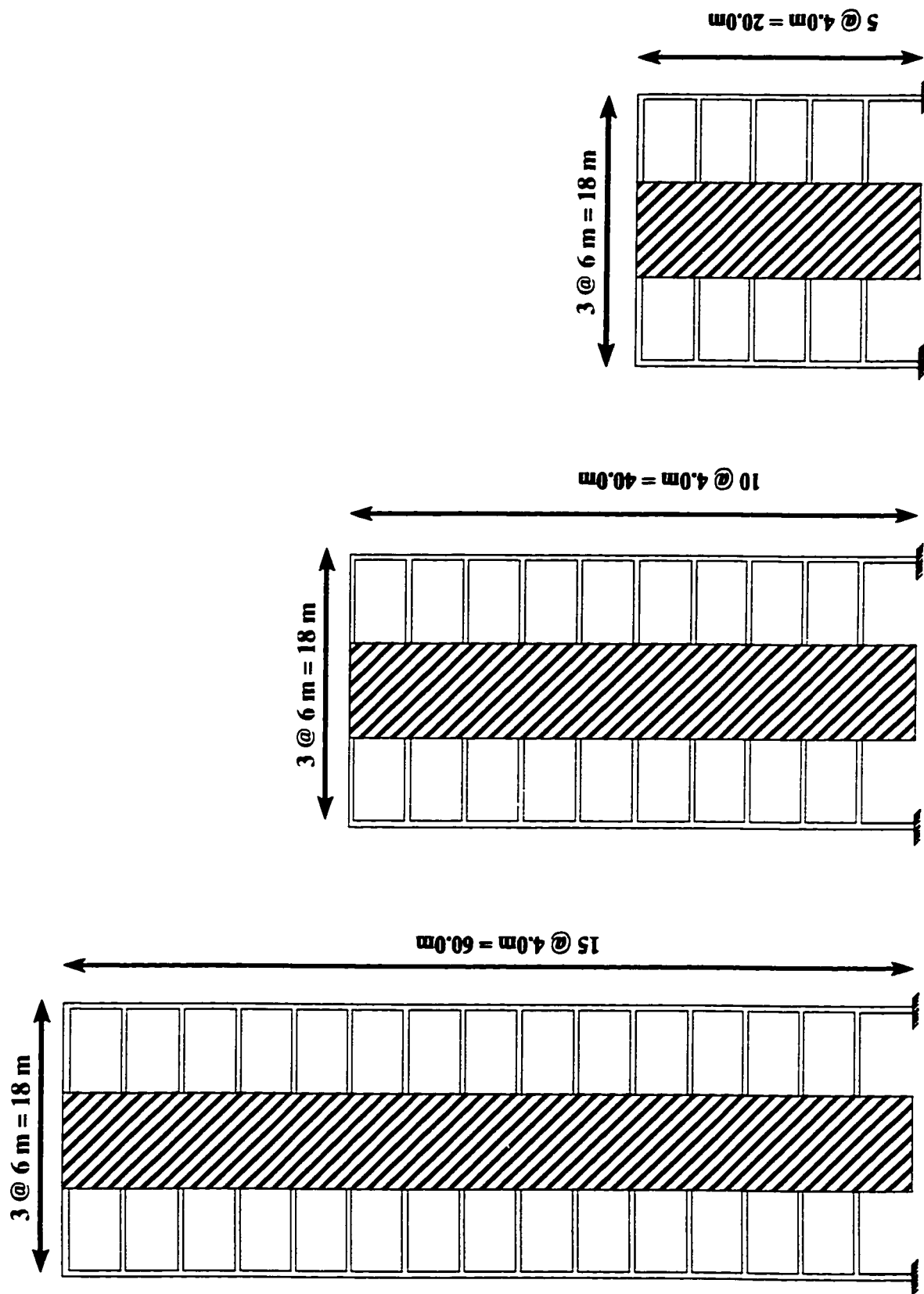


Fig. 3.5 Elevation View of 15, 10 and 5-Storey Frame-Shear Wall Buildings

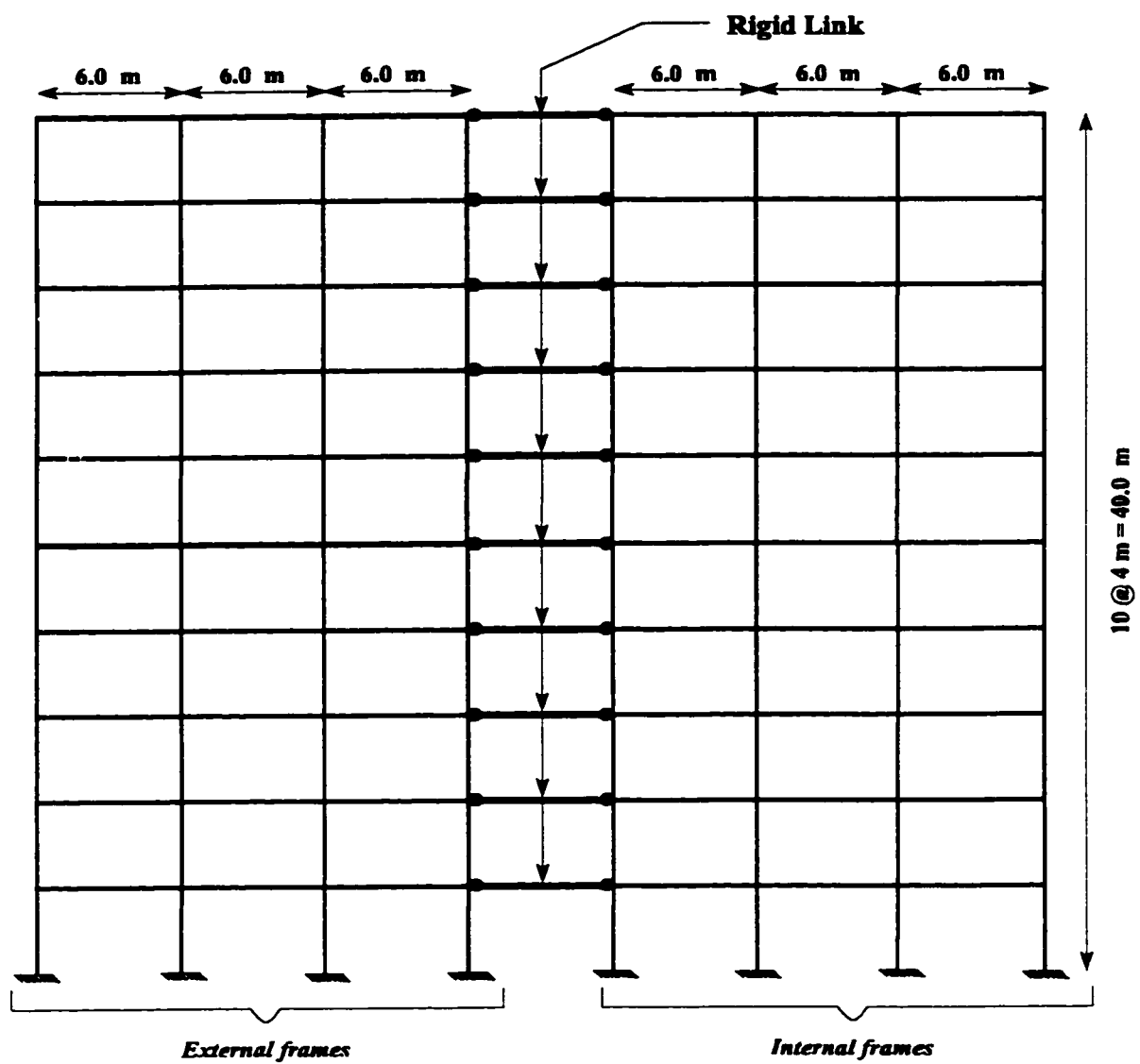


Fig. 3.6 Two Dimensional Lumped Frame Used in Analysis

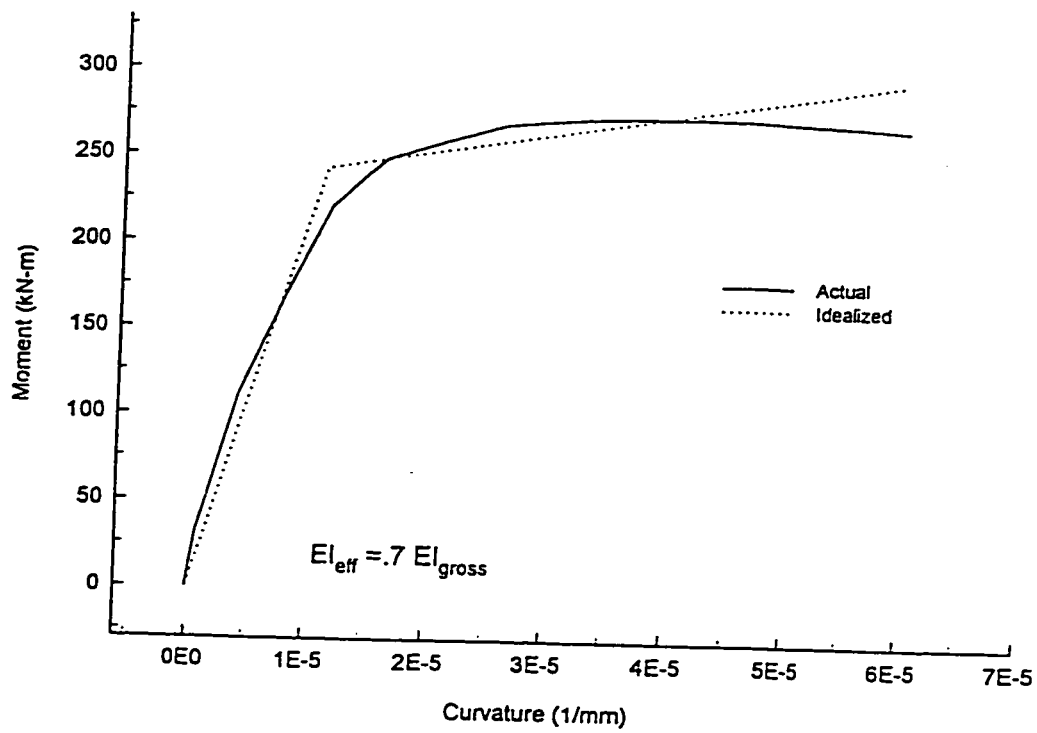


Fig. 3.7 Idealized Moment-Curvature Relationship

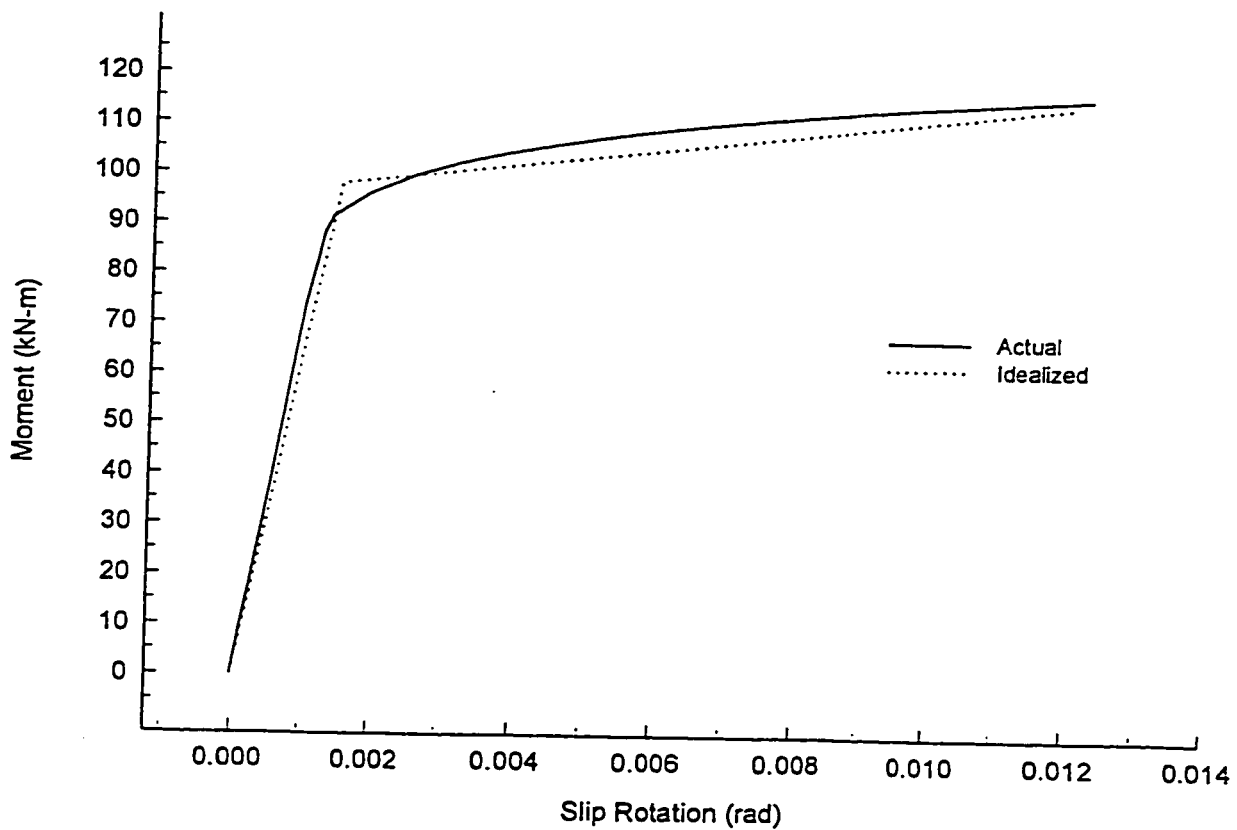


Fig. 3.8 Idealized Moment-Slip Rotation Relationship

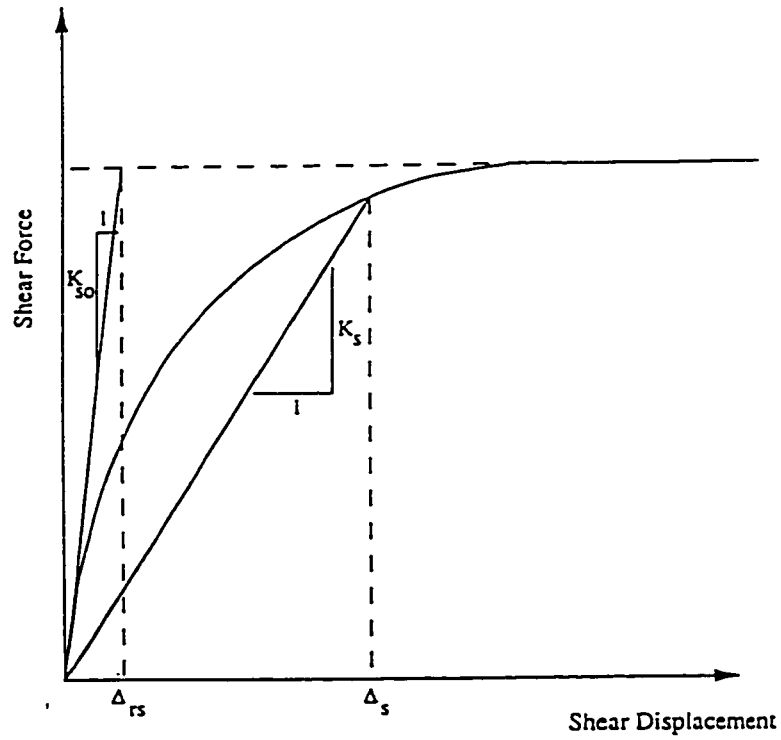


Fig. 3.9 Shear Force- Shear Displacement Relationship [61]

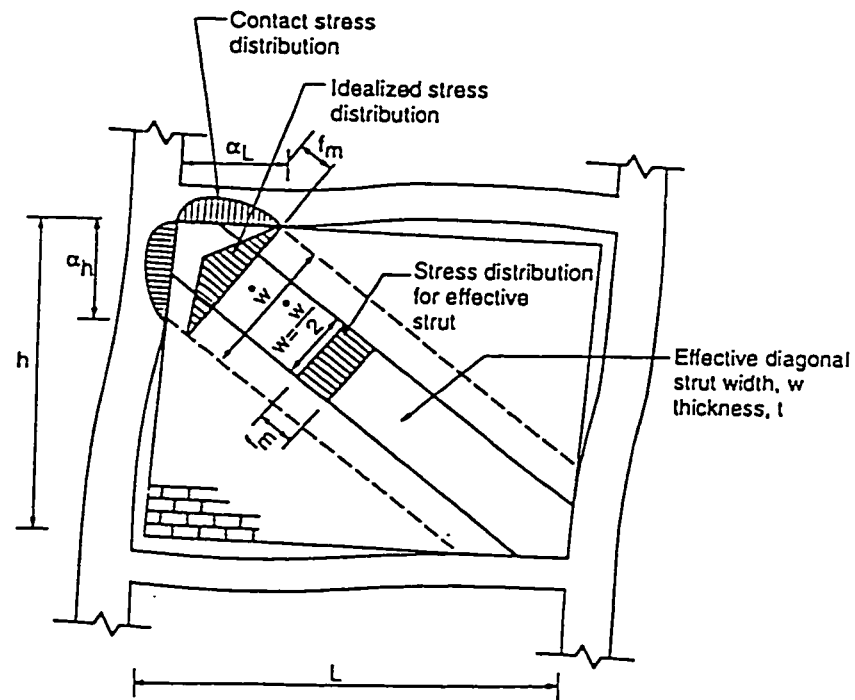


Fig. 3.10 Equivalent Diagonal Strut [65]

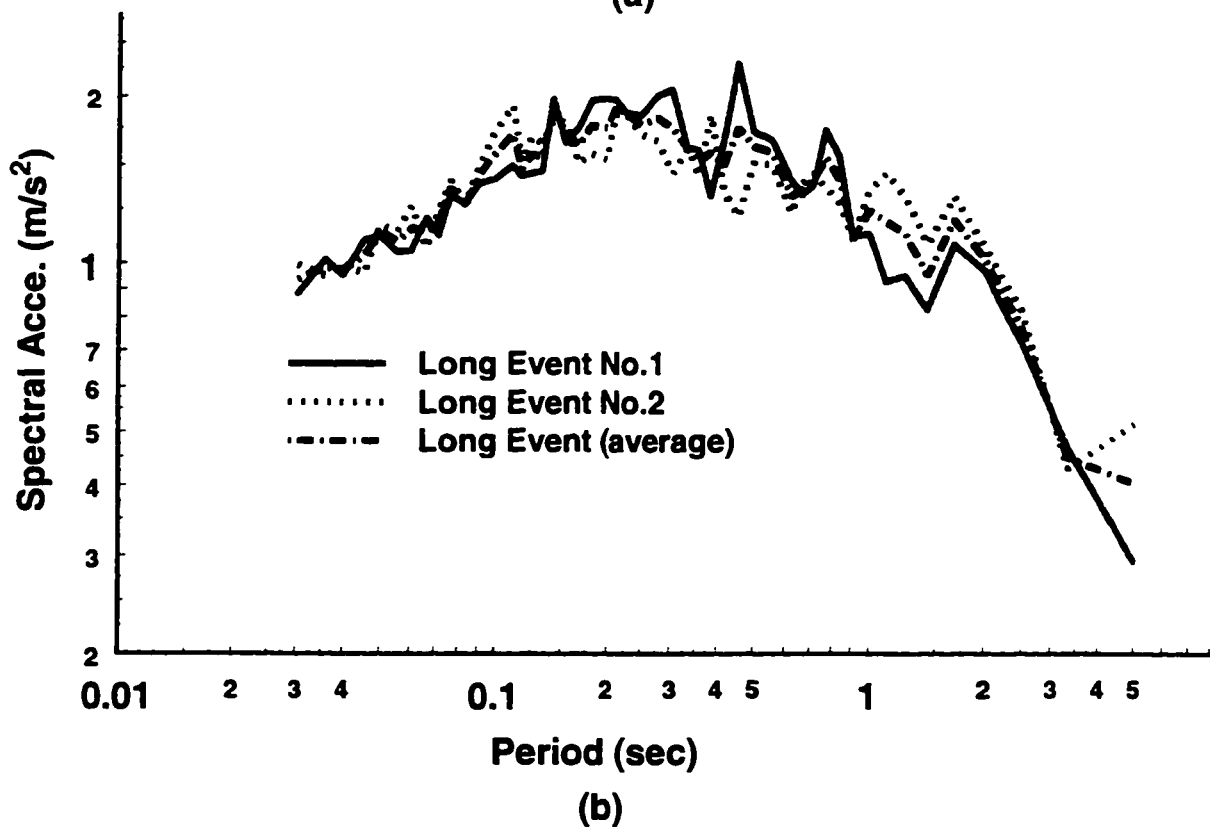
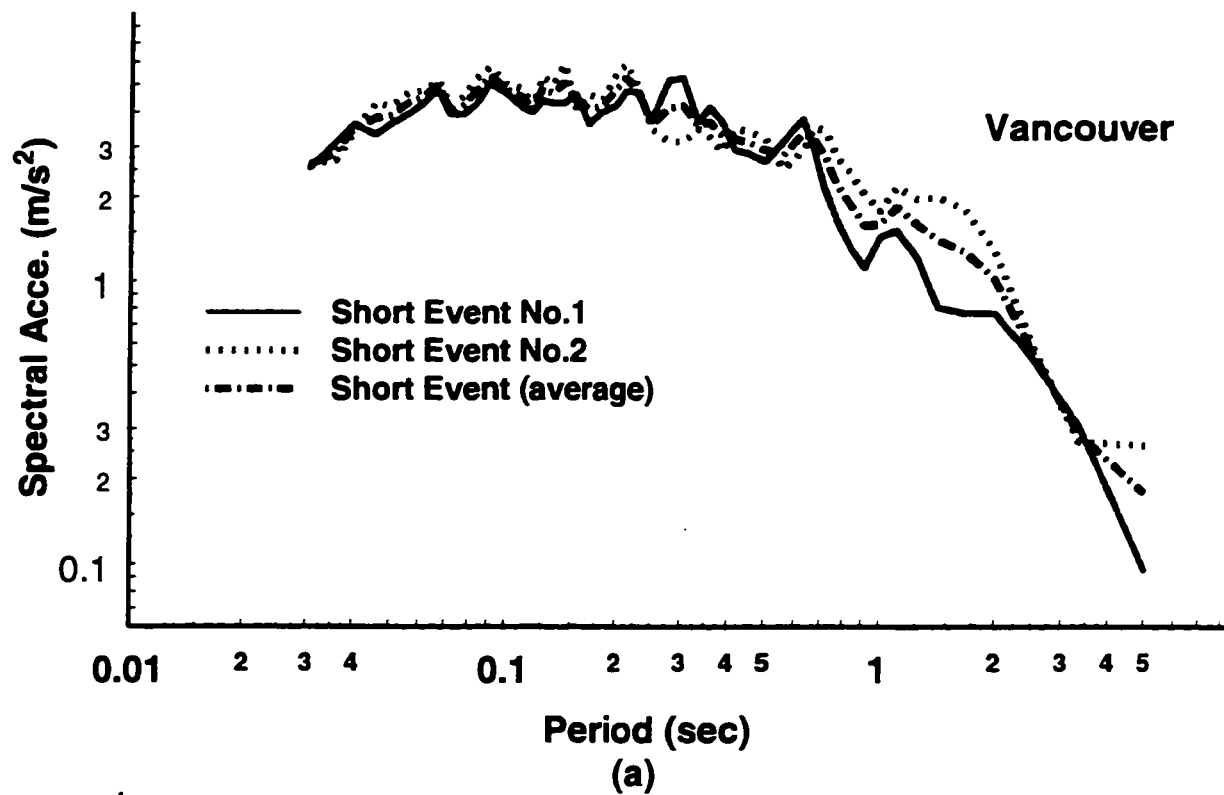


Fig. 3.11 Response Spectra of Artificial Records for Vancouver

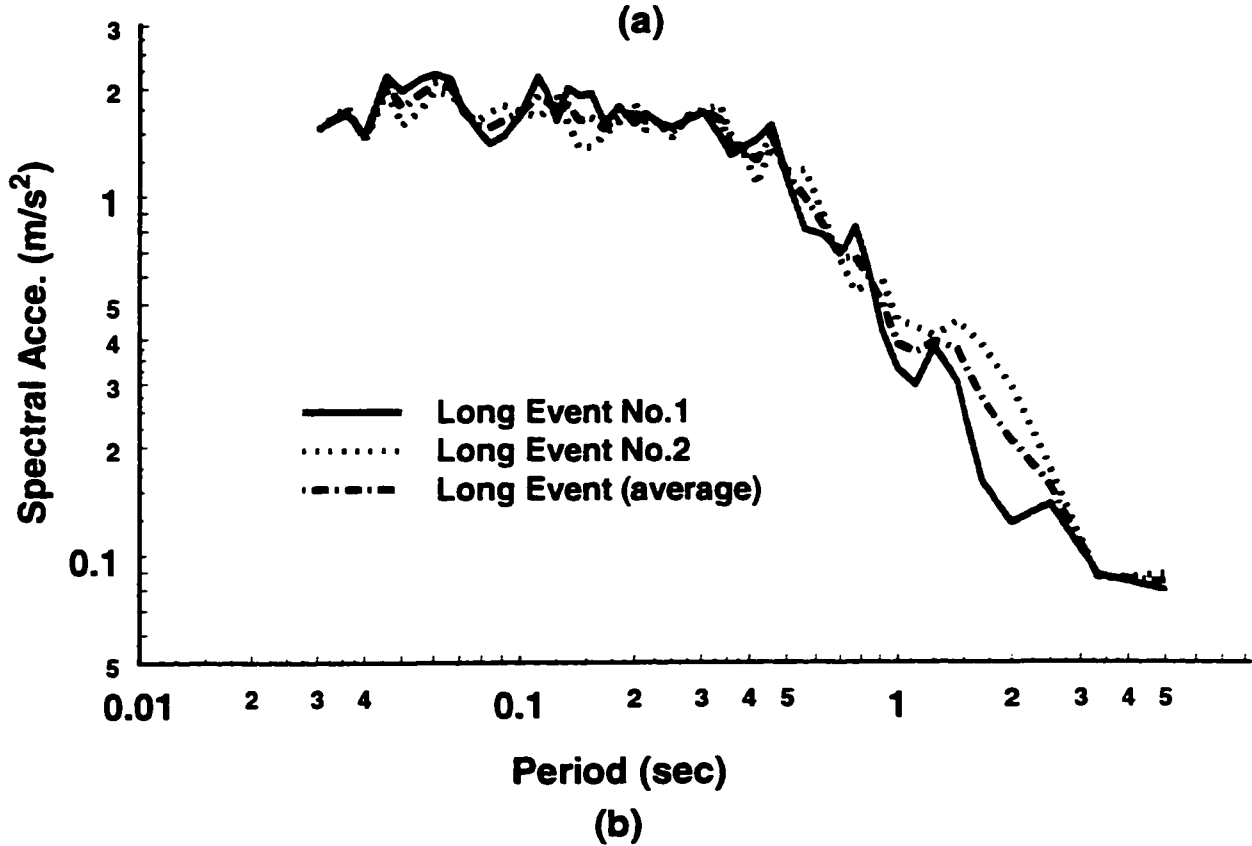
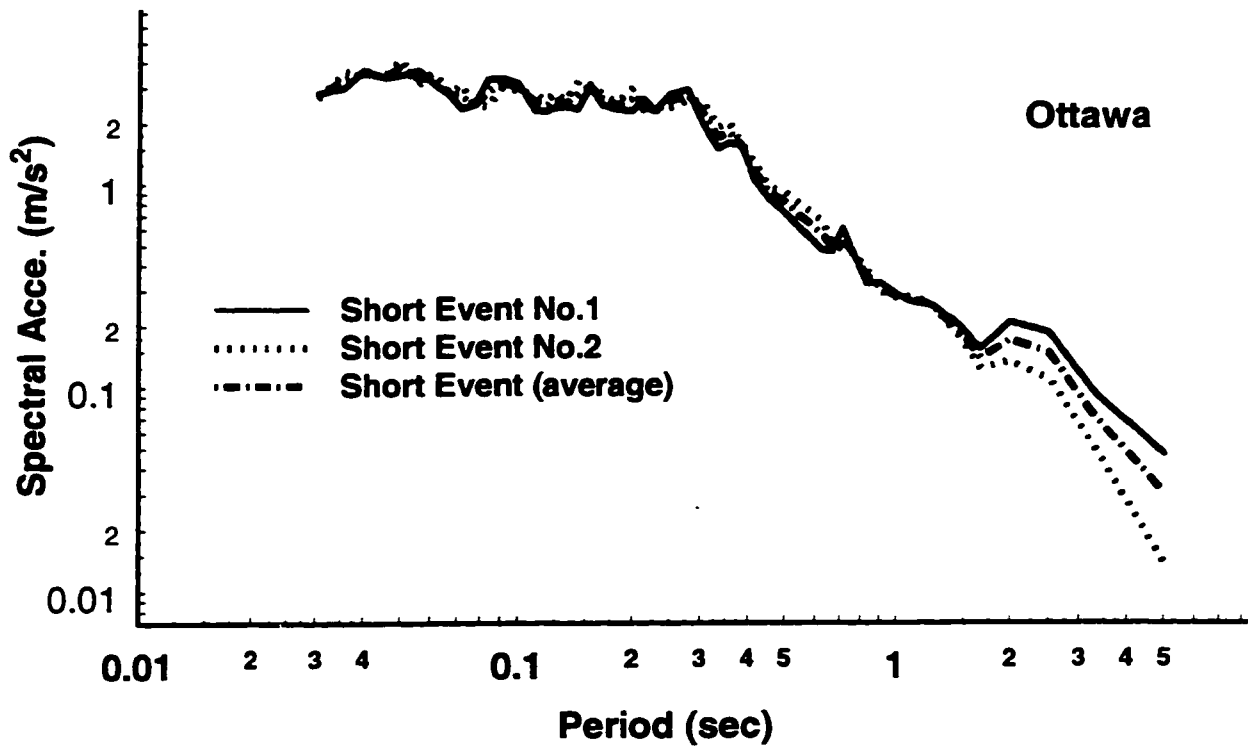


Fig. 3.12 Response Spectra of Artificial Records for Ottawa

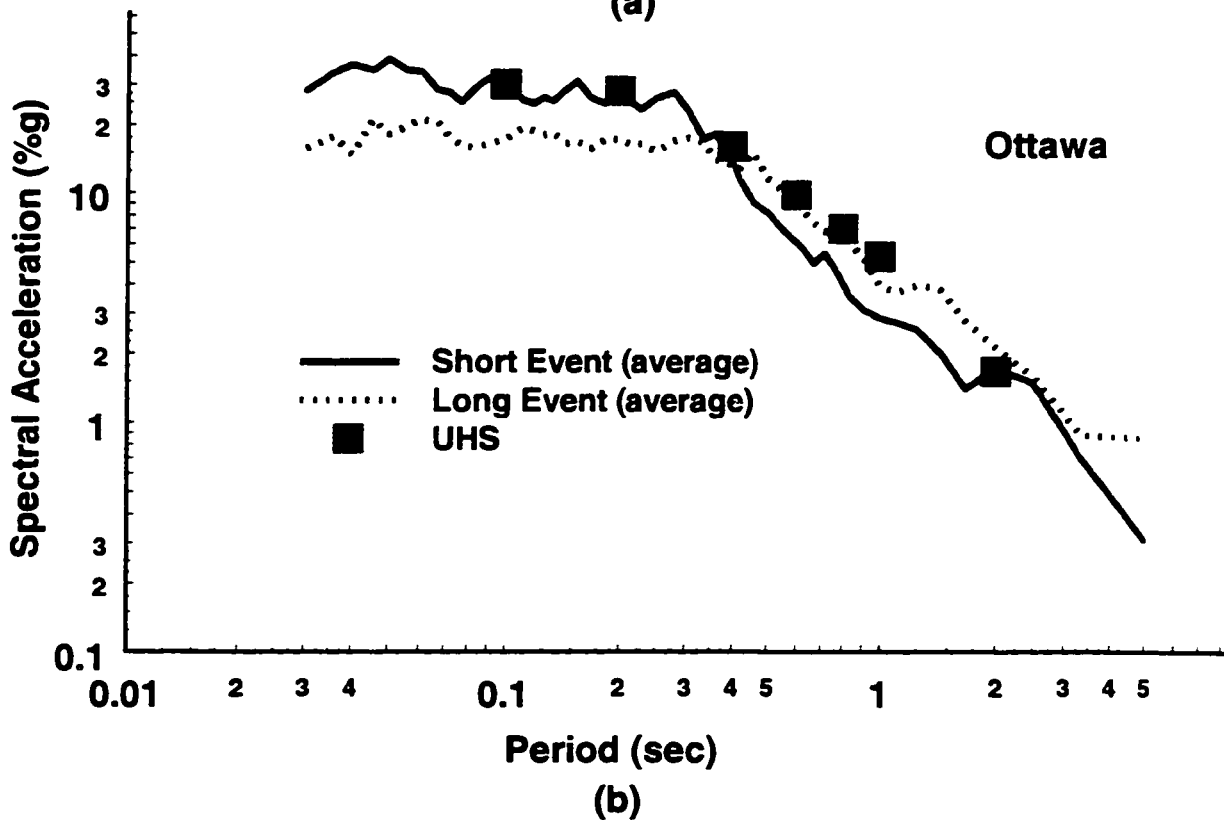
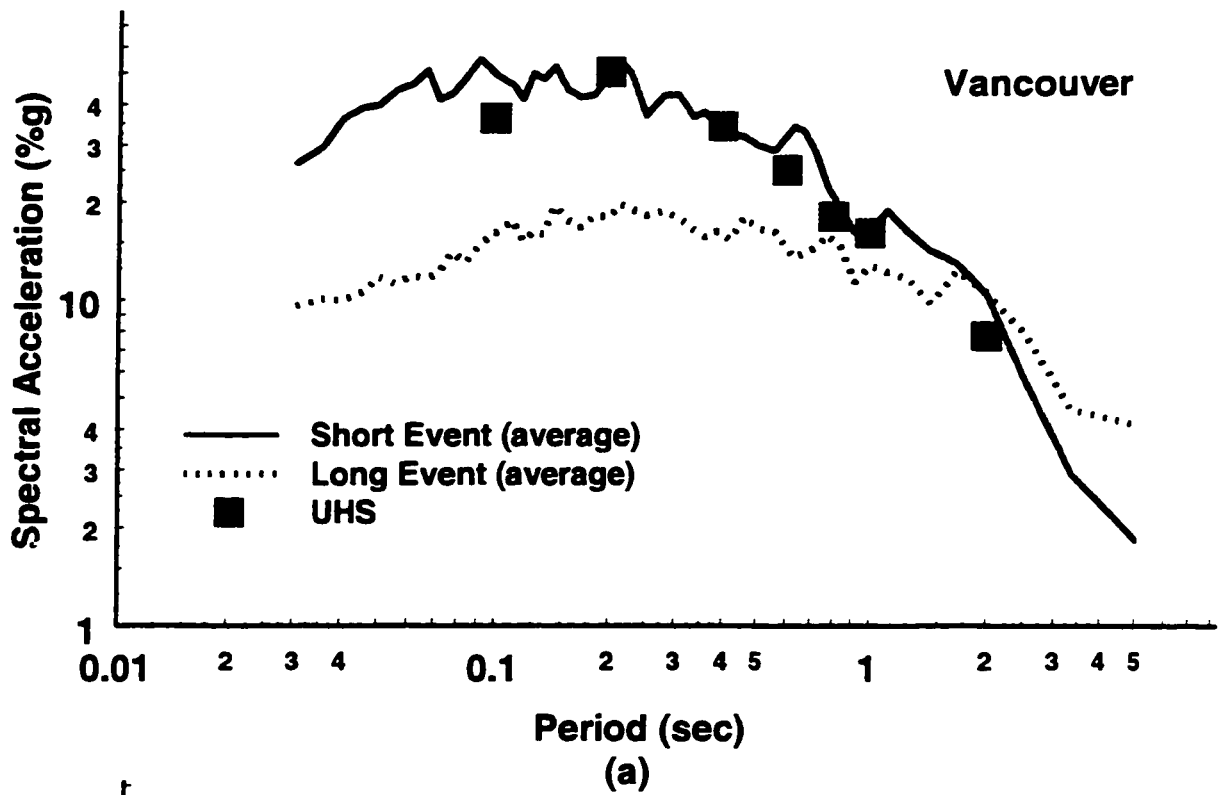


Fig. 3.13 Comparison of Response Spectra of Artificial Records and UHS for Vancouver and Ottawa

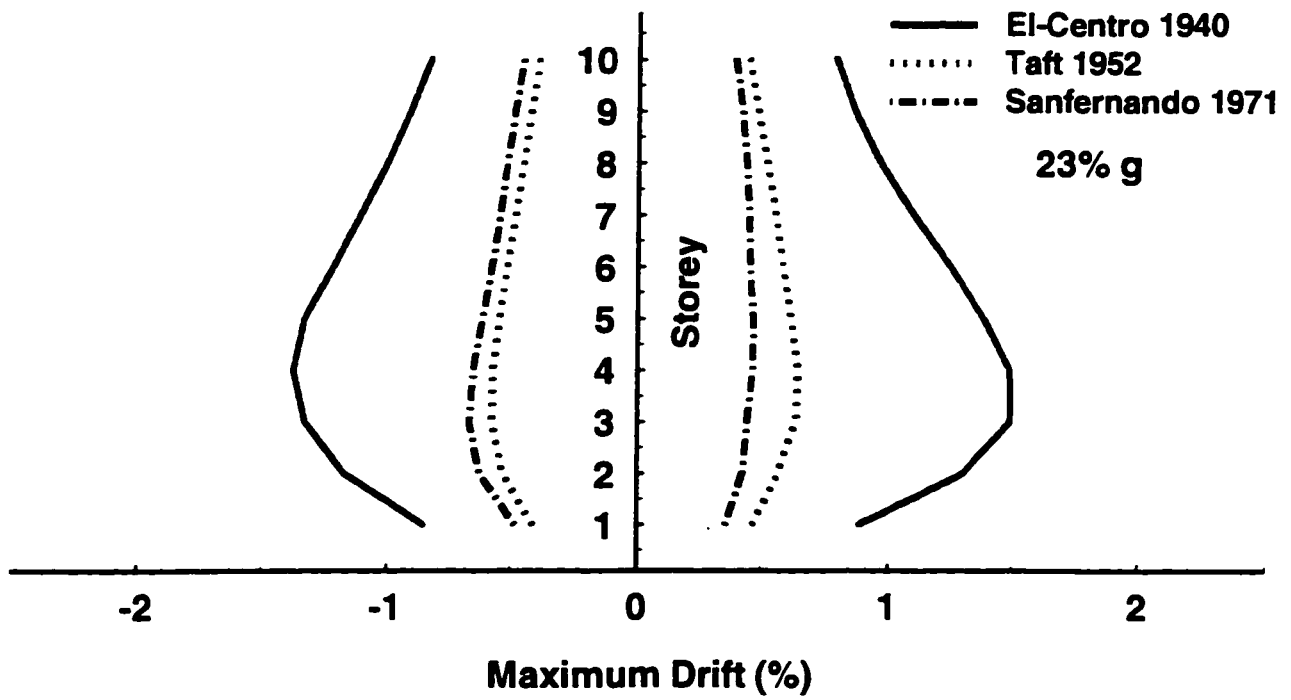


Fig. 4.1 Drift Response of 10-Storey MRF Subjected to Selected Records for Western Canada

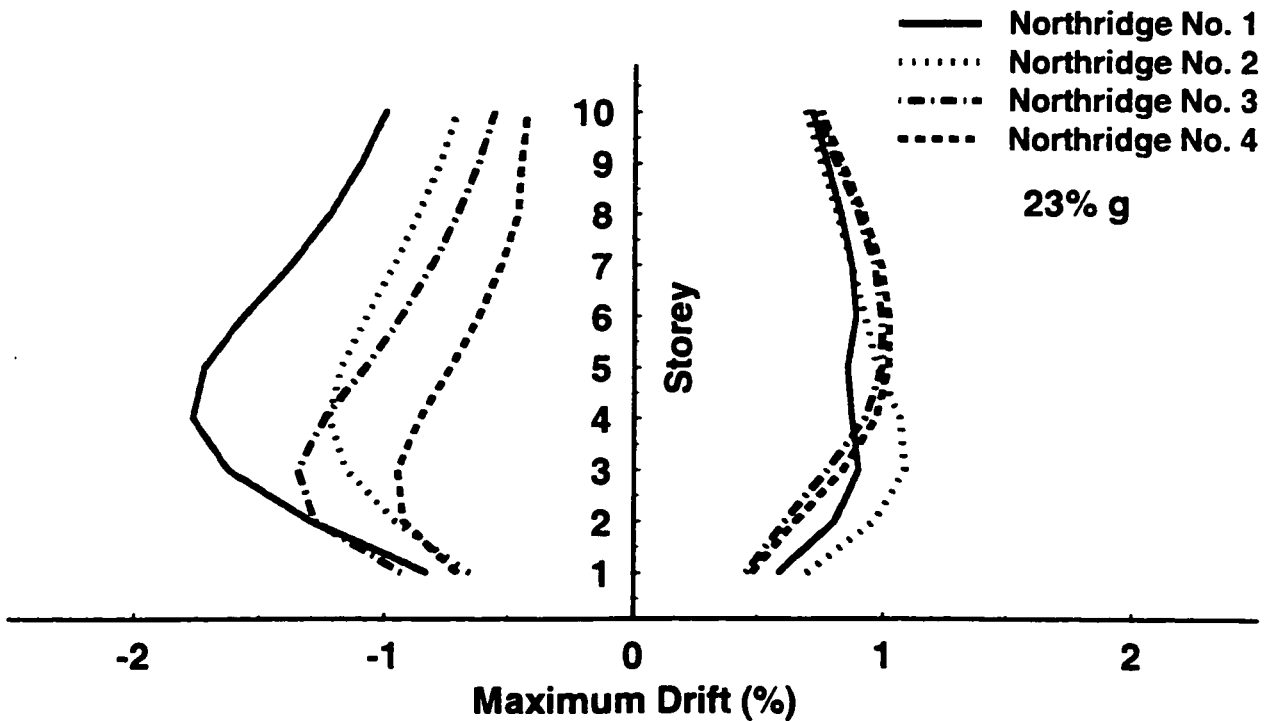


Fig. 4.2 Drift Response of 10-Storey MRF Subjected to 1994 Northridge Records for Western Canada

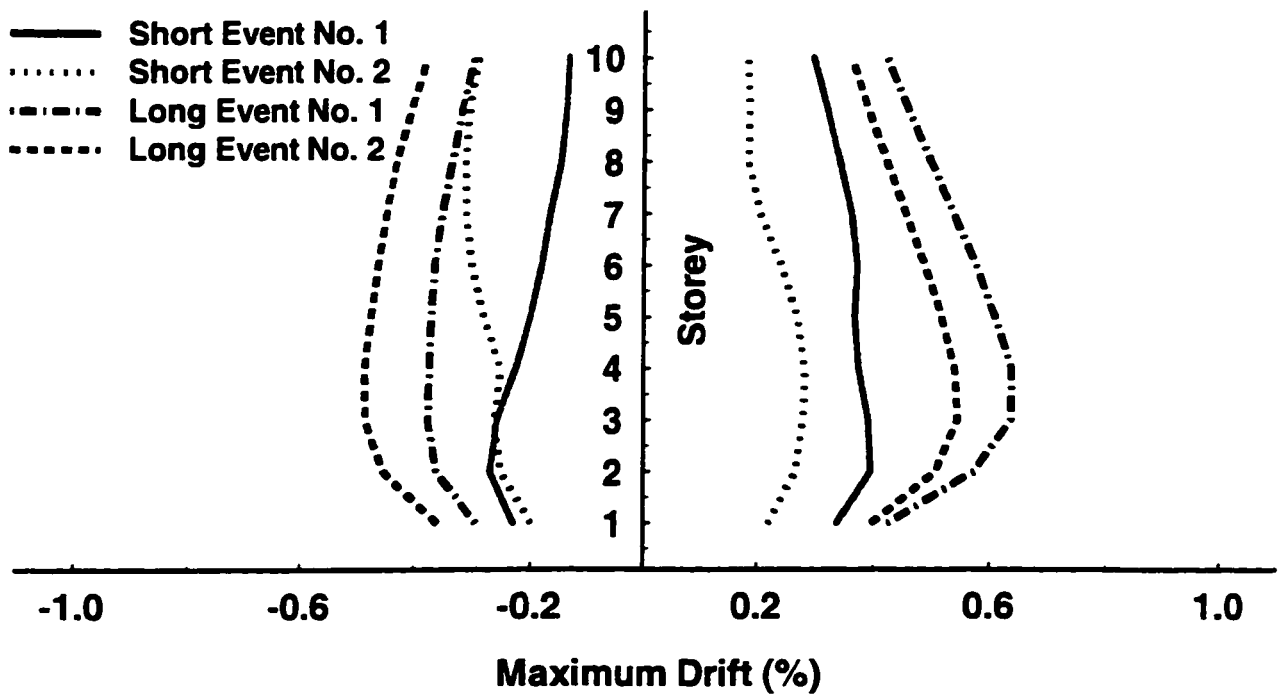


Fig. 4.3 Drift Response of 10-Storey MRF Subjected to Artificial Records for Western Canada

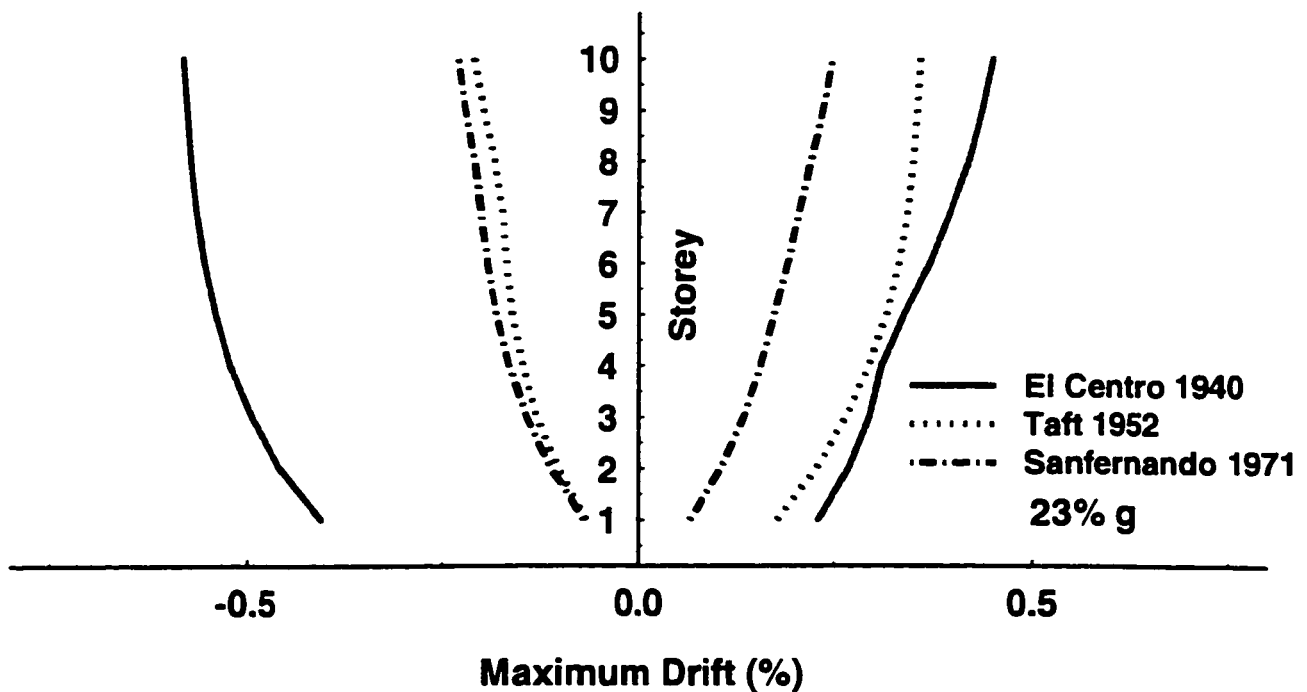


Fig. 4.4 Drift Response of 10-Storey SHW Subjected to Selected Records for Western Canada

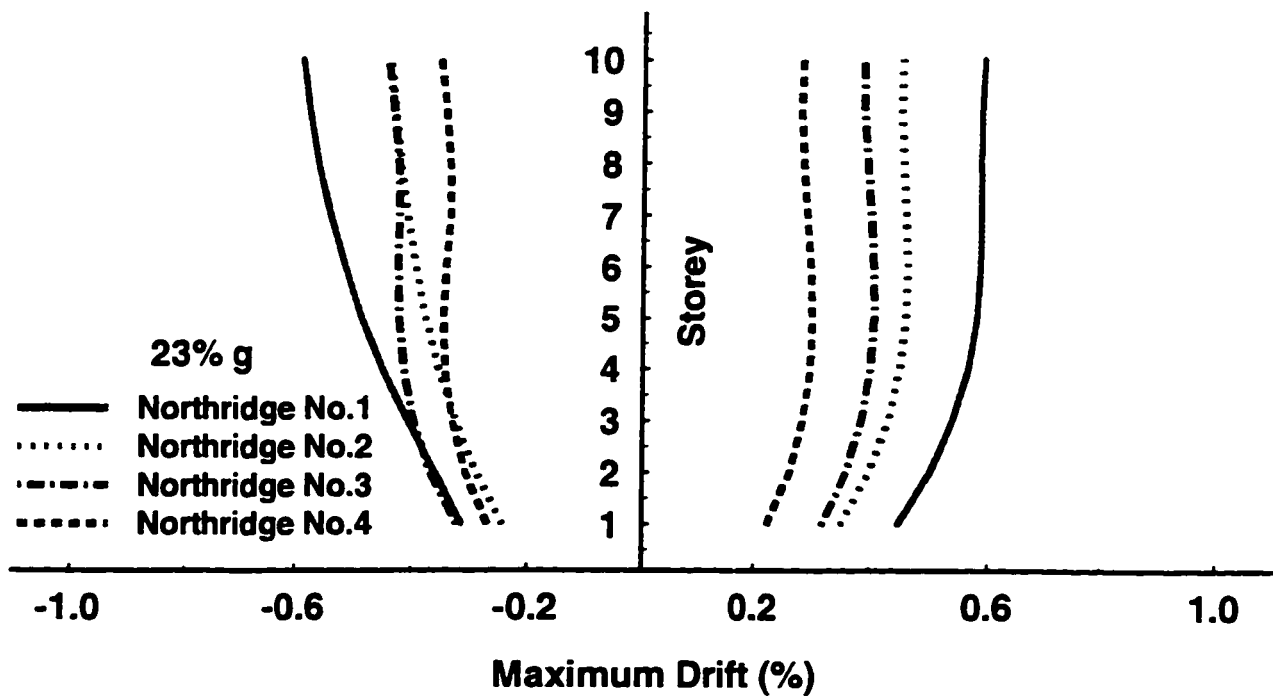


Fig. 4.5 Drift Response of 10-Storey SHW Subjected to 1994 Northridge Records for Western Canada

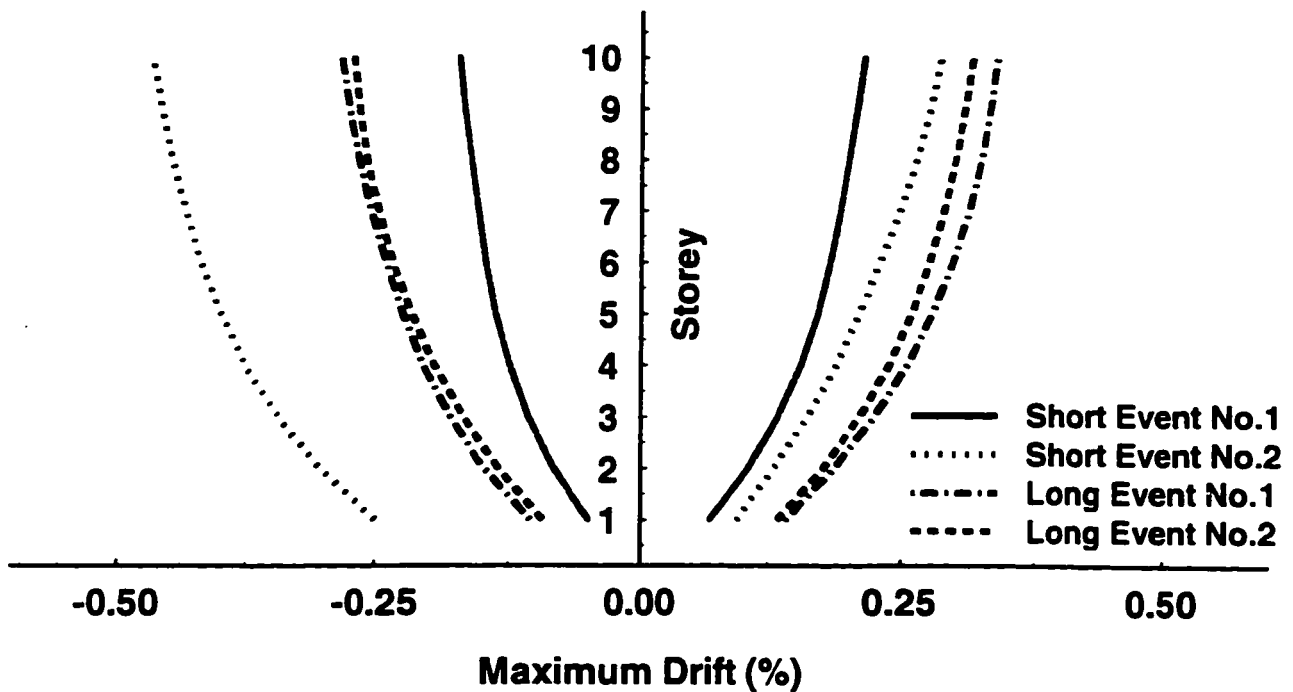


Fig. 4.6 Drift Response of 10-Storey SHW Subjected to Artificial Records for Western Canada

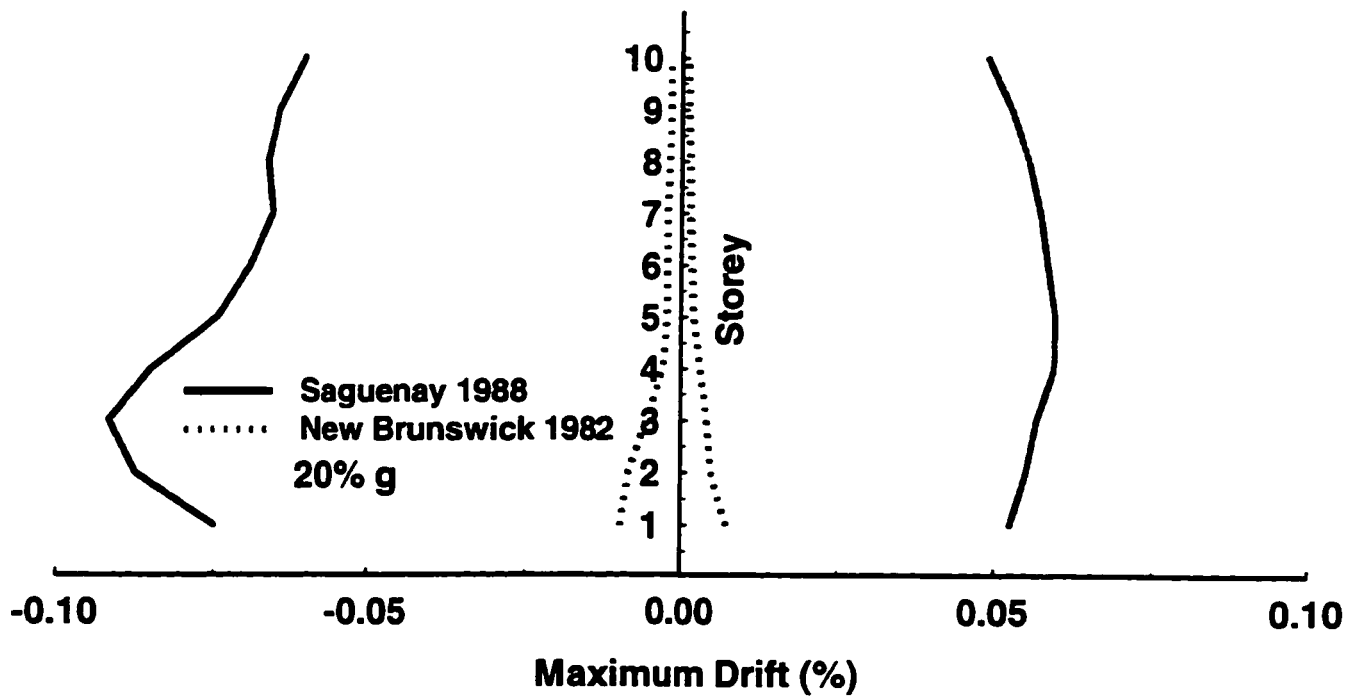


Fig. 4.7 Drift Response of 10-Storey MRF Subjected to Selected Records for Eastern Canada

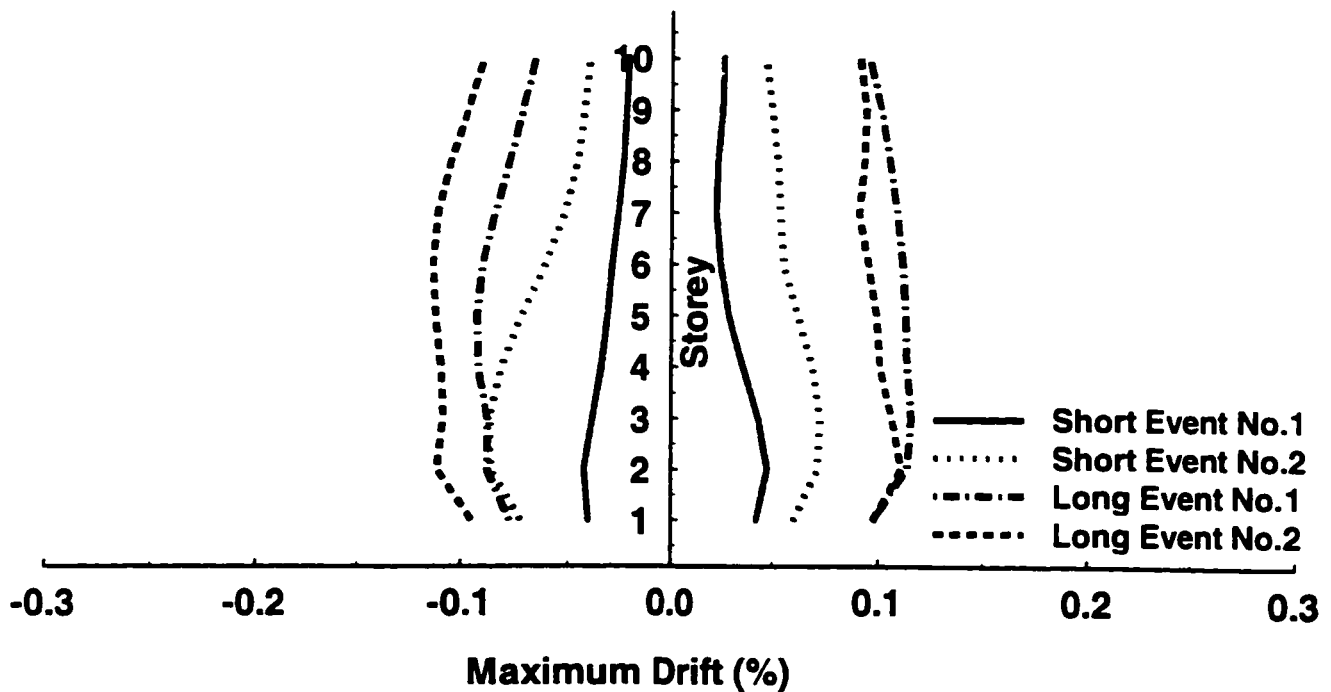


Fig. 4.8 Drift Response of 10-Storey MRF Subjected to Artificial Records for Eastern Canada

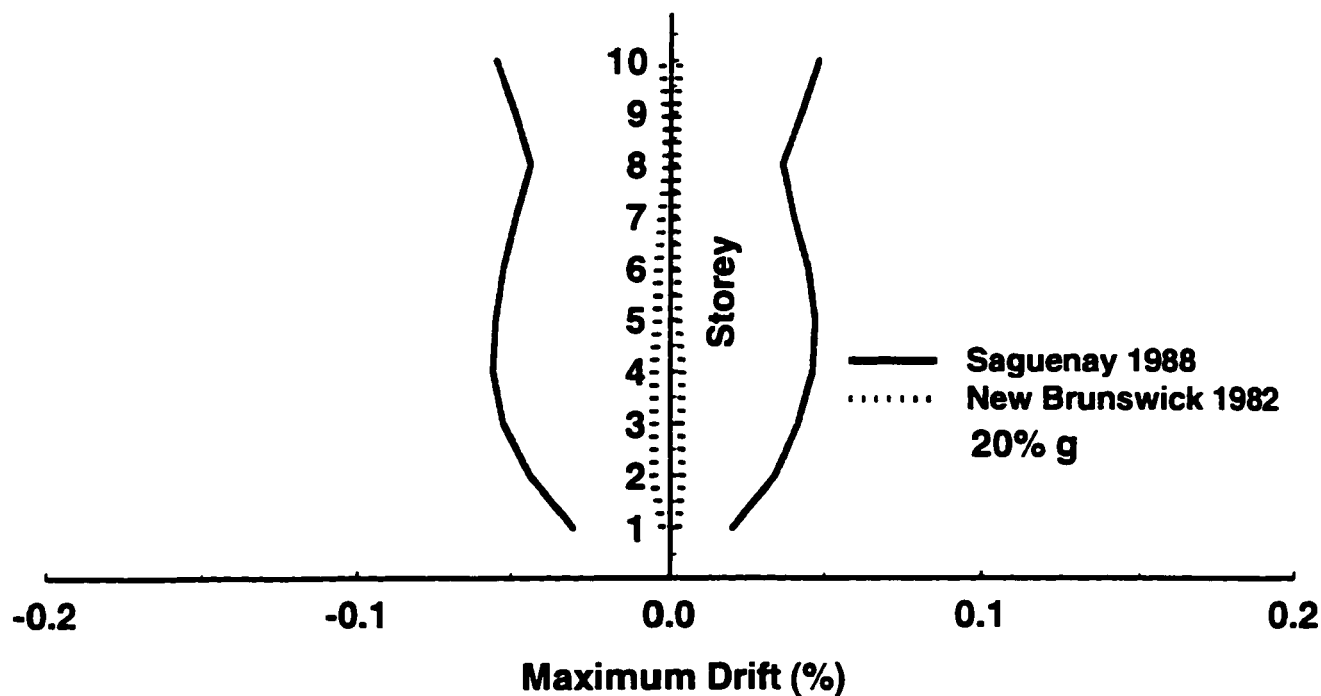


Fig. 4.9 Drift Response of 10-Storey SHW Subjected to Selected Records for Eastern Canada

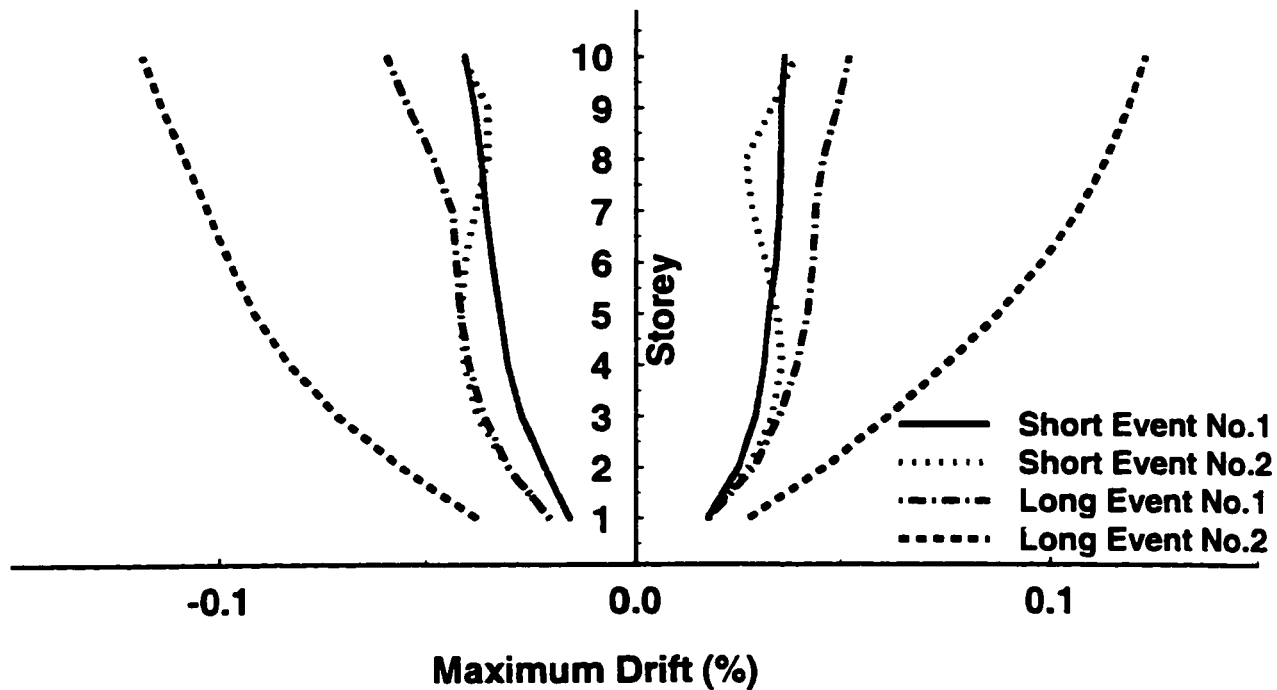
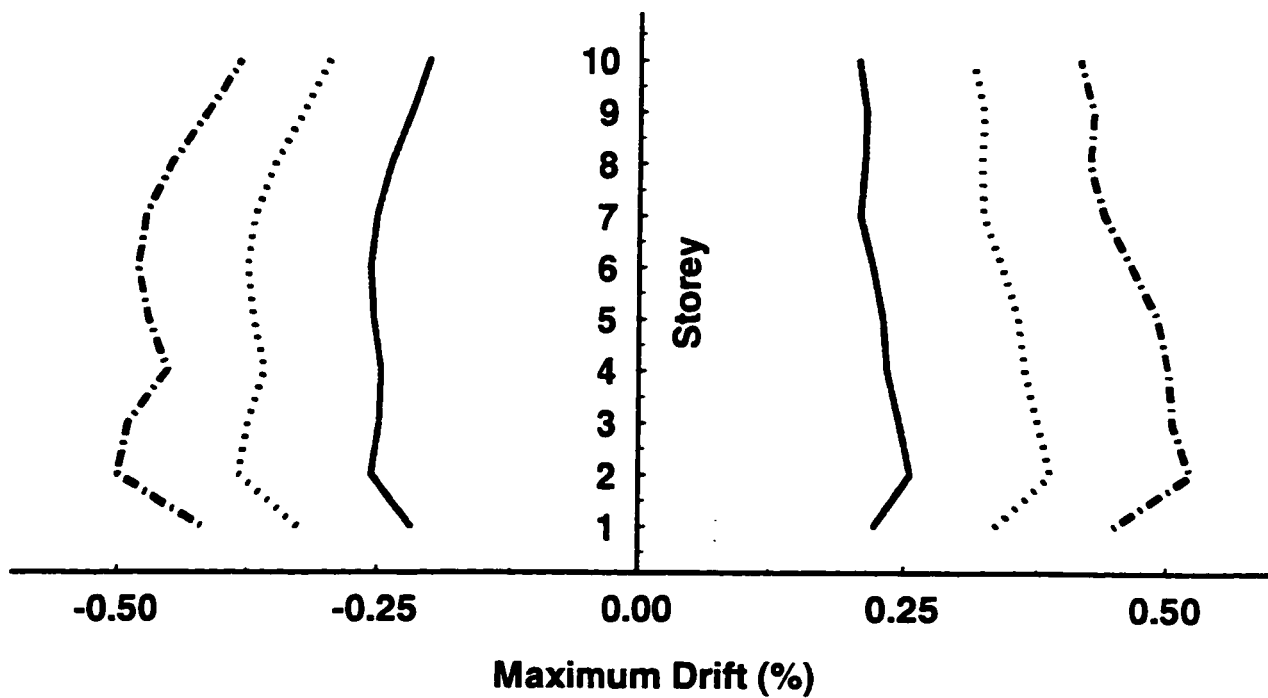
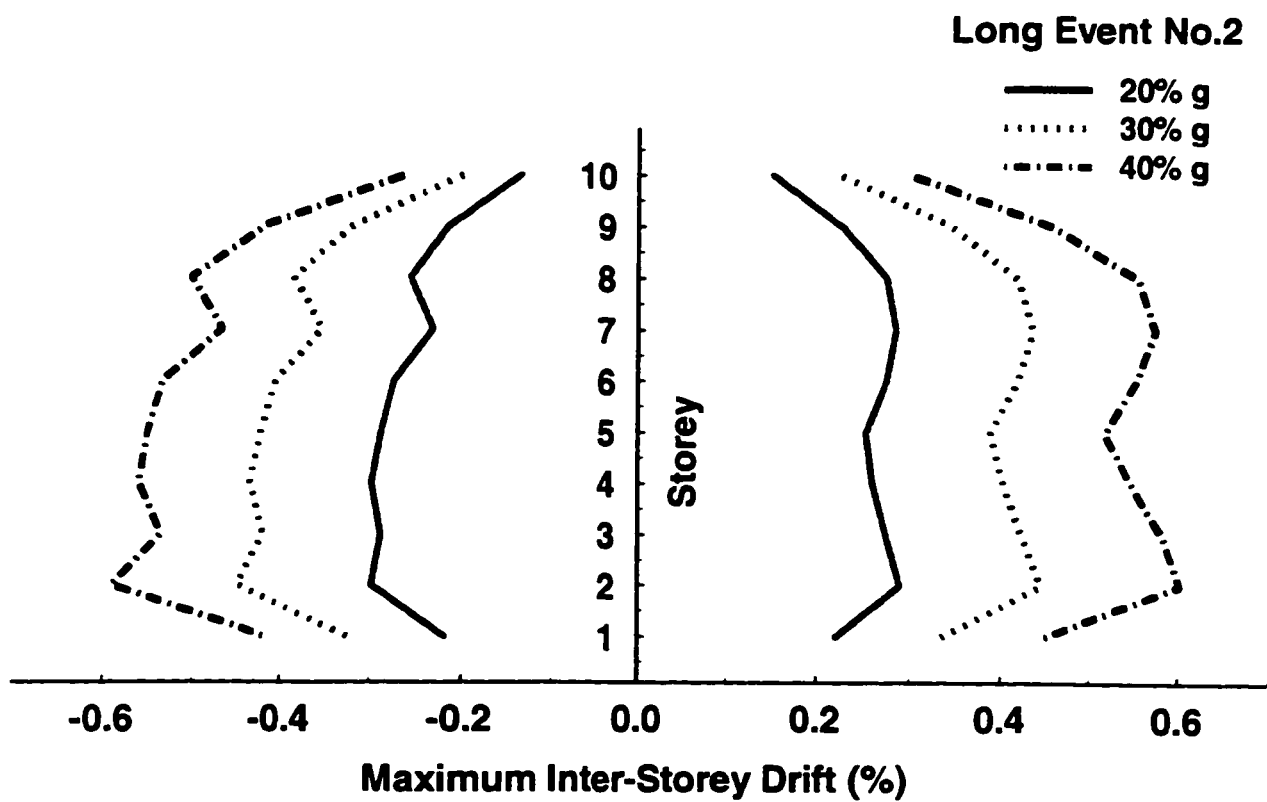


Fig. 4.10 Drift Response of 10-Storey SHW Subjected to Artificial Records for Eastern Canada



(a)



(b)

Fig. 4.11 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to Long Event No.2 for Eastern Canada

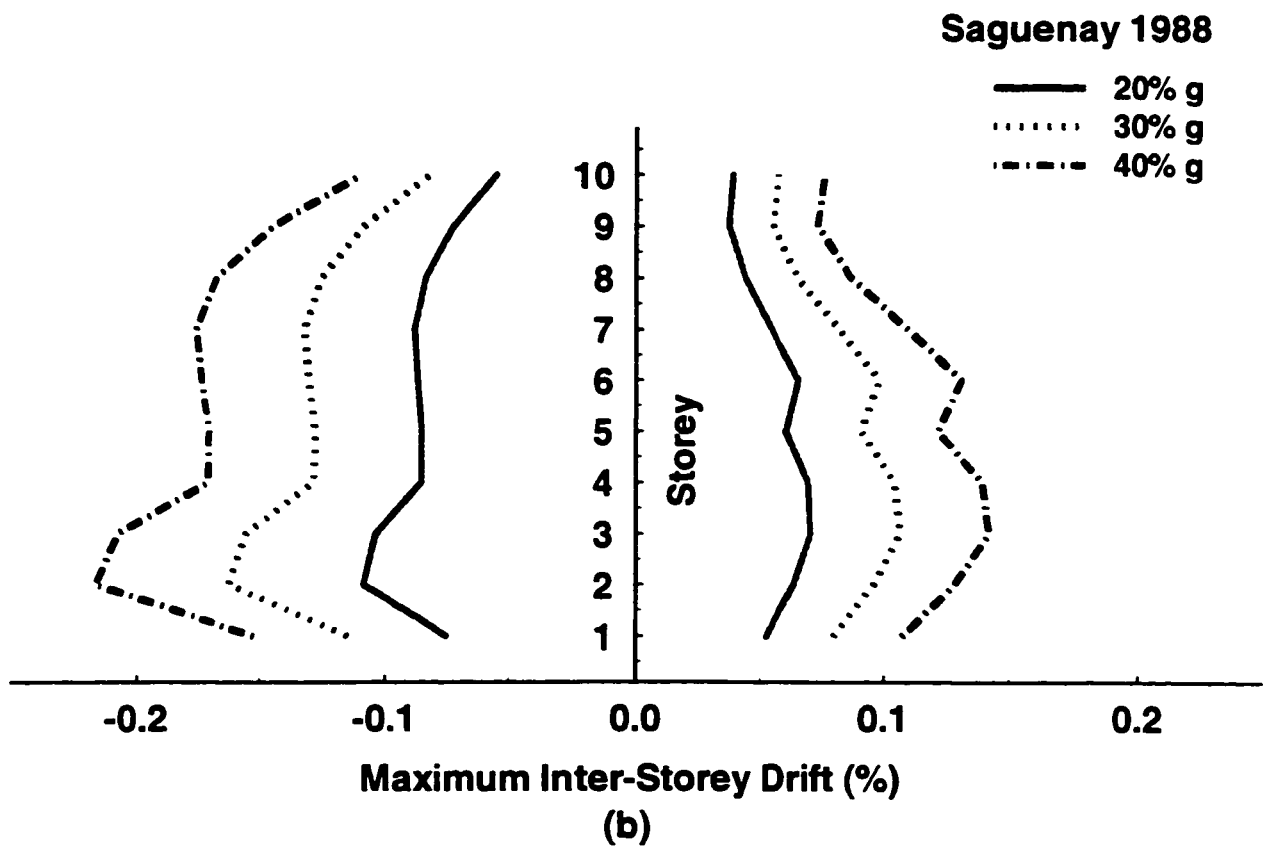
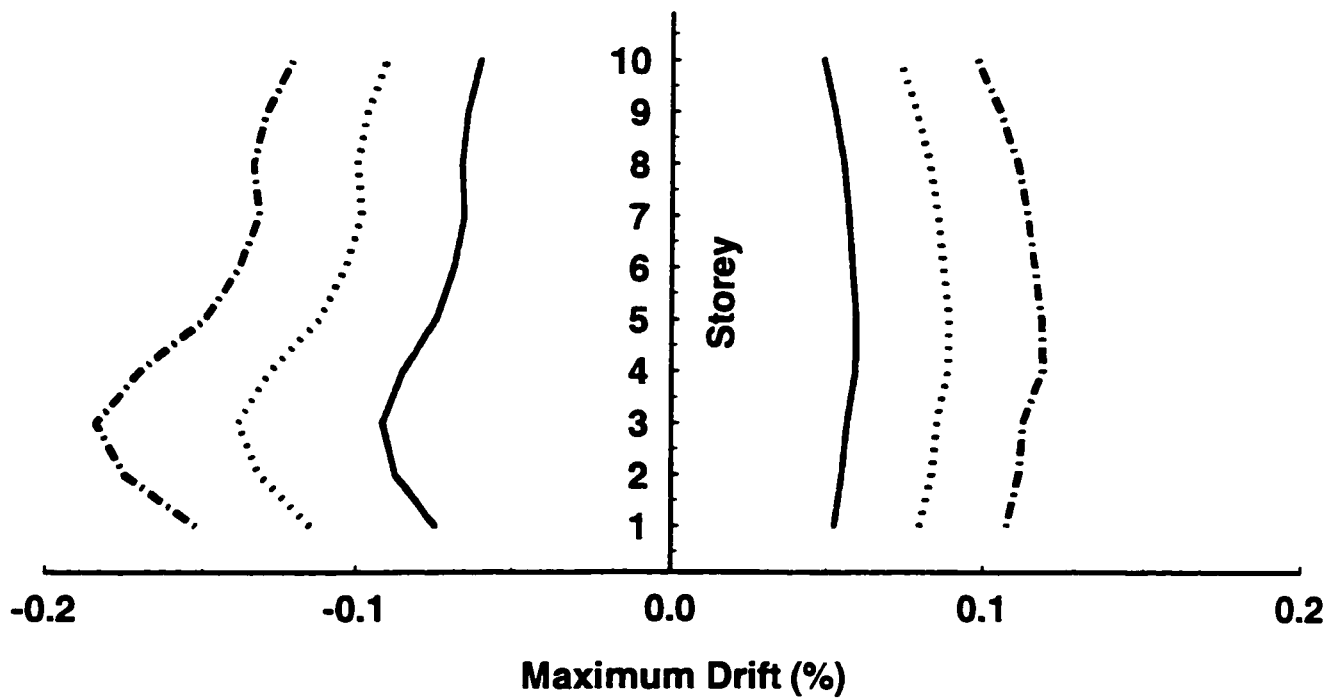
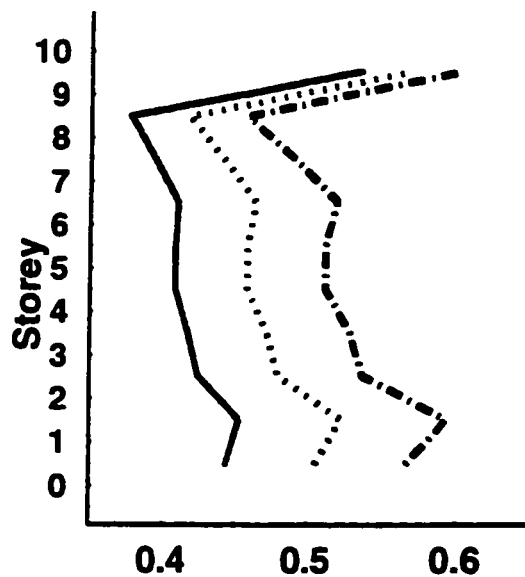
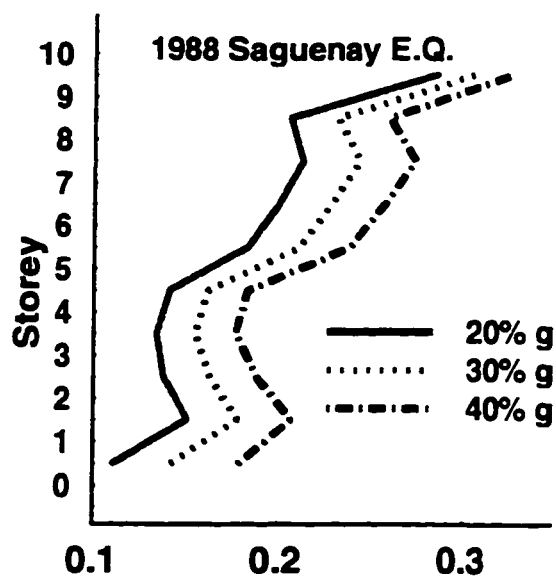


Fig. 4.12 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay records for Eastern Canada

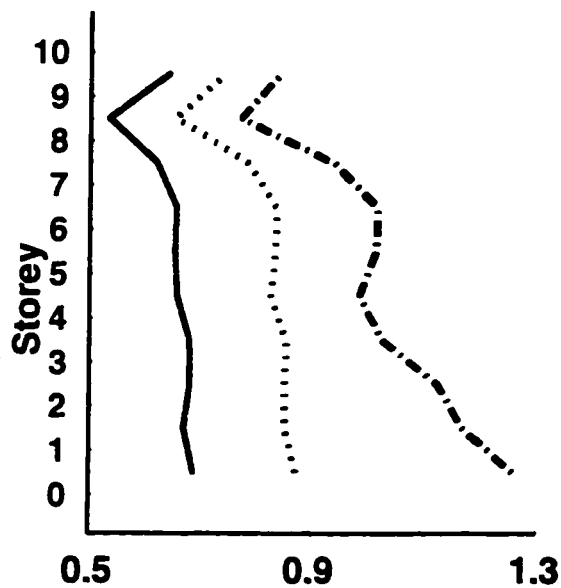


Maximum Beam Ductility Ratio
(a)

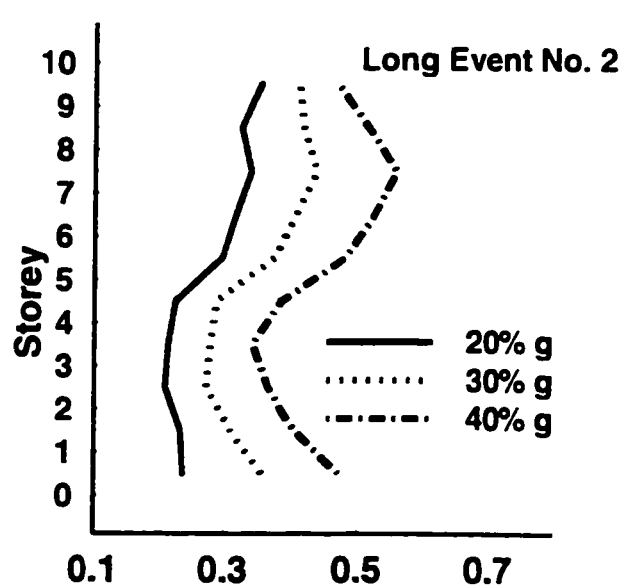


Maximum Column Ductility Ratio
(b)

Fig. 4.13 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada

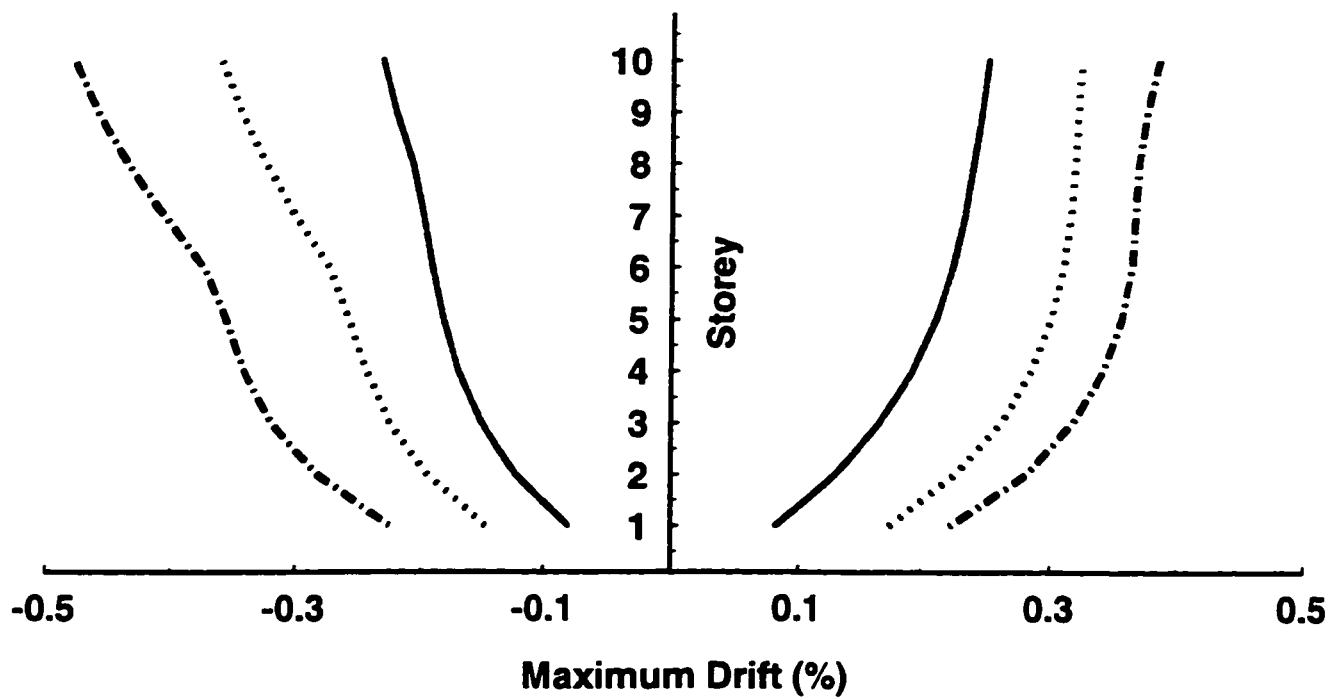


Maximum Beam Ductility Ratio
(a)

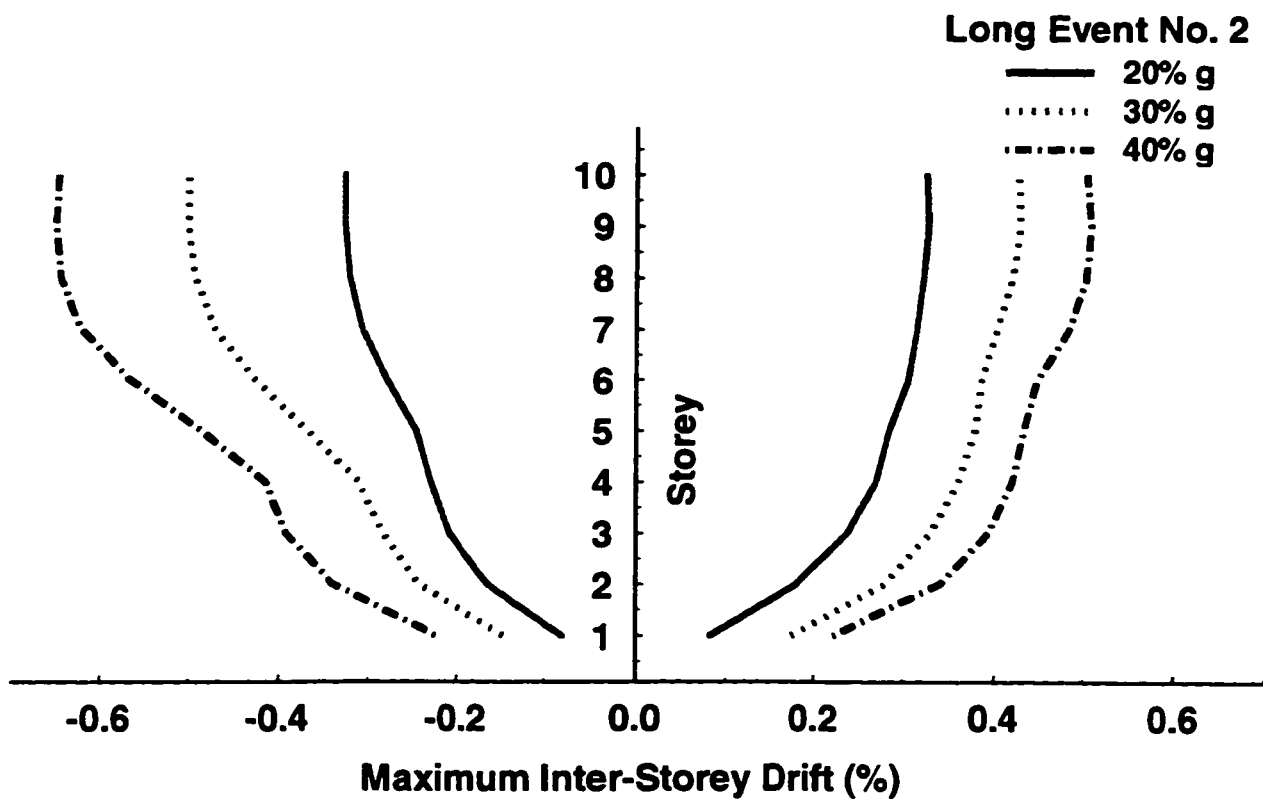


Maximum Column Ductility Ratio
(b)

Fig. 4.14 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to 1988 Saguenay E.Q. for Eastern Canada

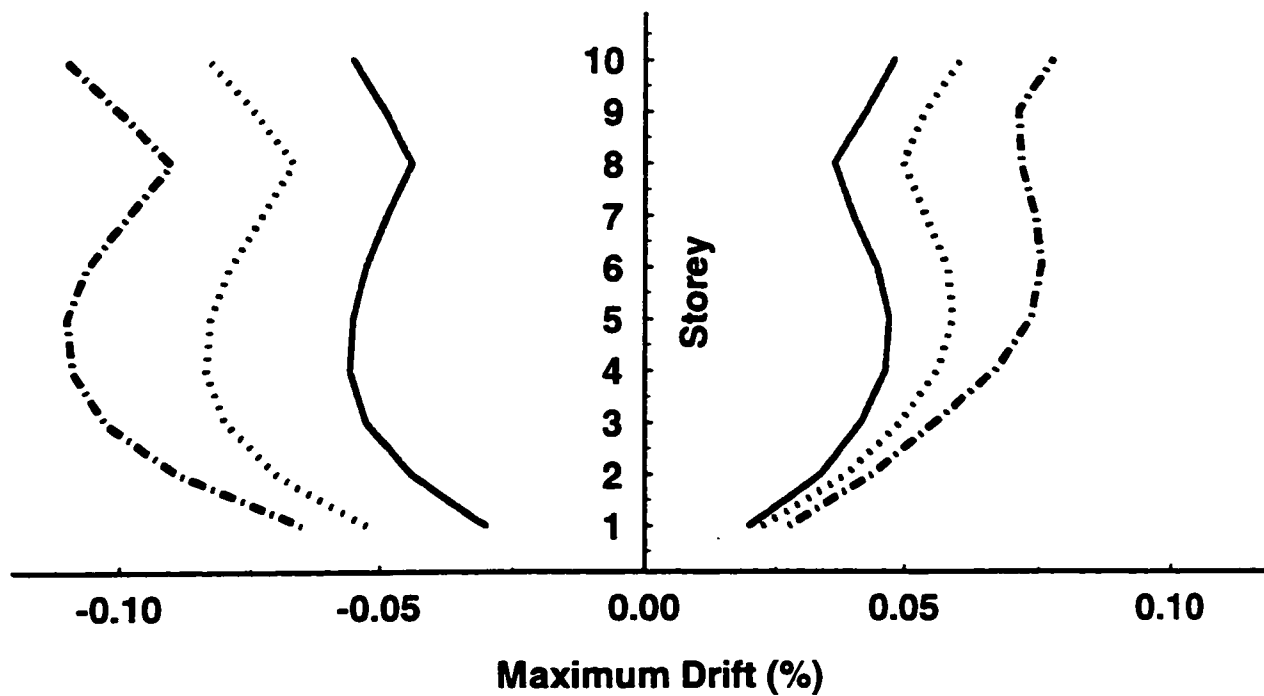


(a)

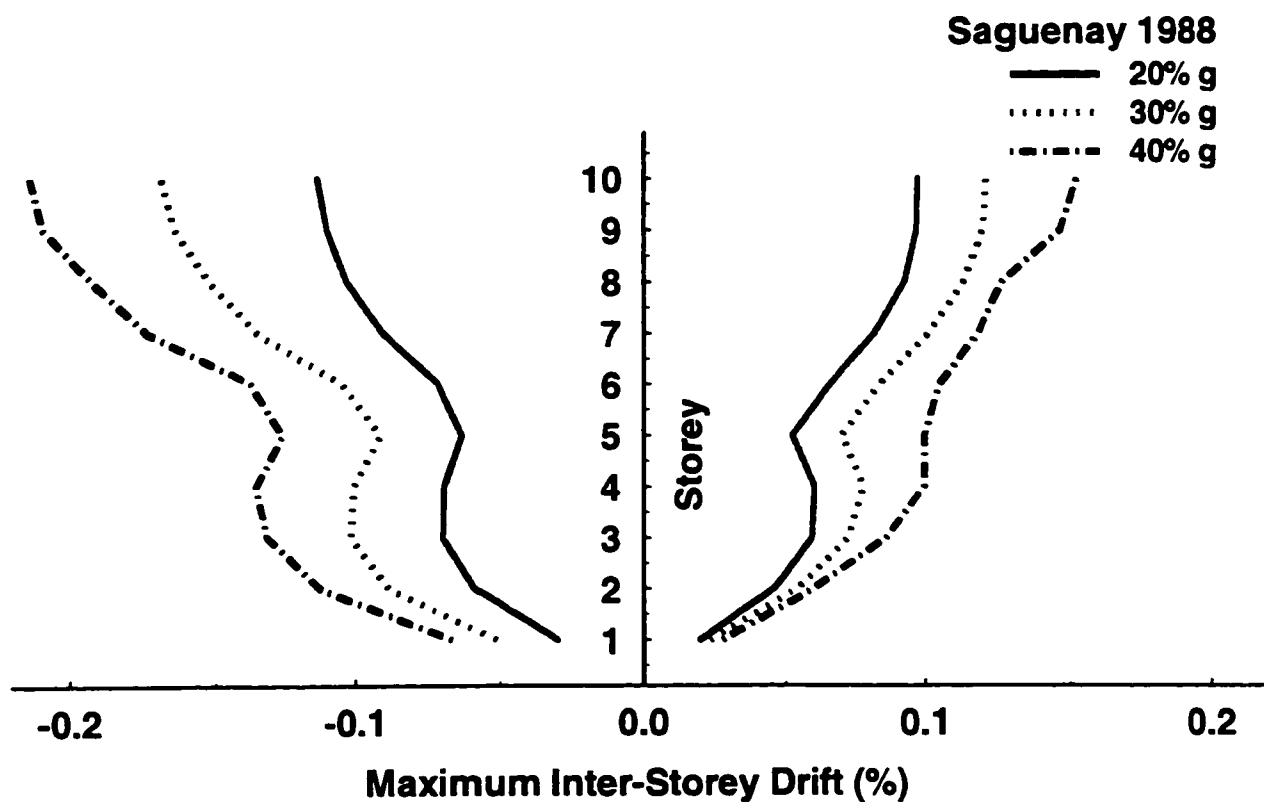


(b)

Fig. 4.15 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada

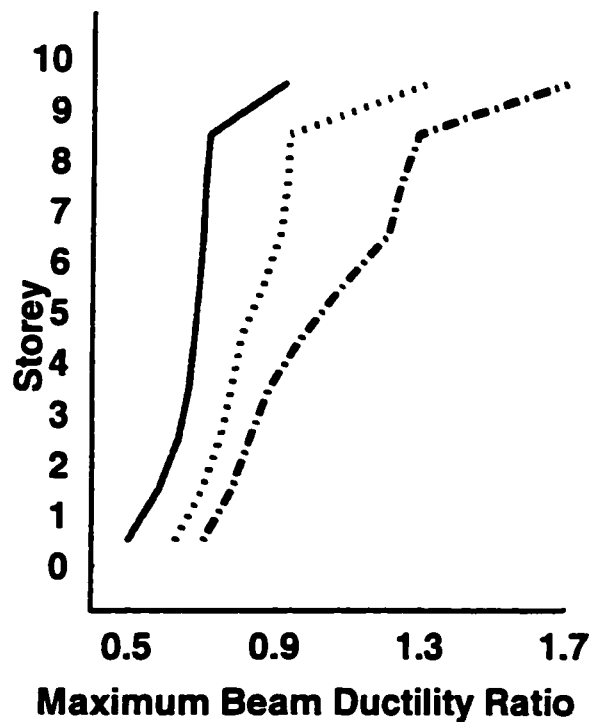


(a)

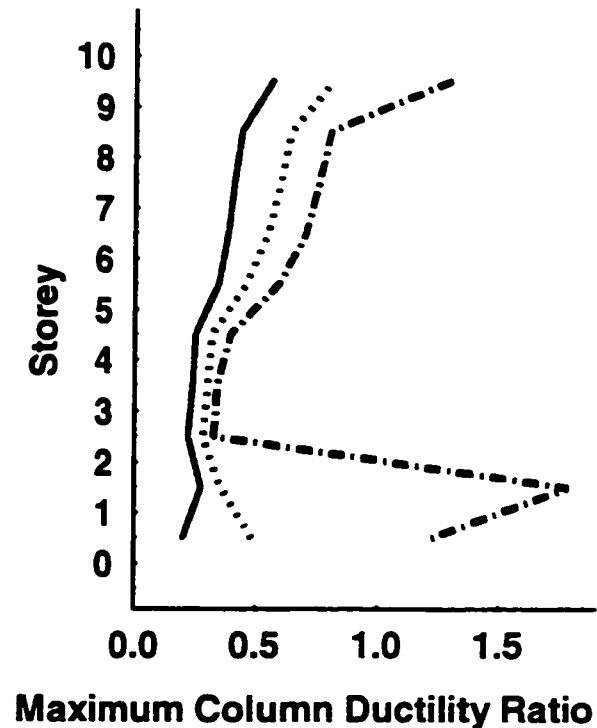


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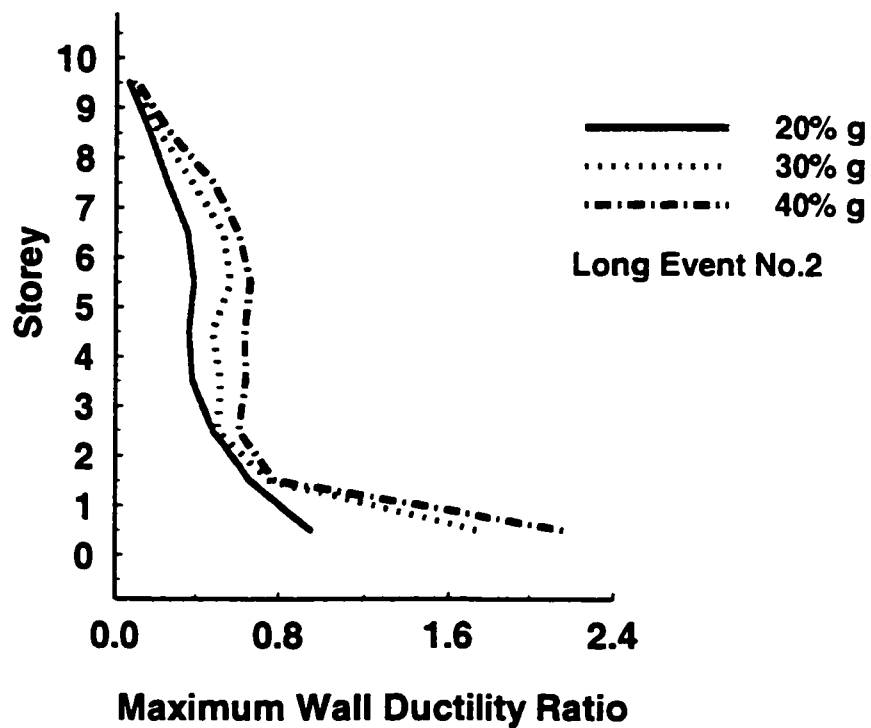
Fig. 4.16 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay records for Eastern Canada



(a)



(b)



(c)

Fig. 4.17 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to Long Event No.2 for Eastern Canada

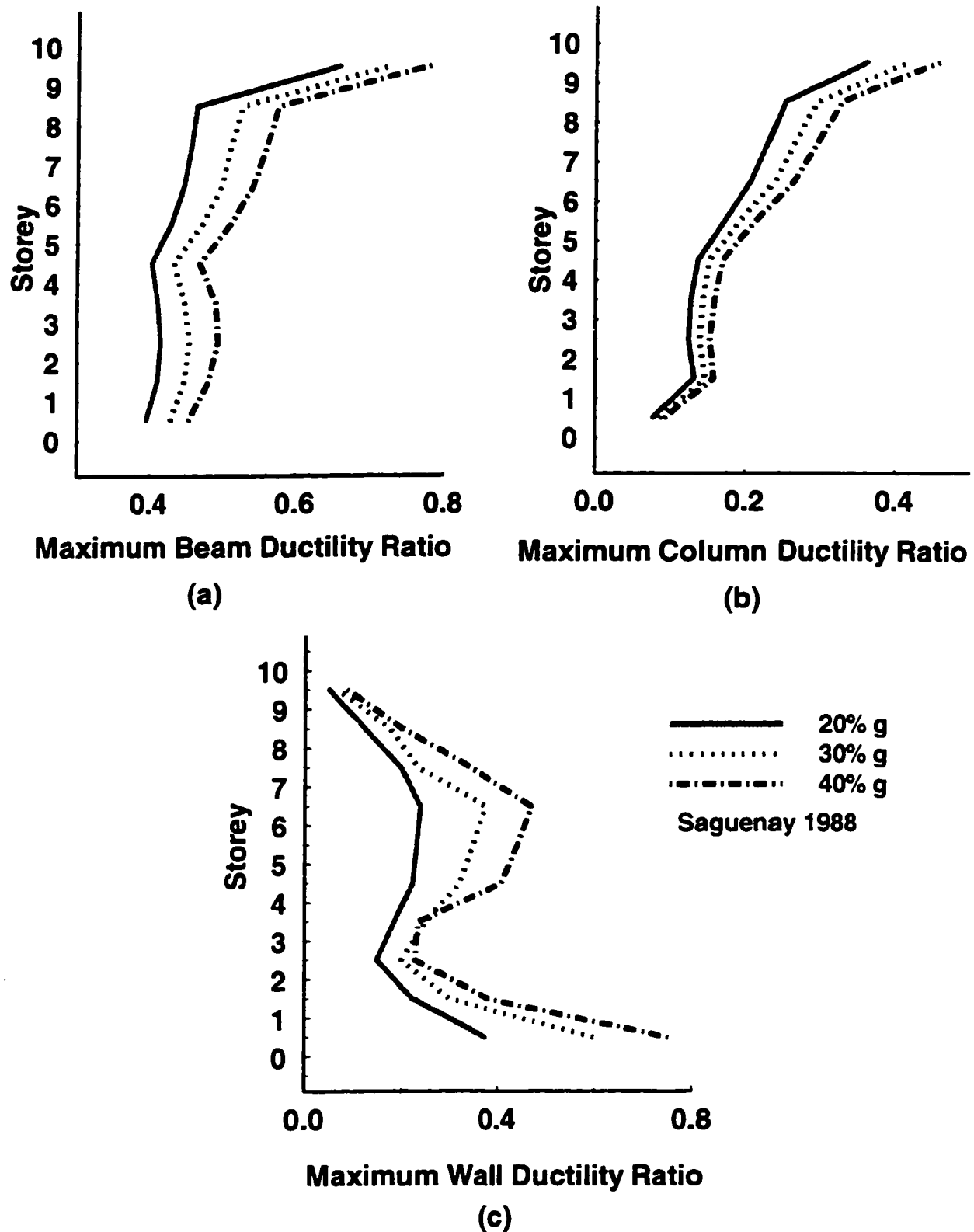
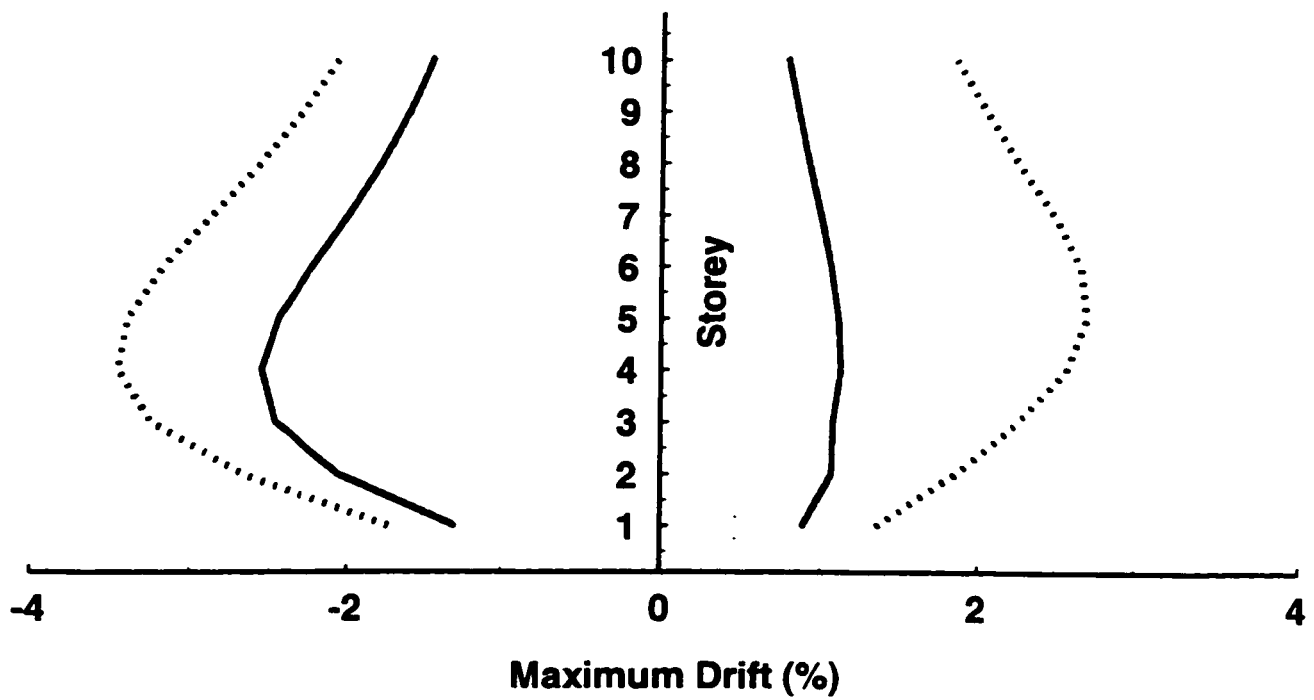
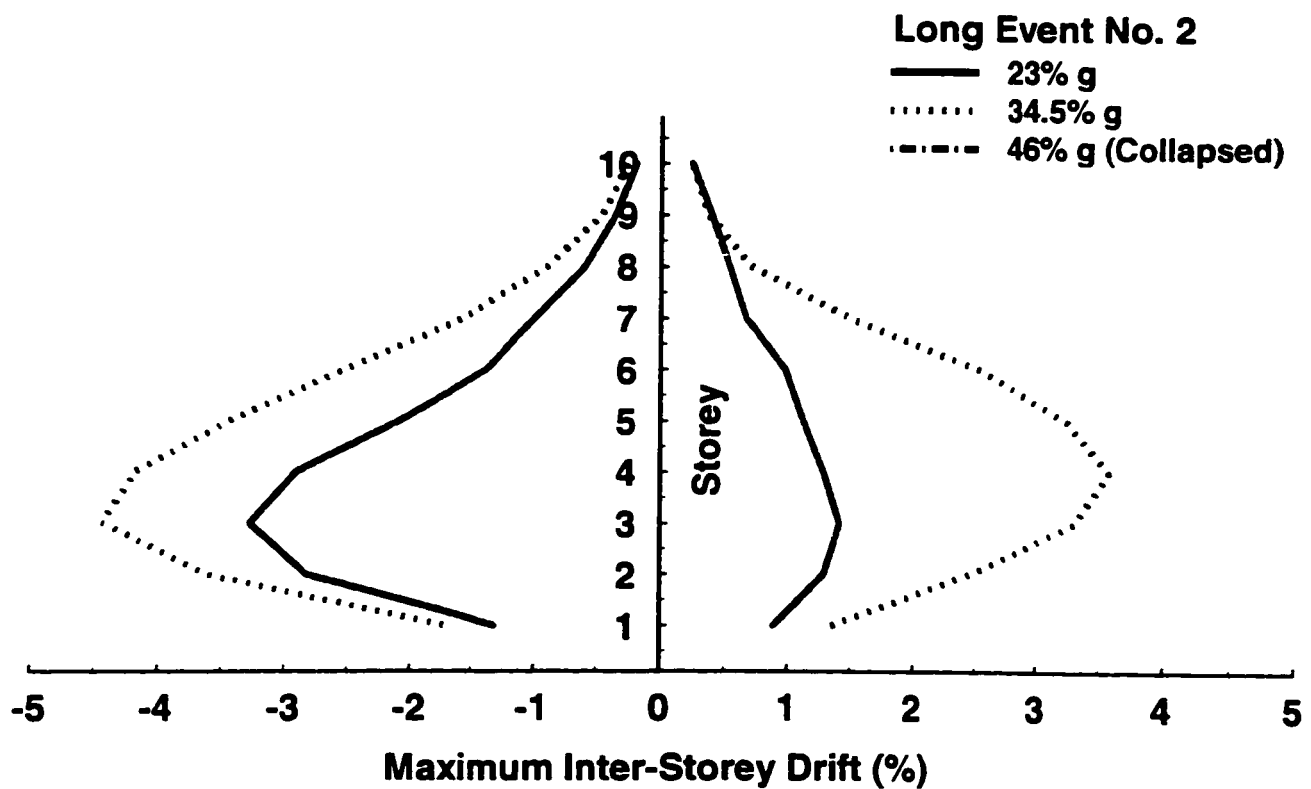


Fig. 4.18 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to 1988 Saguenay record for Eastern Canada

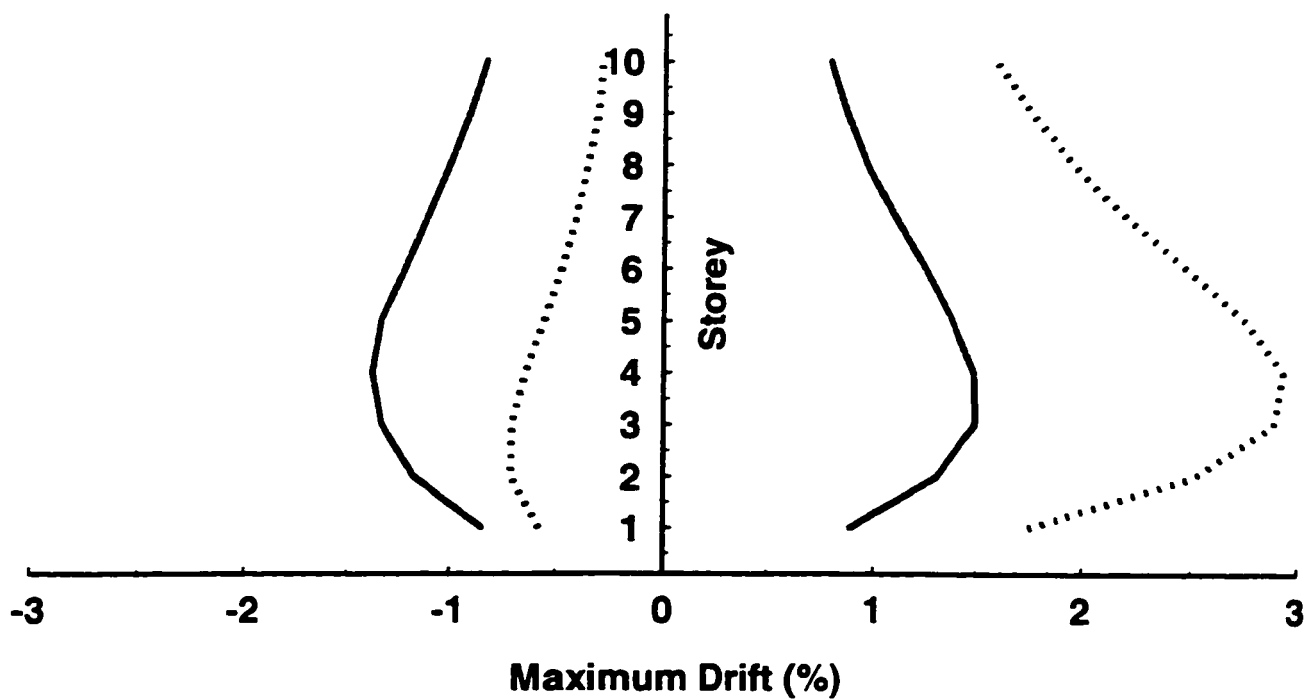


(a)

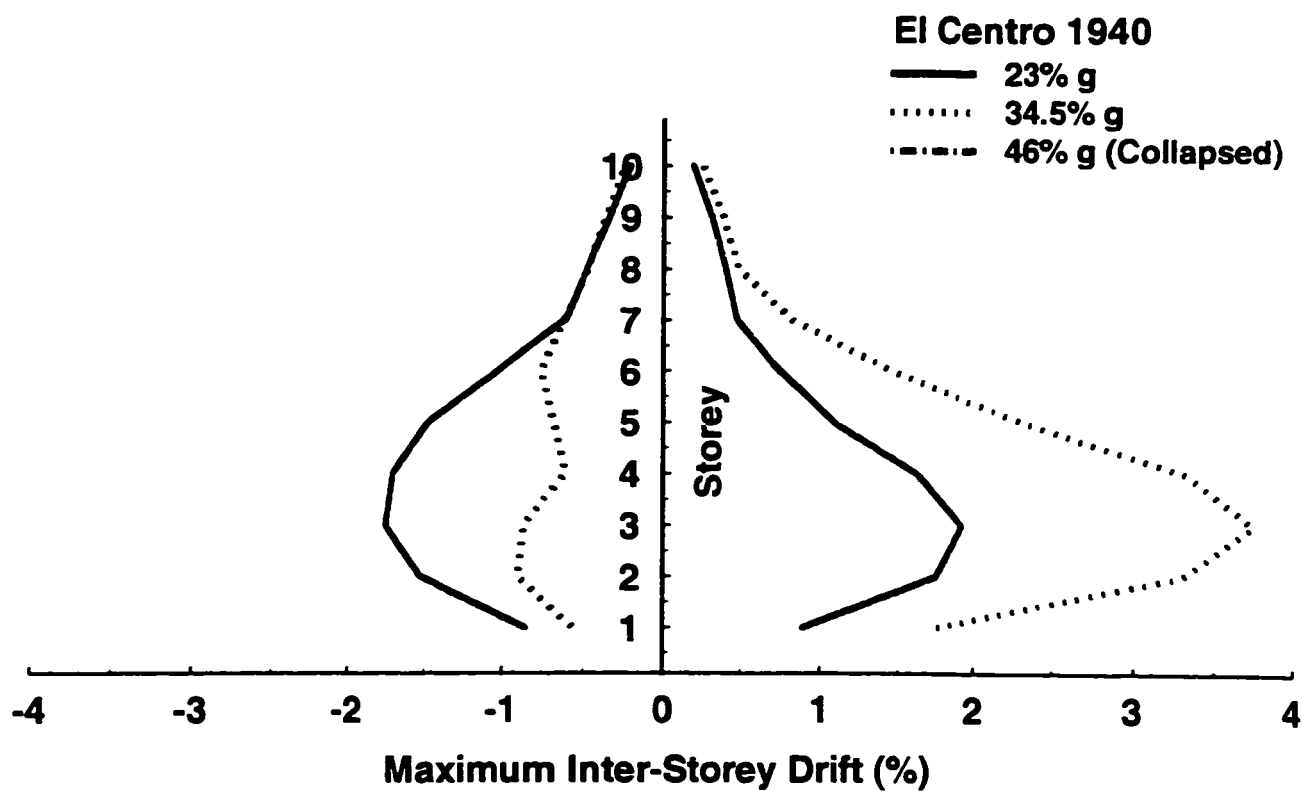


(b)

Fig. 4.19 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



(a)



(b)

Fig. 4.20 Effect of E.Q. Intensity on Drift Response. 10-Storey MRF Subjected to 1940 El Centro record for Western Canada

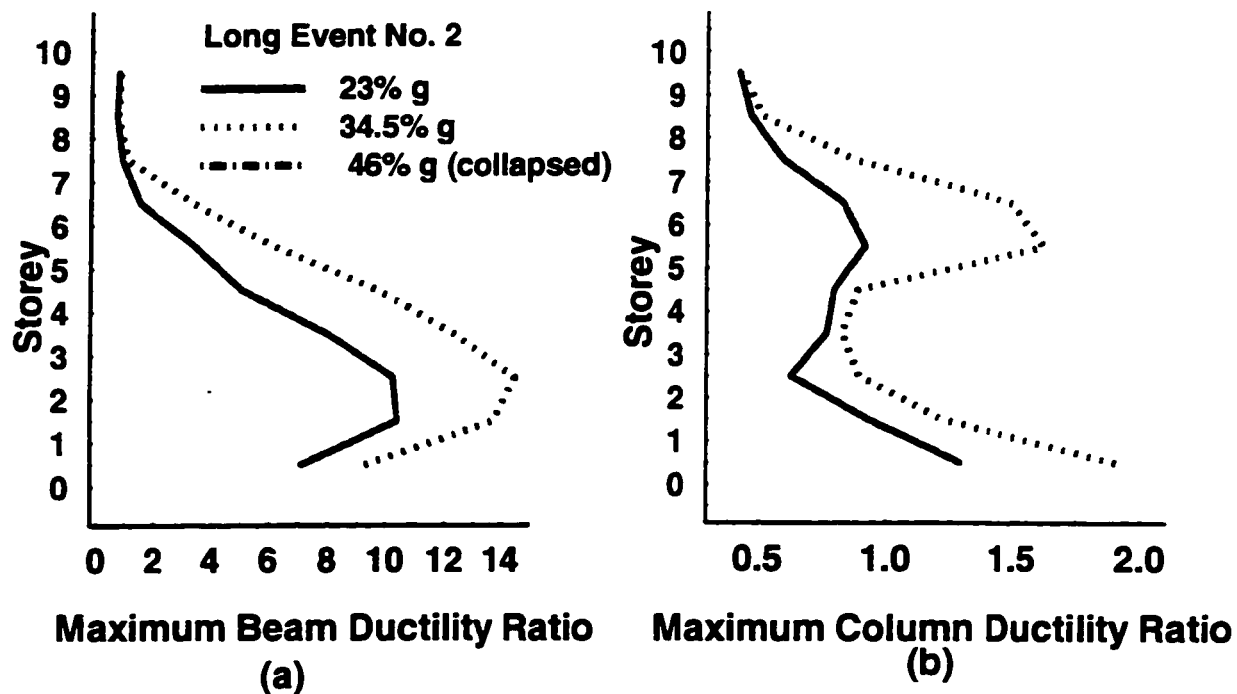


Fig. 4.21 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

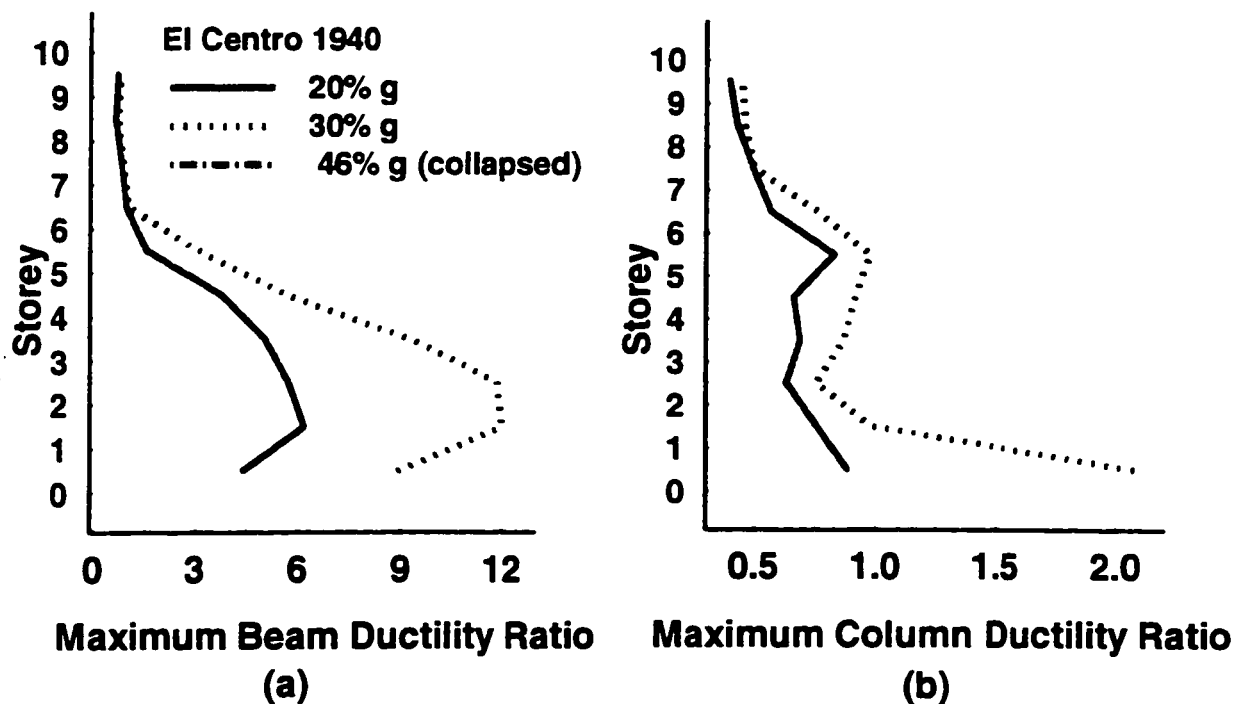


Fig. 4.22 Effect of E.Q. Intensity on Ductility Response. 10-Storey MRF Subjected to 1940 El Centro E.Q. for Western Canada

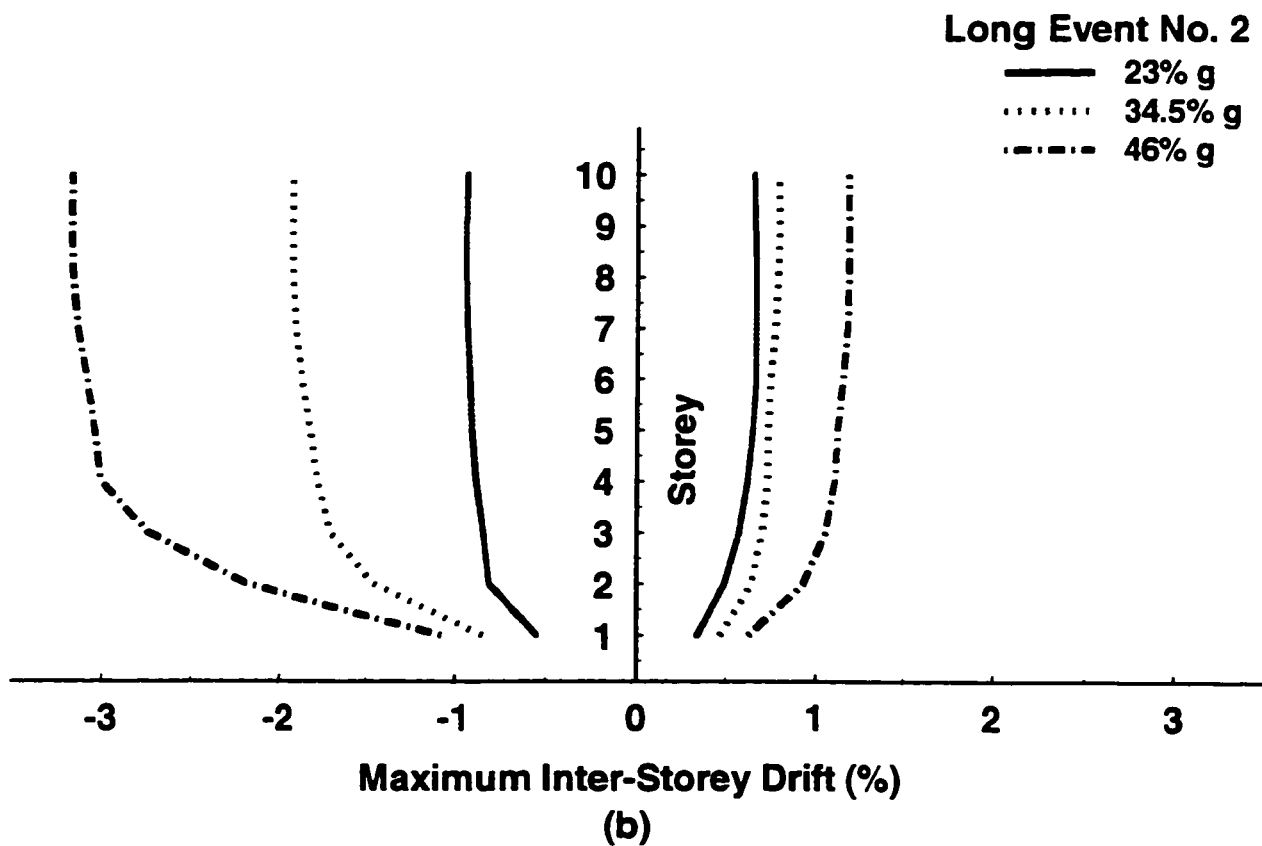
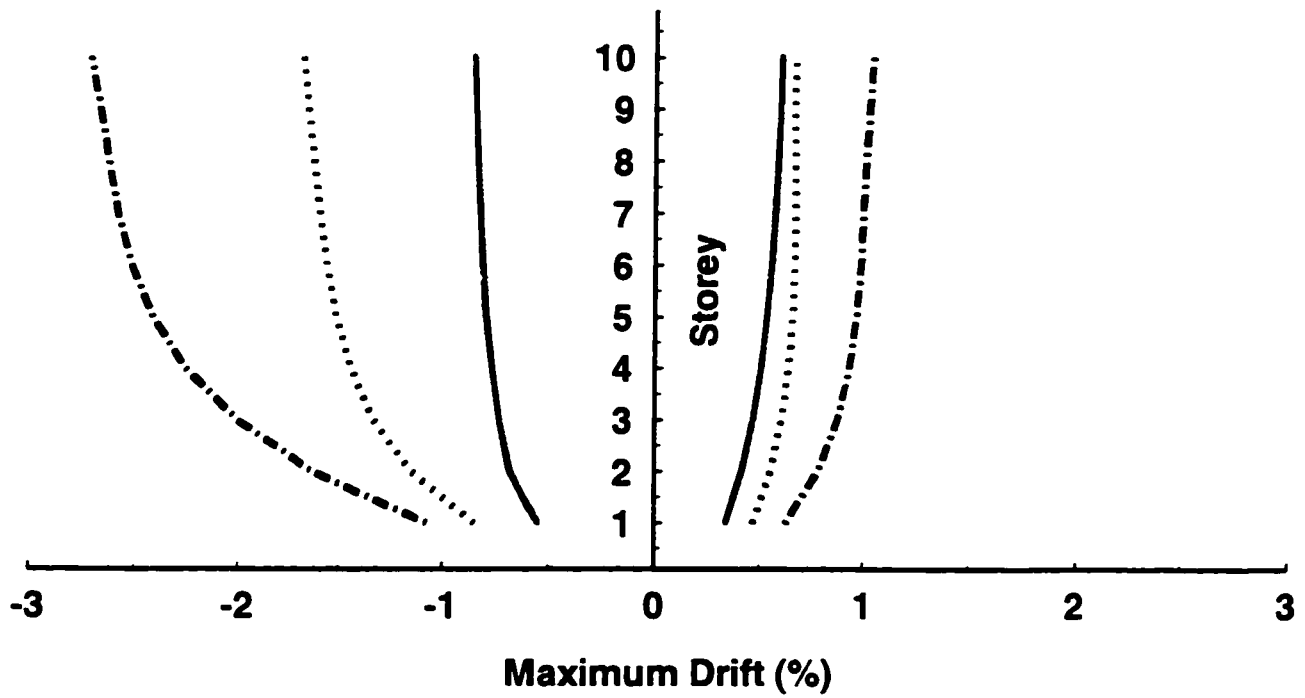
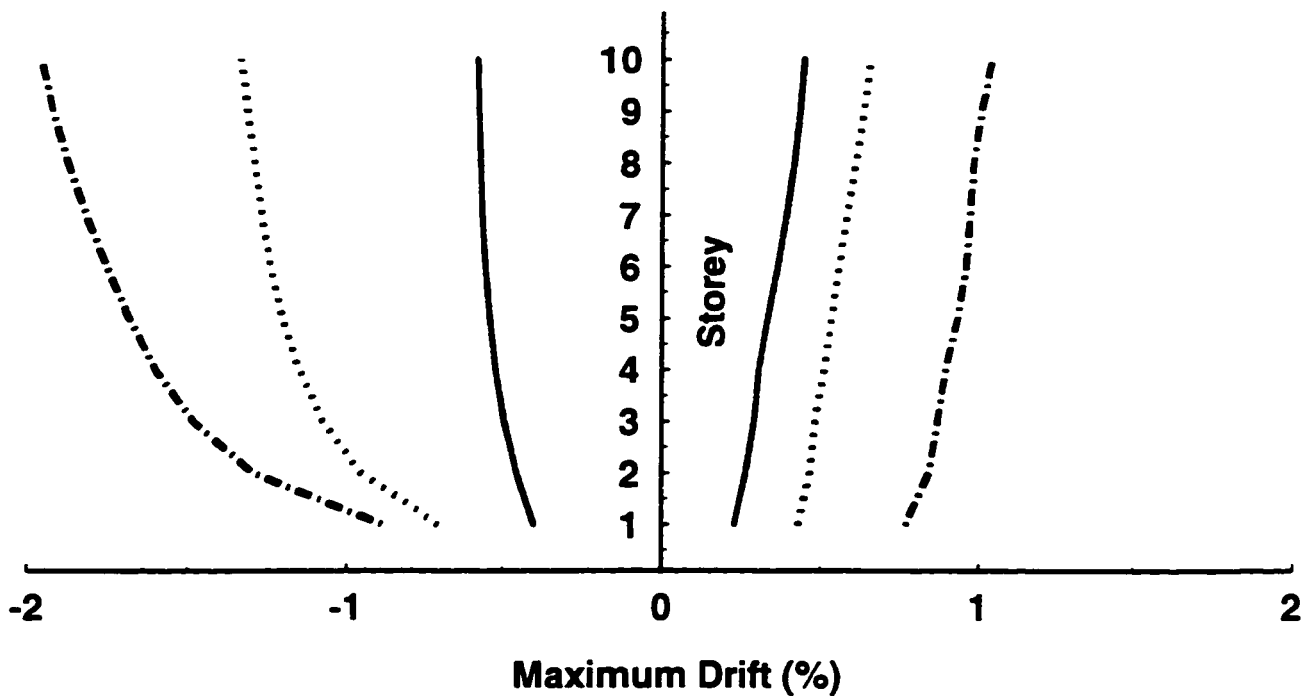
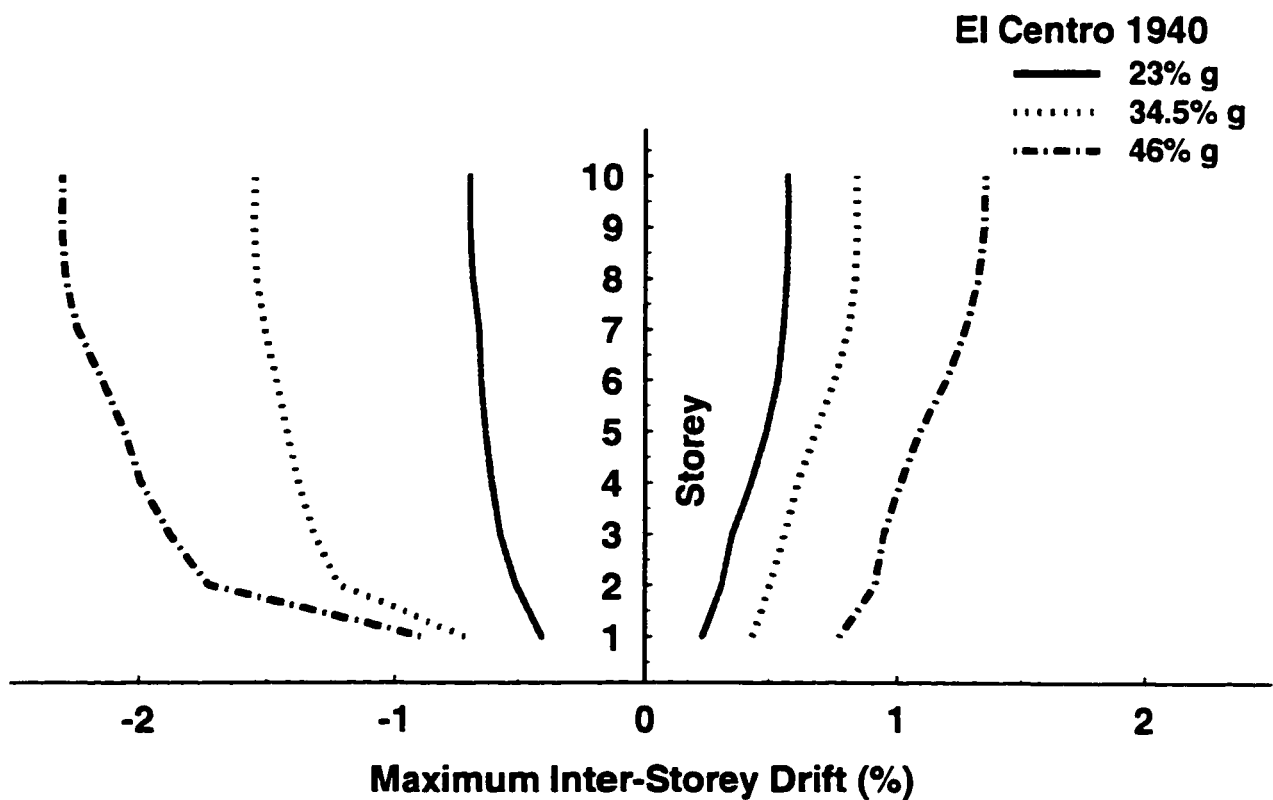


Fig. 4.23 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada

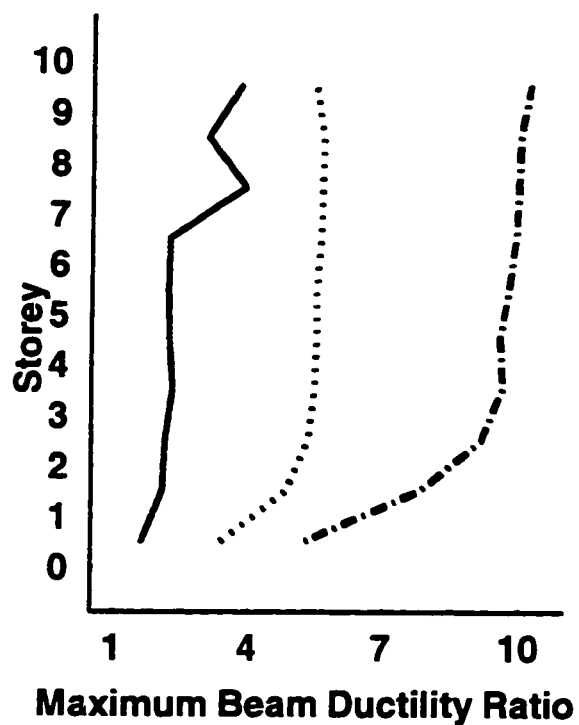


(a)

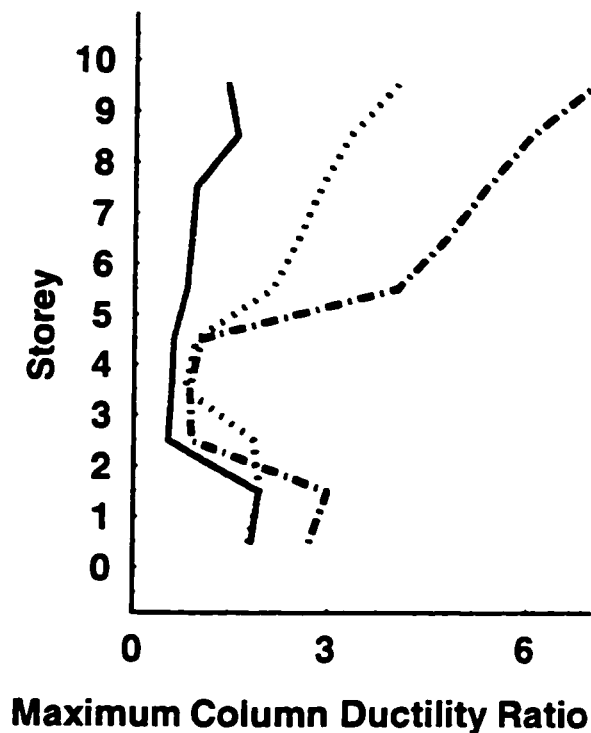


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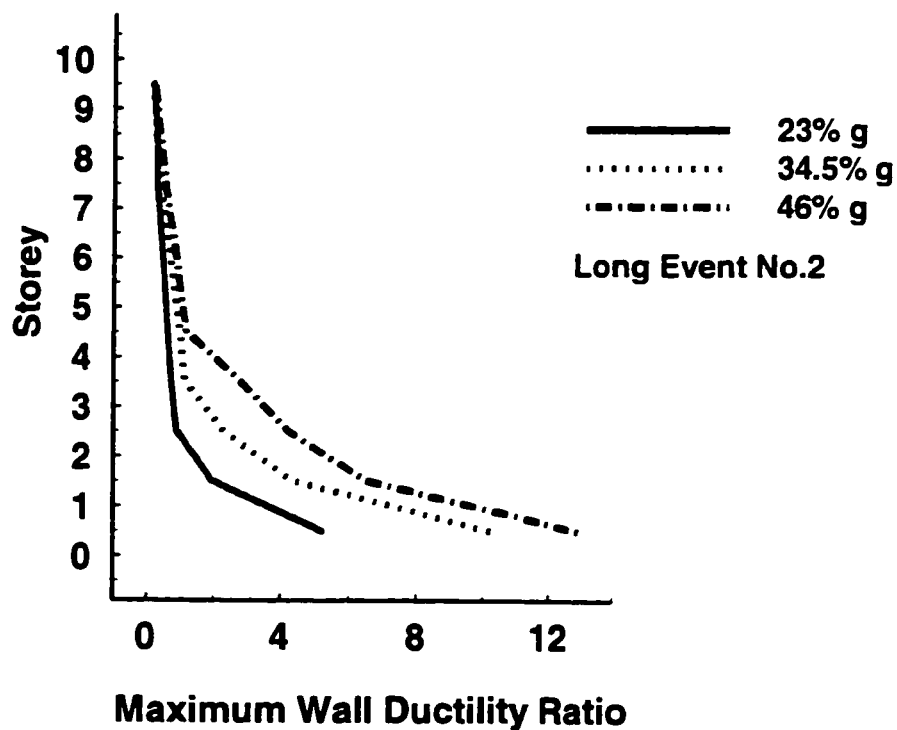
Fig. 4.24 Effect of E.Q. Intensity on Drift Response. 10-Storey SHW Subjected to 1940 El Centro record for Western Canada



(a)

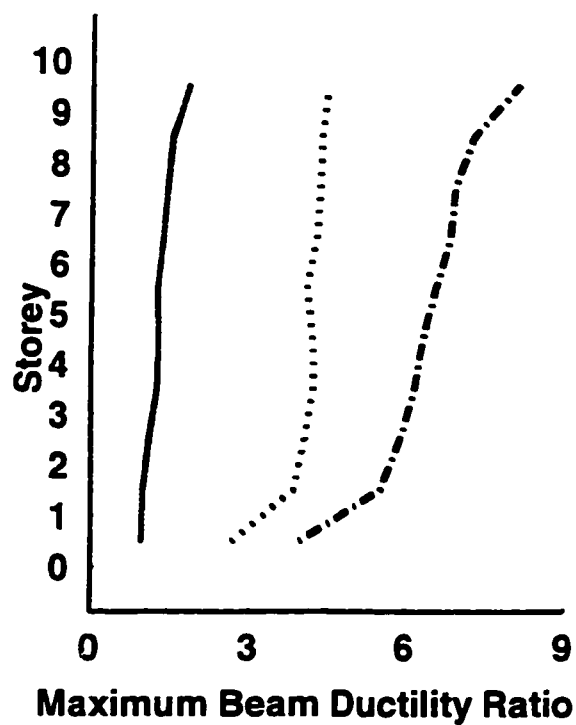


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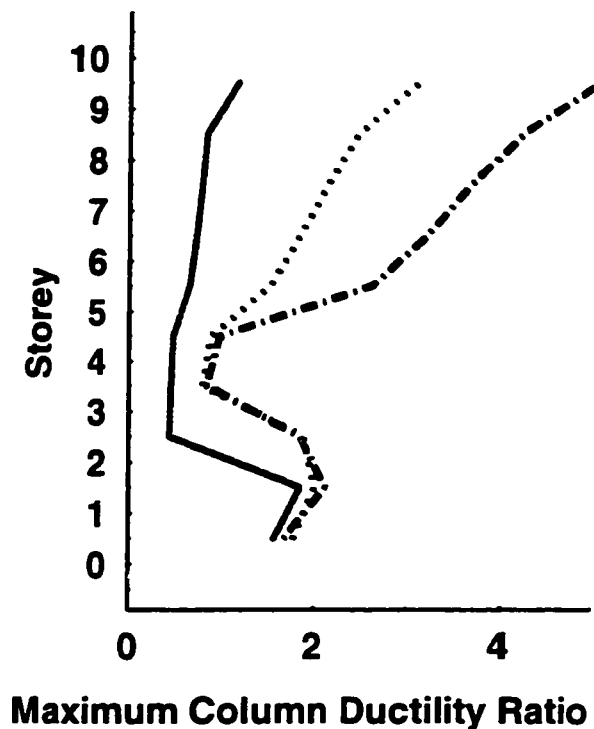


(c)

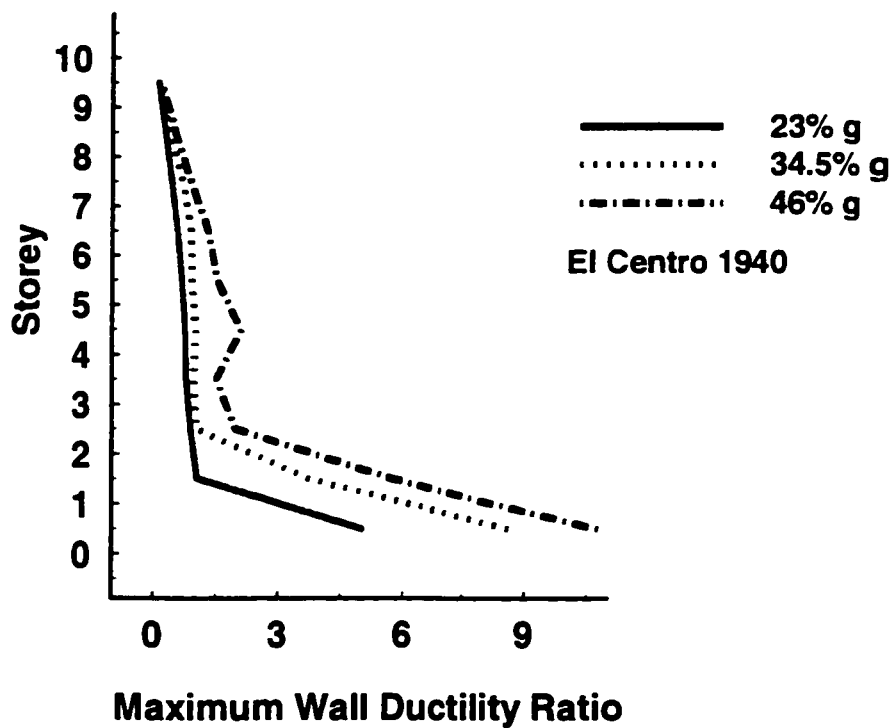
Fig. 4.25 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to Long Event No.2 for Western Canada



(a)

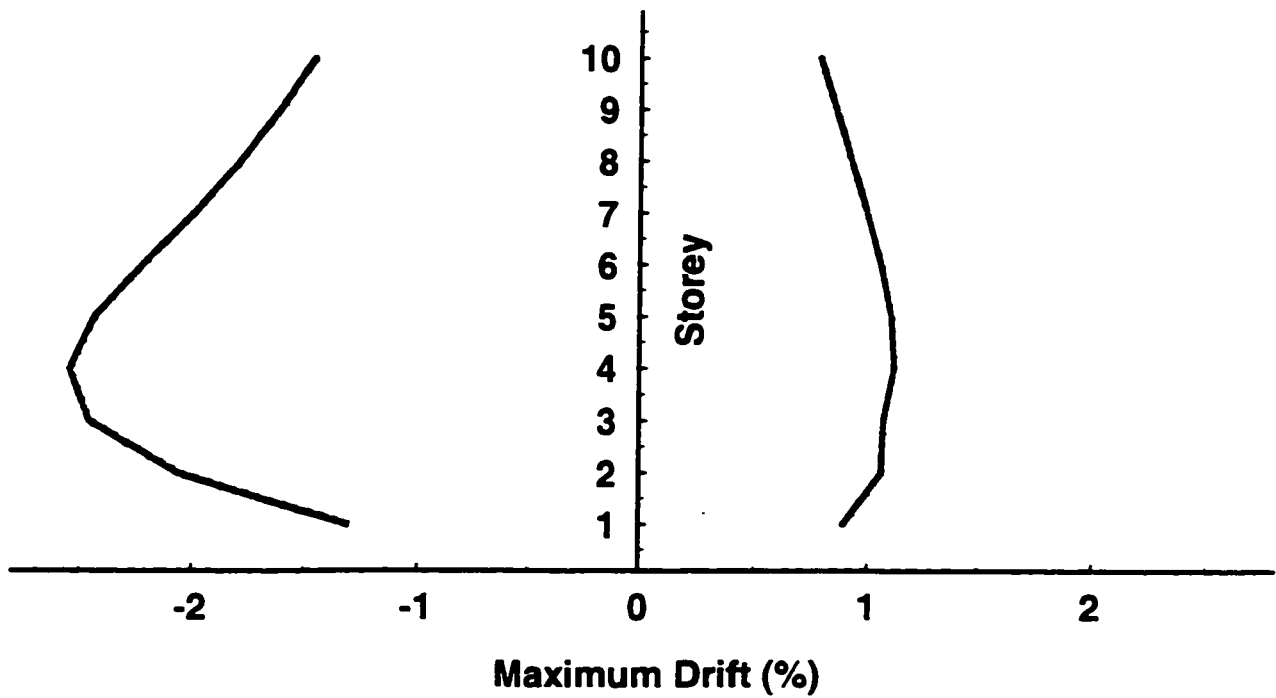


(b)



(c)

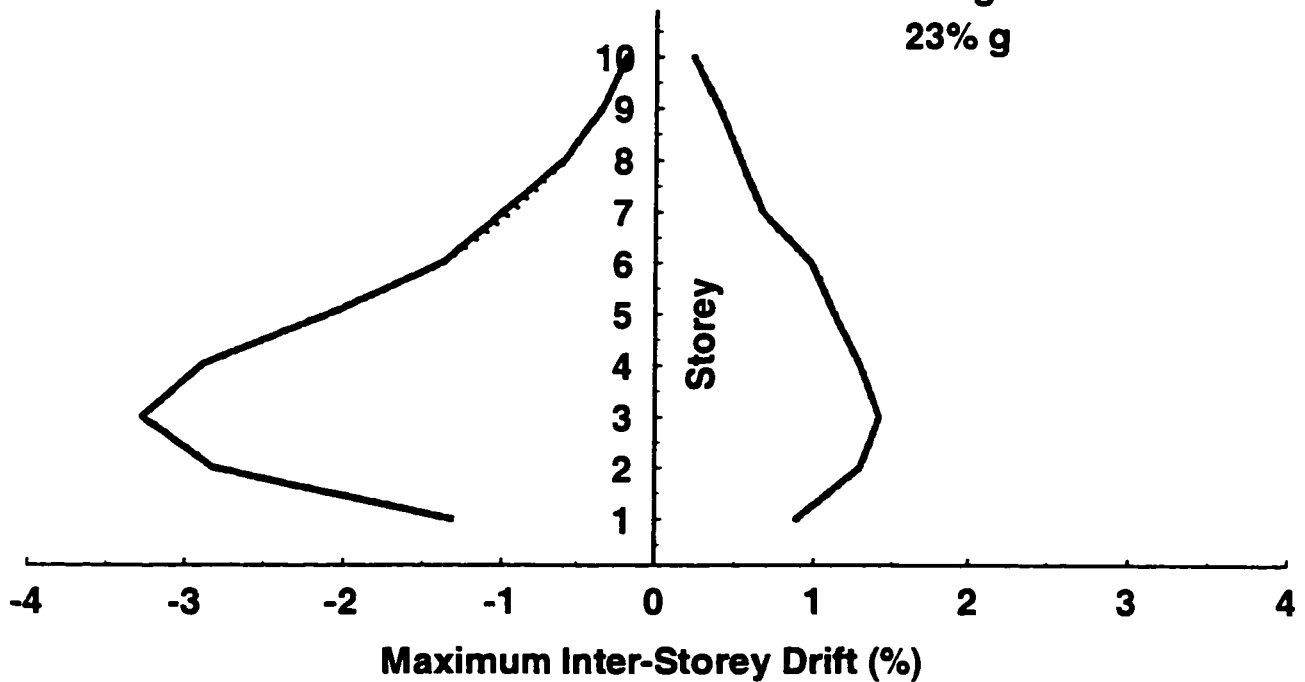
Fig. 4.26 Effect of E.Q. Intensity on Ductility Response. 10-Storey SHW Subjected to 1940 El Centro E.Q. for Western Canada



(a)

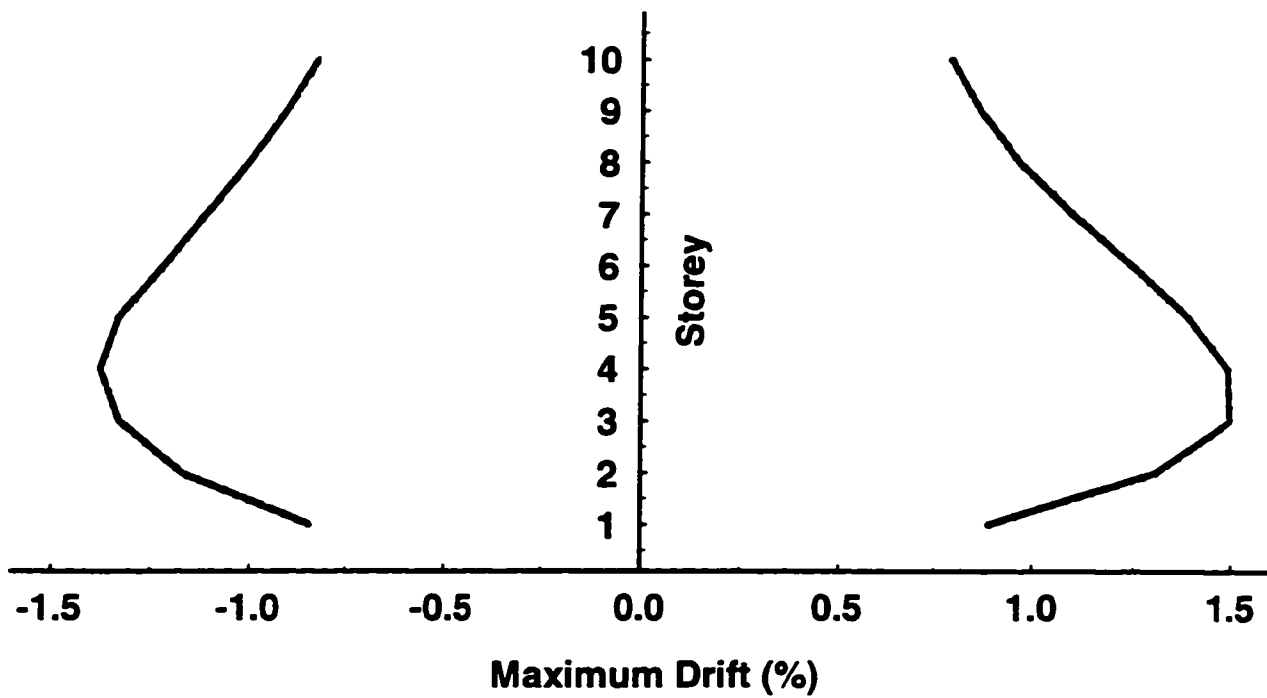
— with P-M Interaction
 without P-M Interaction

Long Event No. 2
 23% g

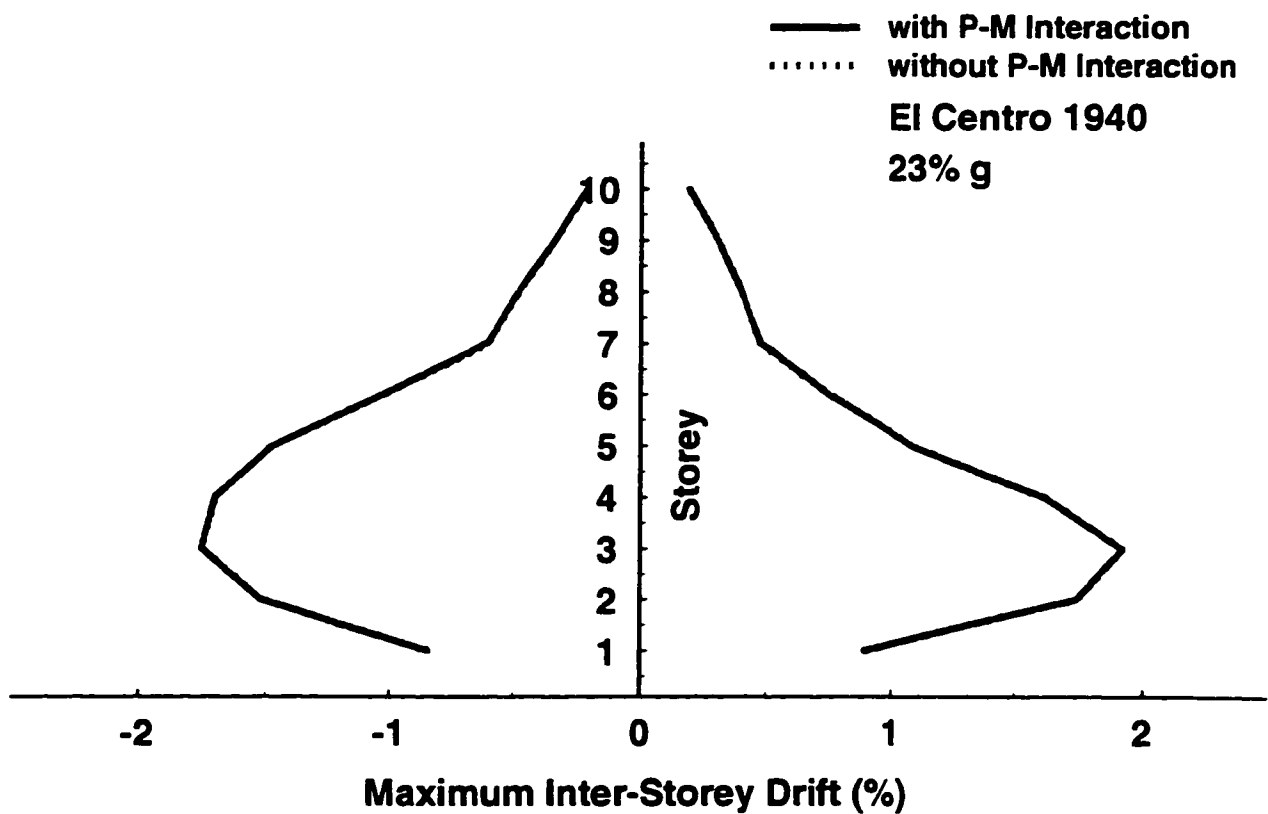


(b)

Fig. 4.27 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

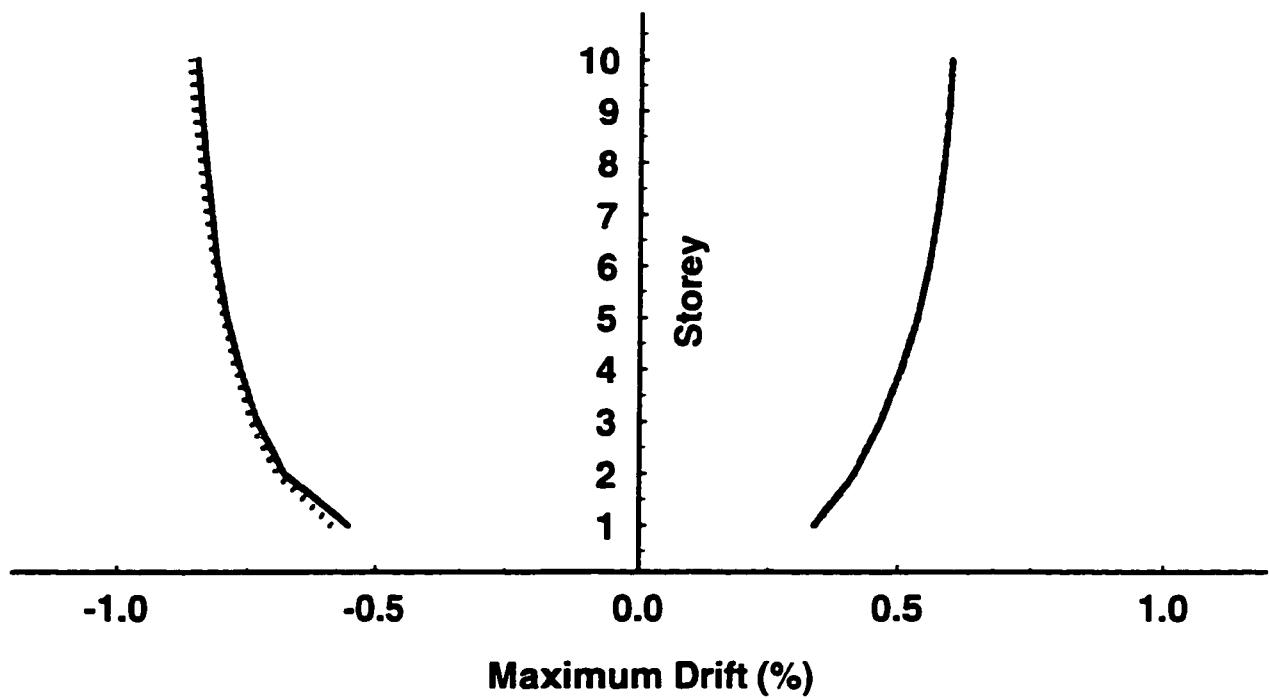


(a)

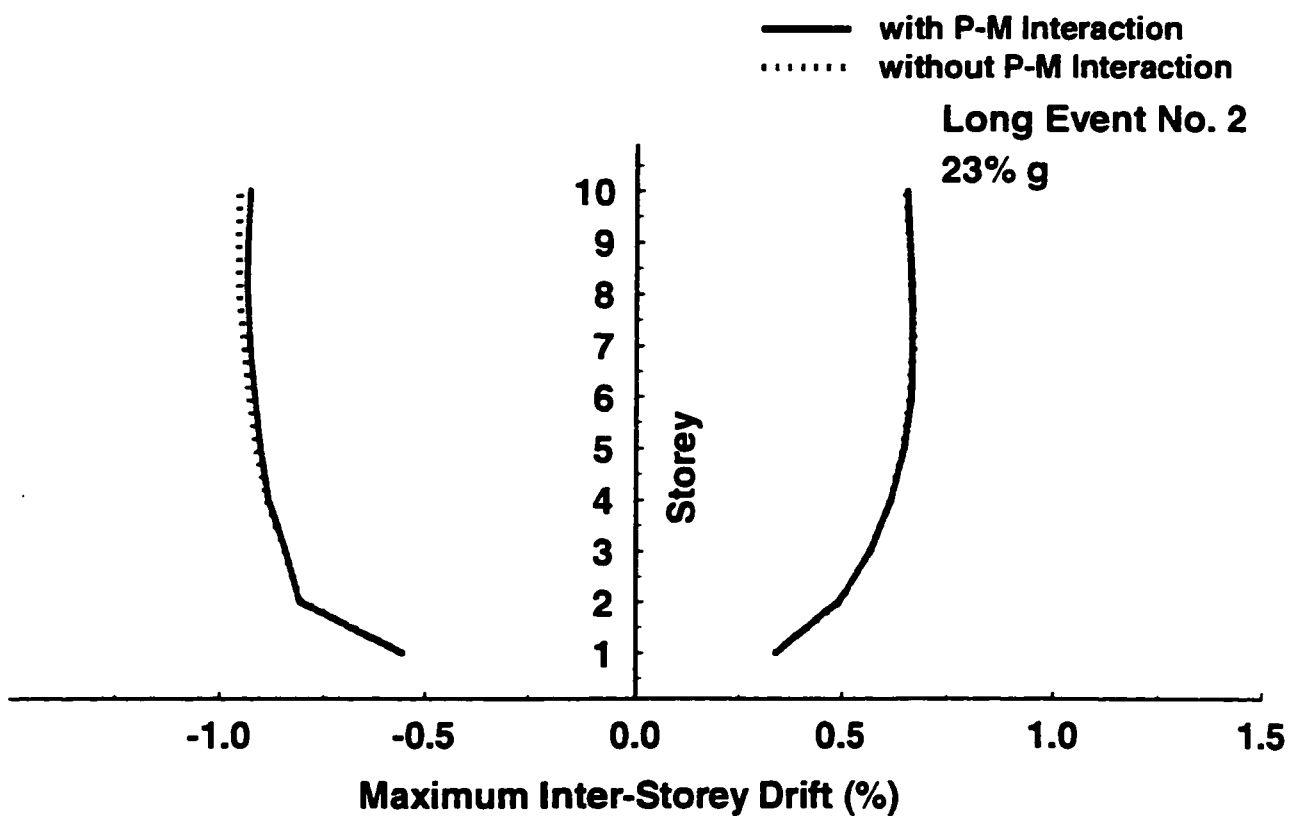


(b)

Fig. 4.28 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to 1940 El Centro E.Q. for Western Canada

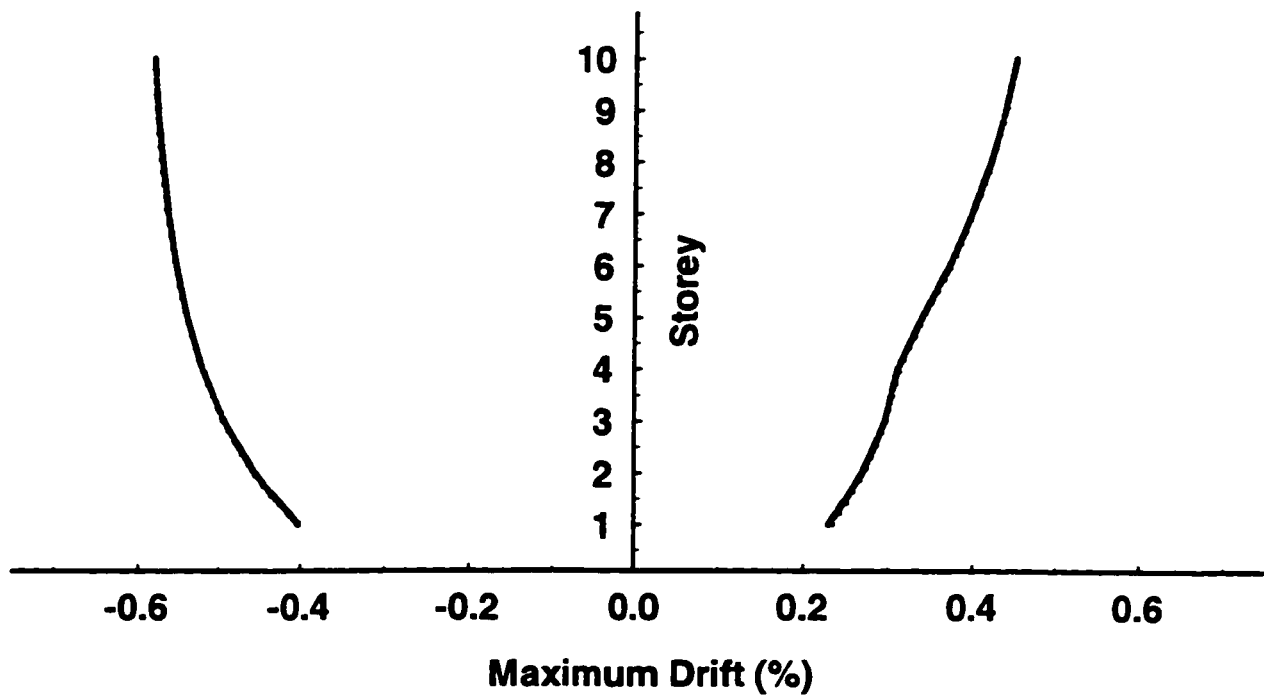


(a)

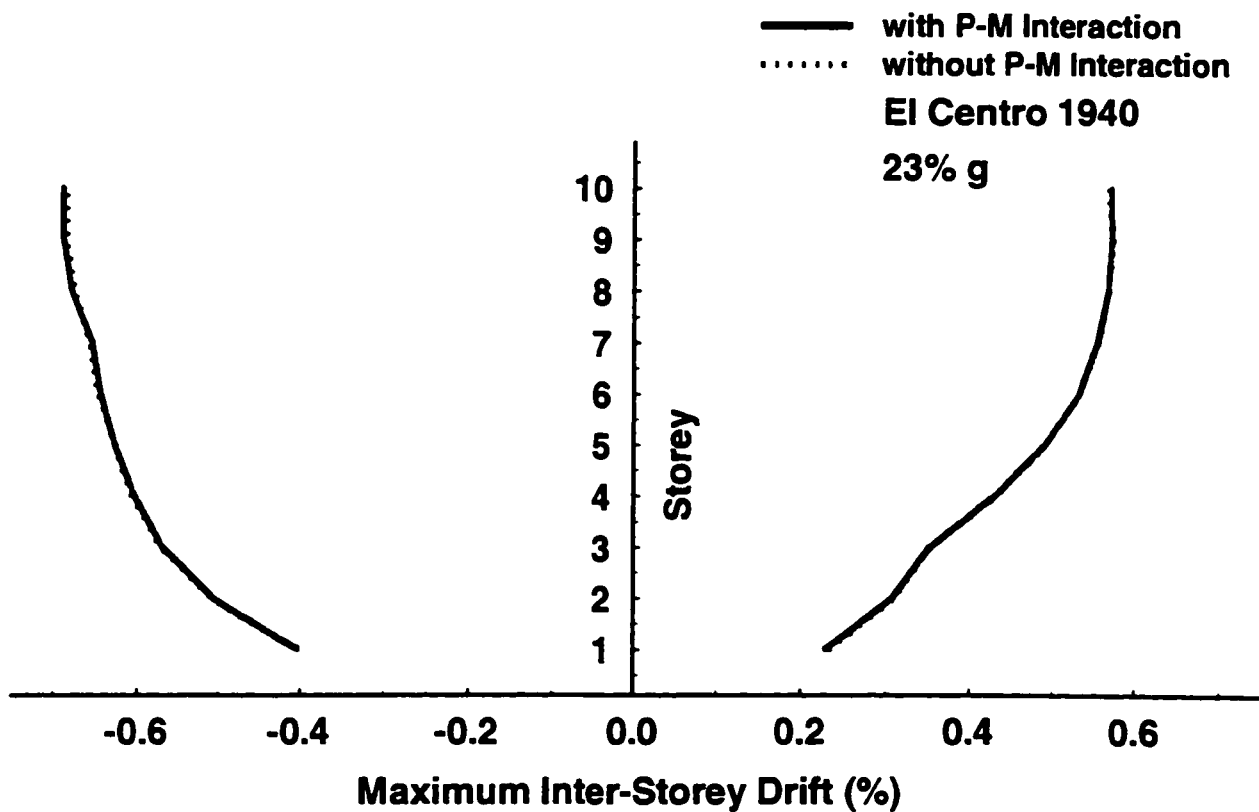


(b)

Fig. 4.29 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada

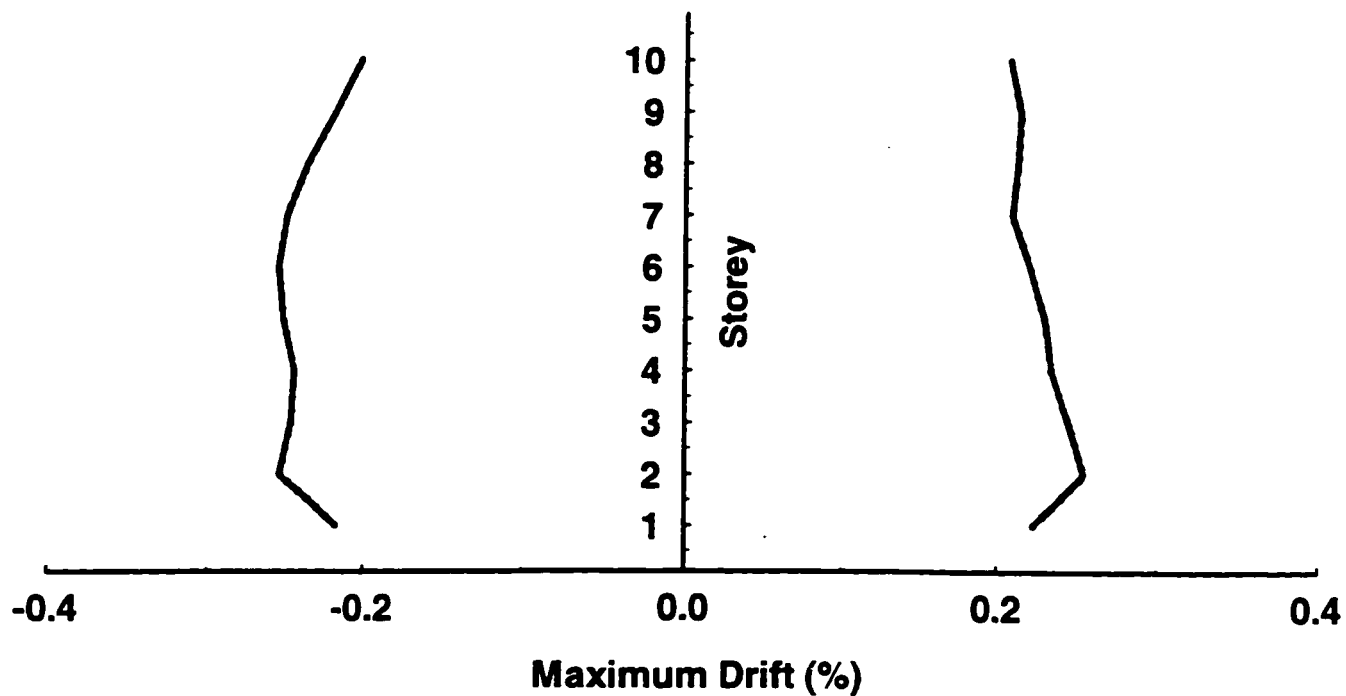


(a)

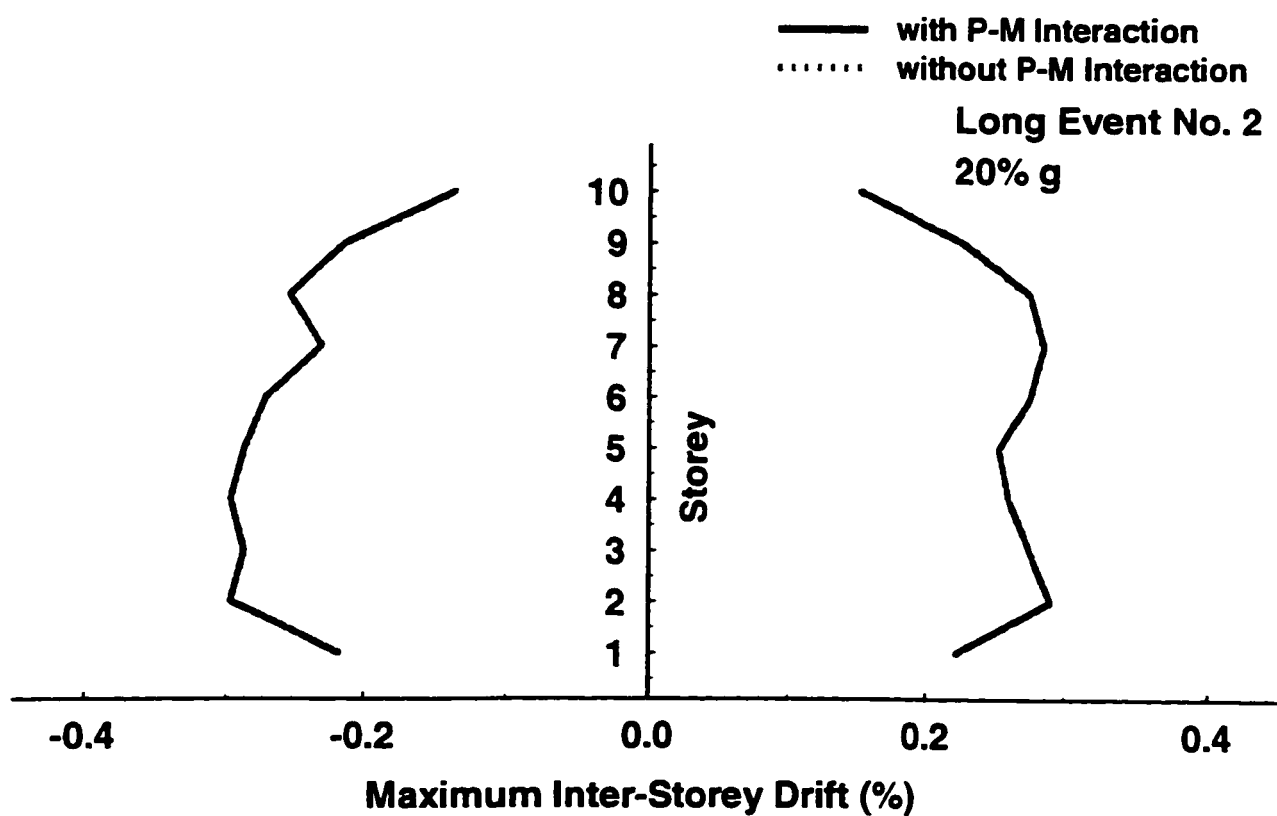


(b)

Fig. 4.30 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to 1940 El Centro E.Q. for Western Canada

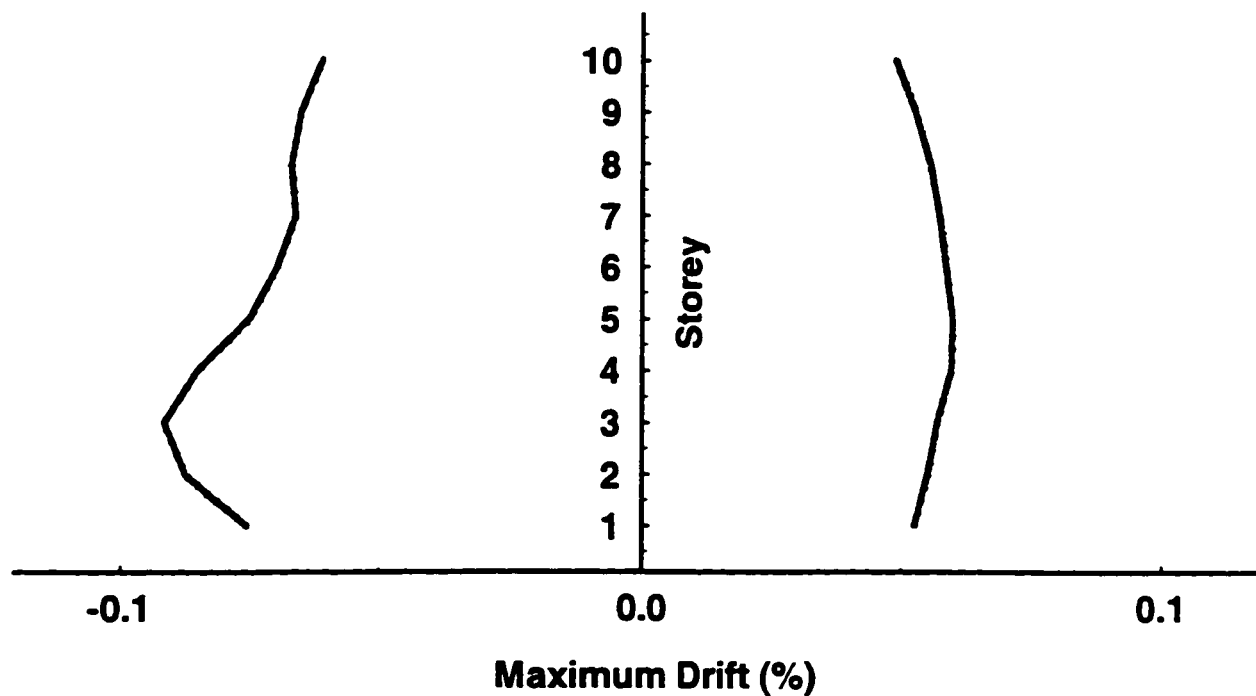


(a)

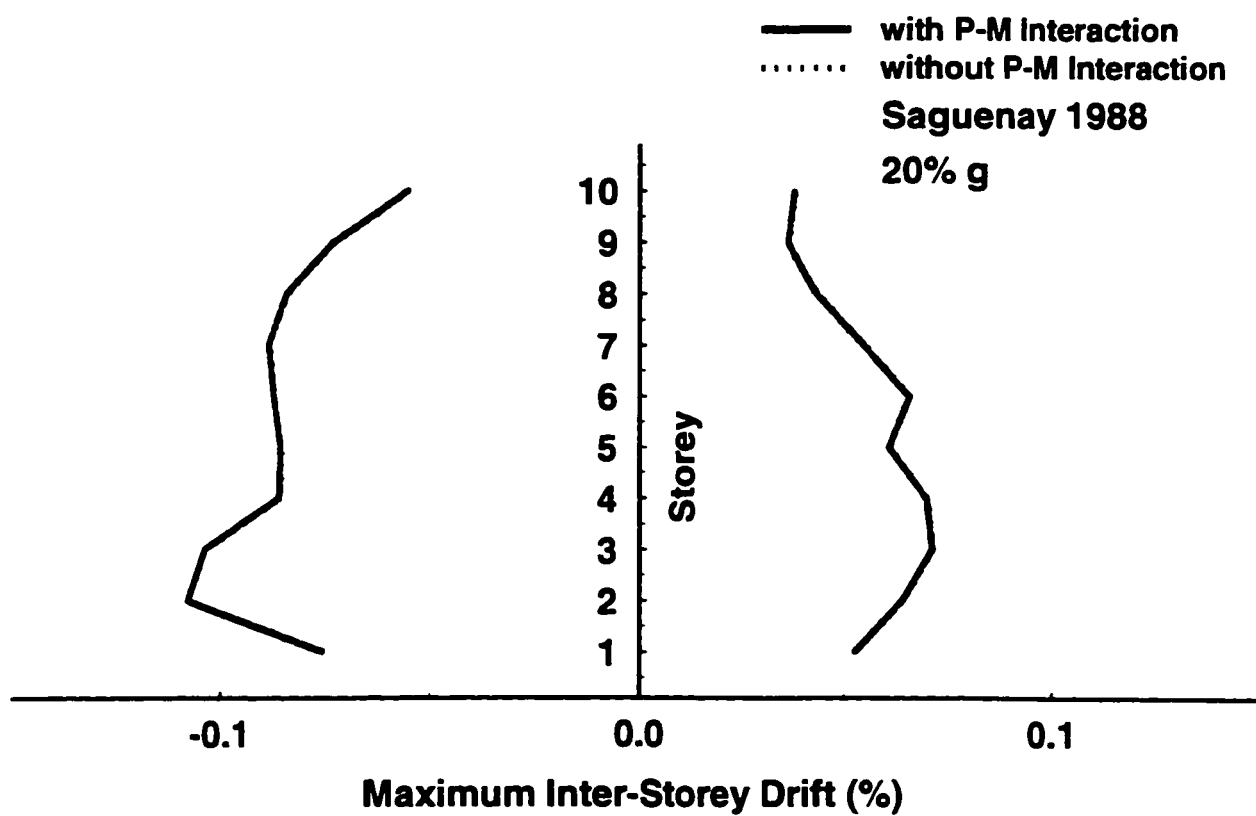


(b)

Fig. 4.31 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada

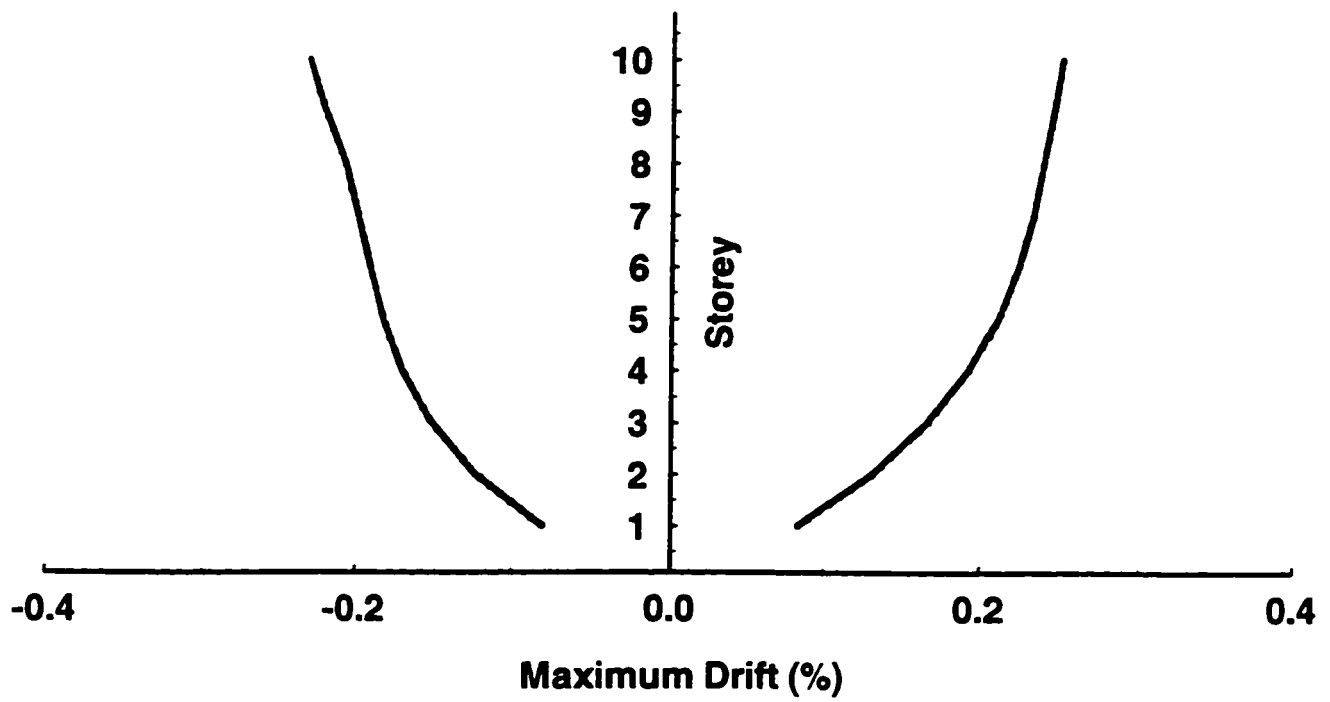


(a)

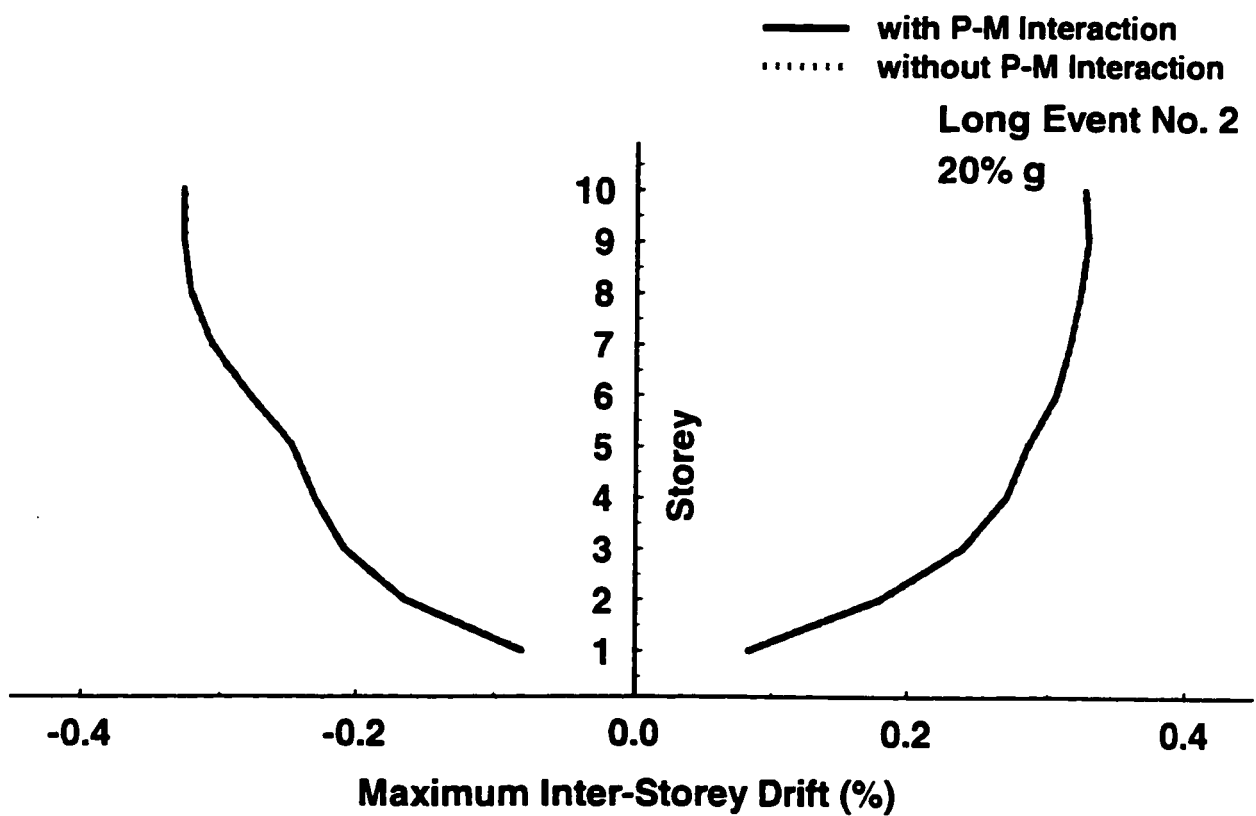


(b)

Fig. 4.32 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay E.Q. for Eastern Canada

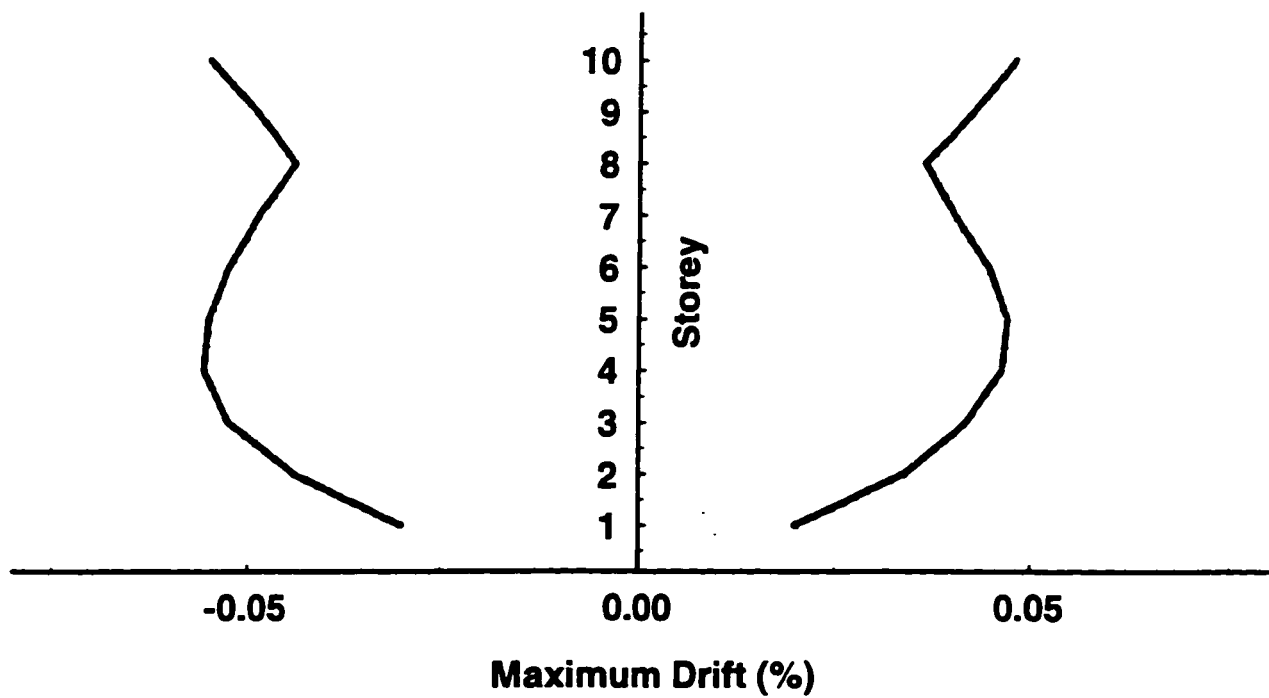


(a)

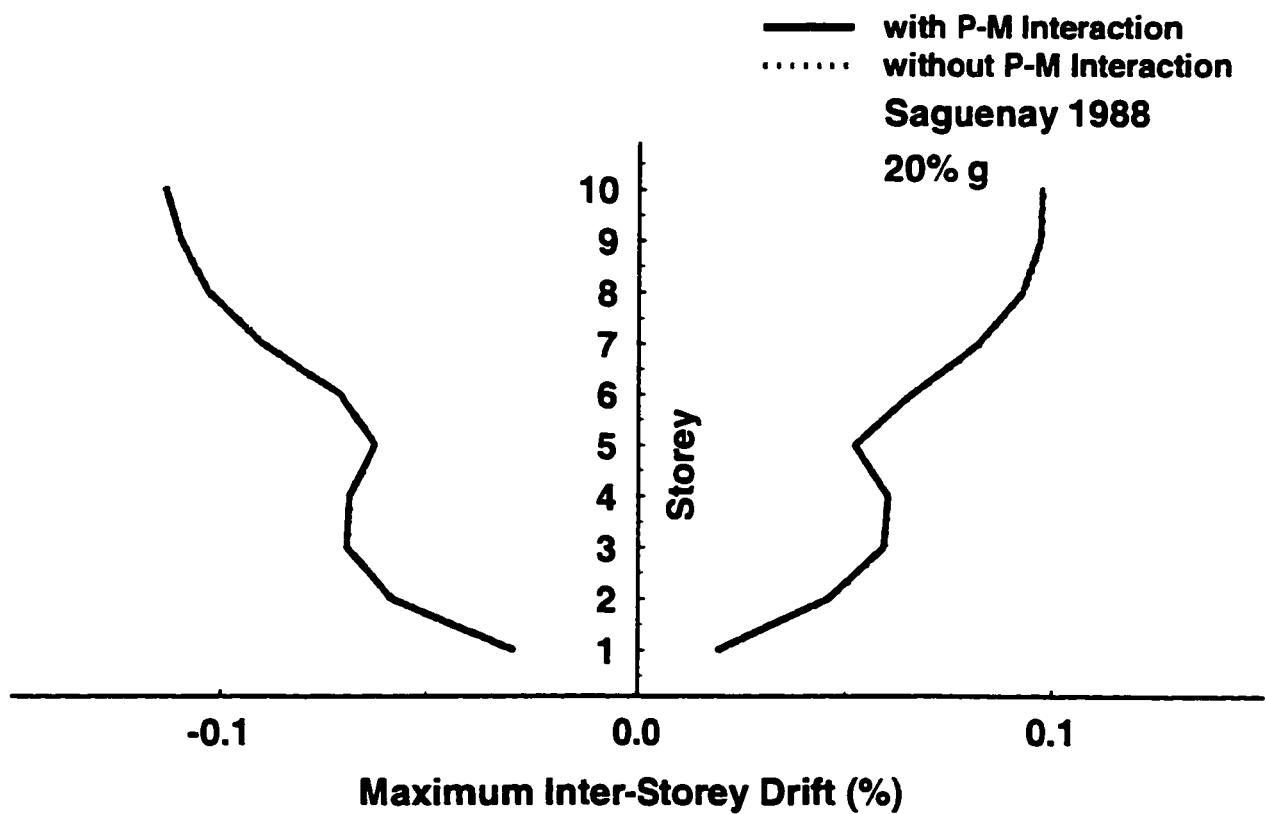


(b)

Fig. 4.33 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada



(a)



(b)

Fig. 4.34 Effect of P-M Interaction on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay E.Q. for Eastern Canada

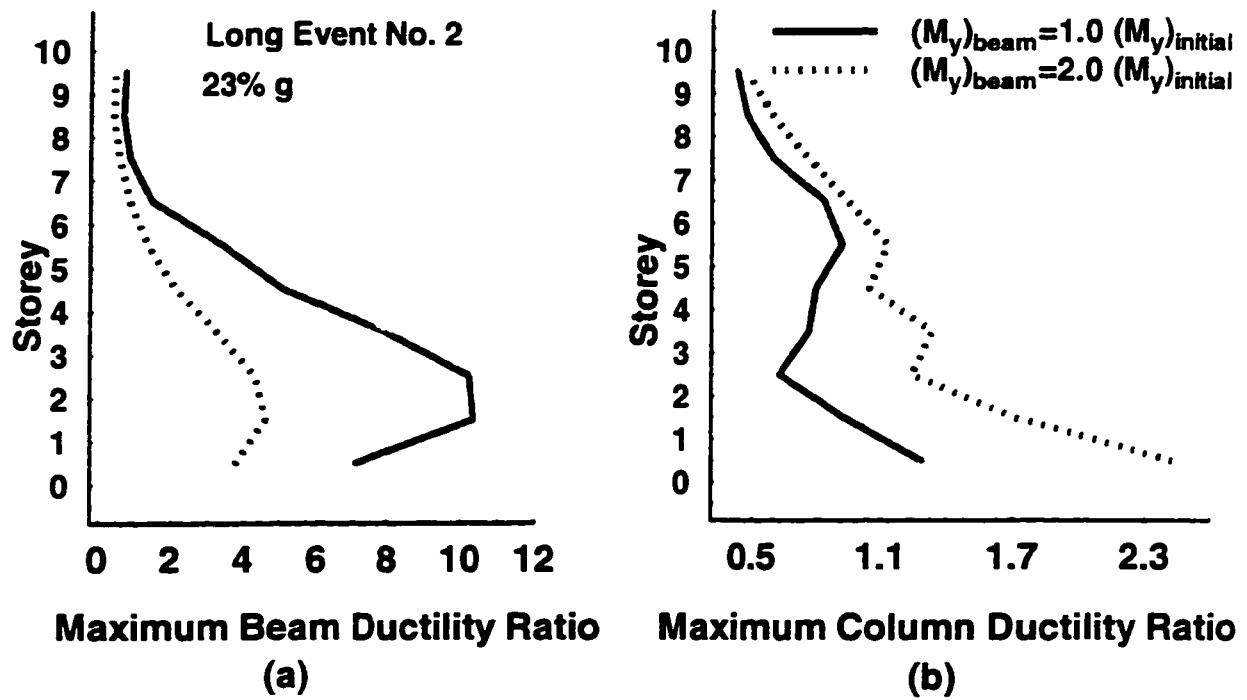


Fig. 4.35 Effect of P-M Interaction and Yield Moment on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

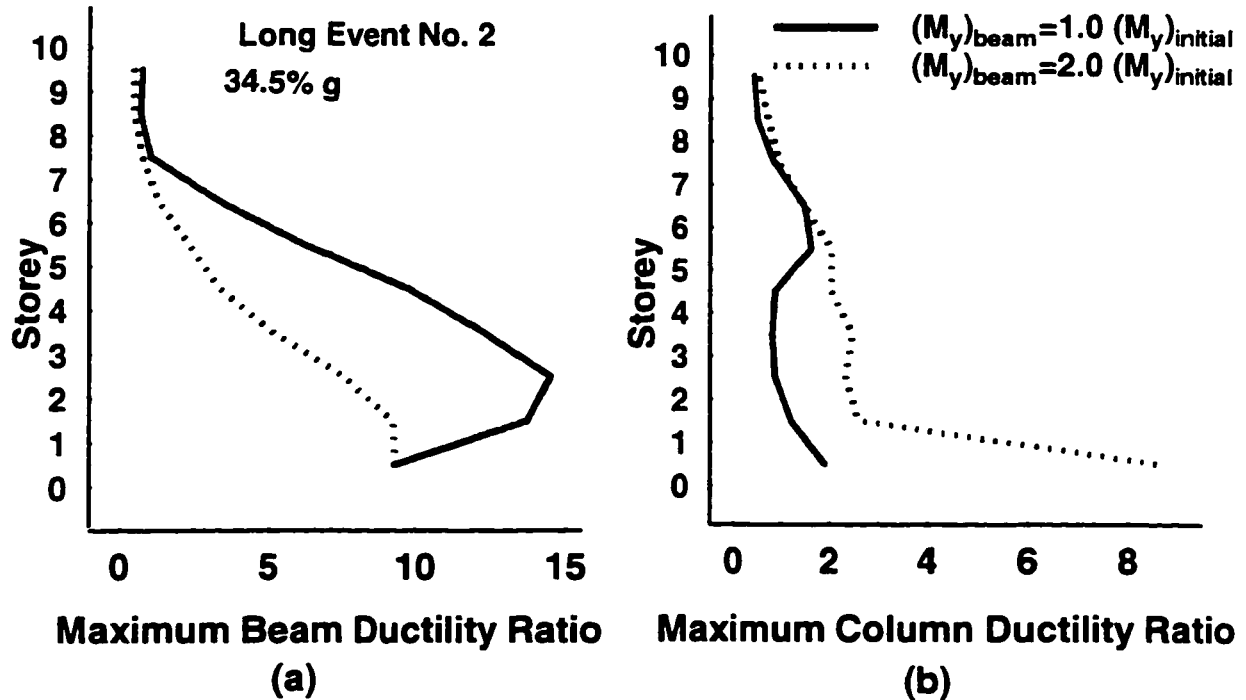
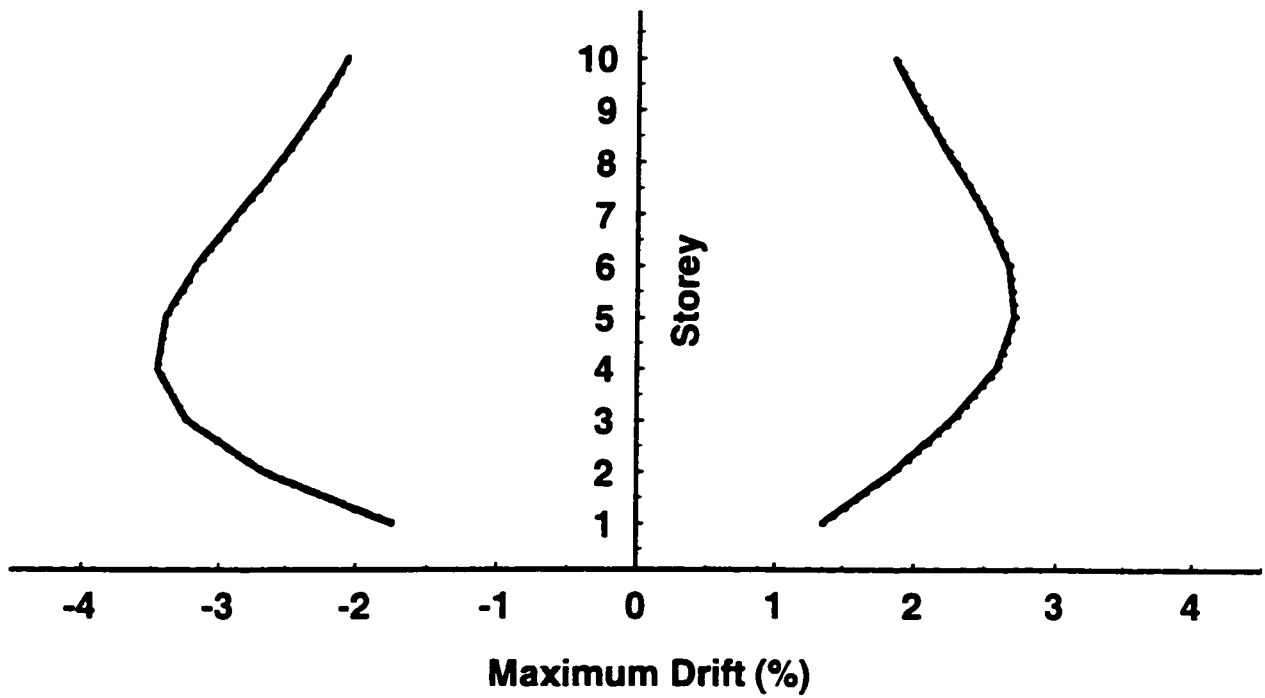
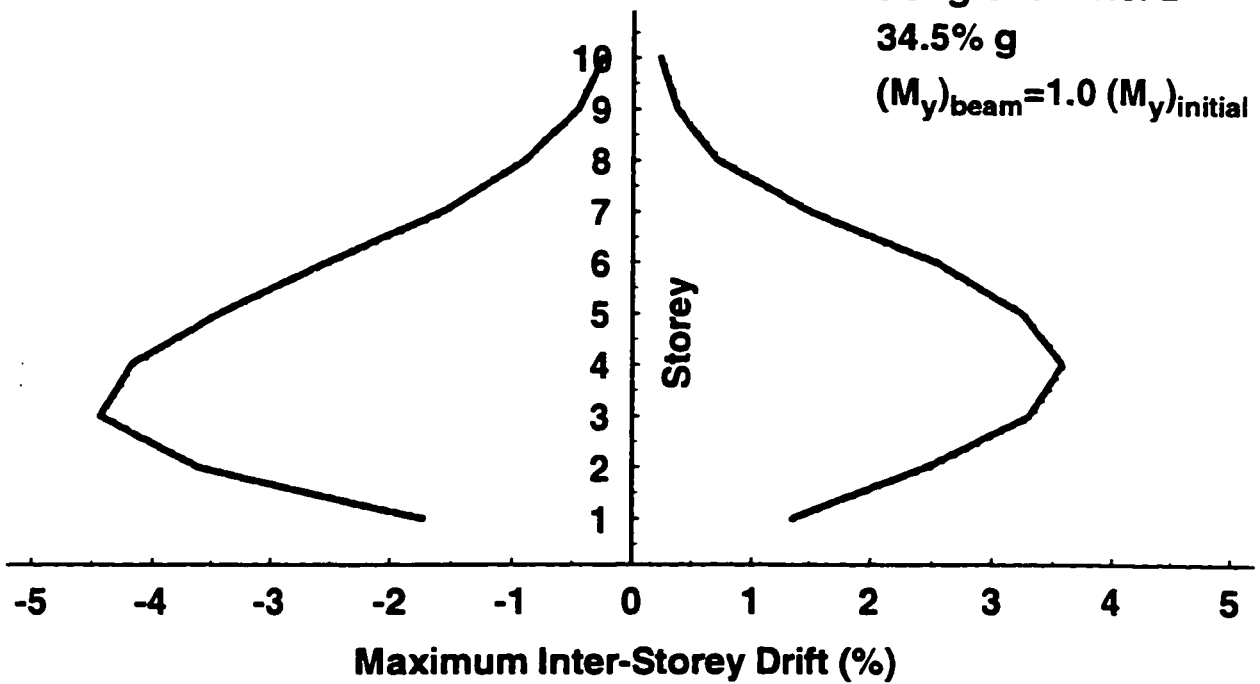


Fig. 4.36 Effect of P-M Interaction and Yield Moment on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



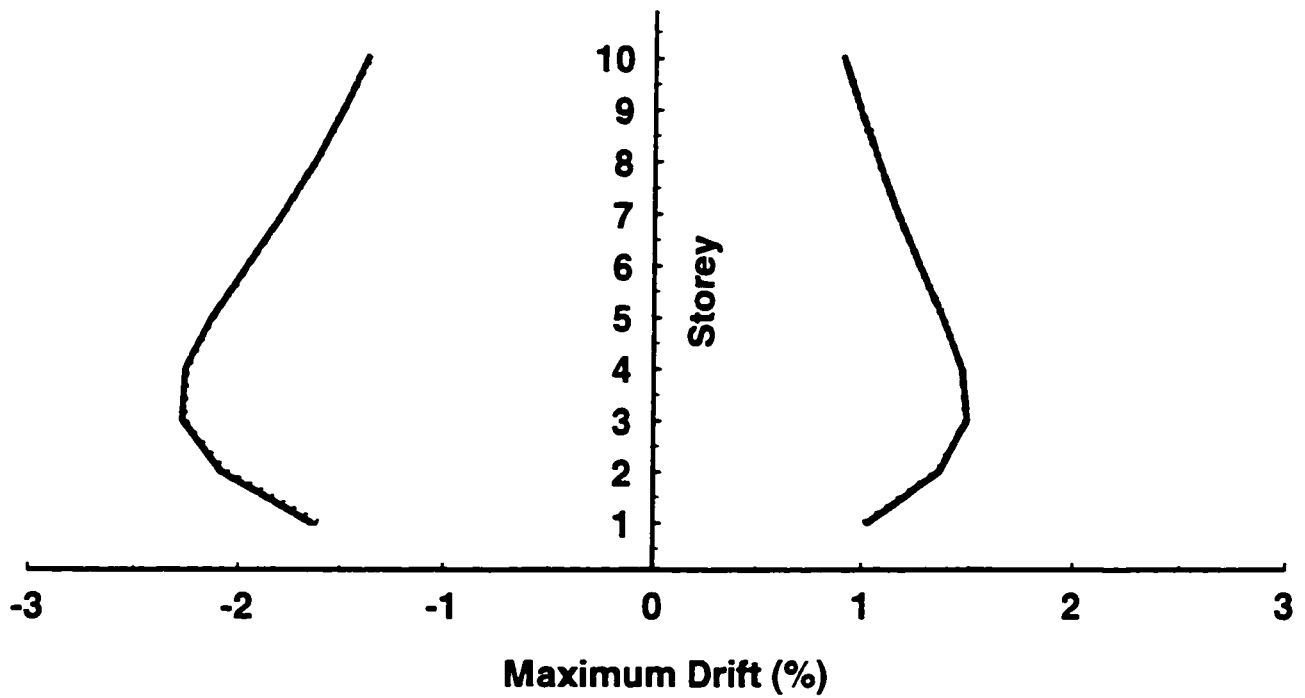
(a)

— with P-M Interaction
 without P-M Interaction
Long Event No. 2
34.5% g
 $(M_y)_{\text{beam}} = 1.0 (M_y)_{\text{initial}}$

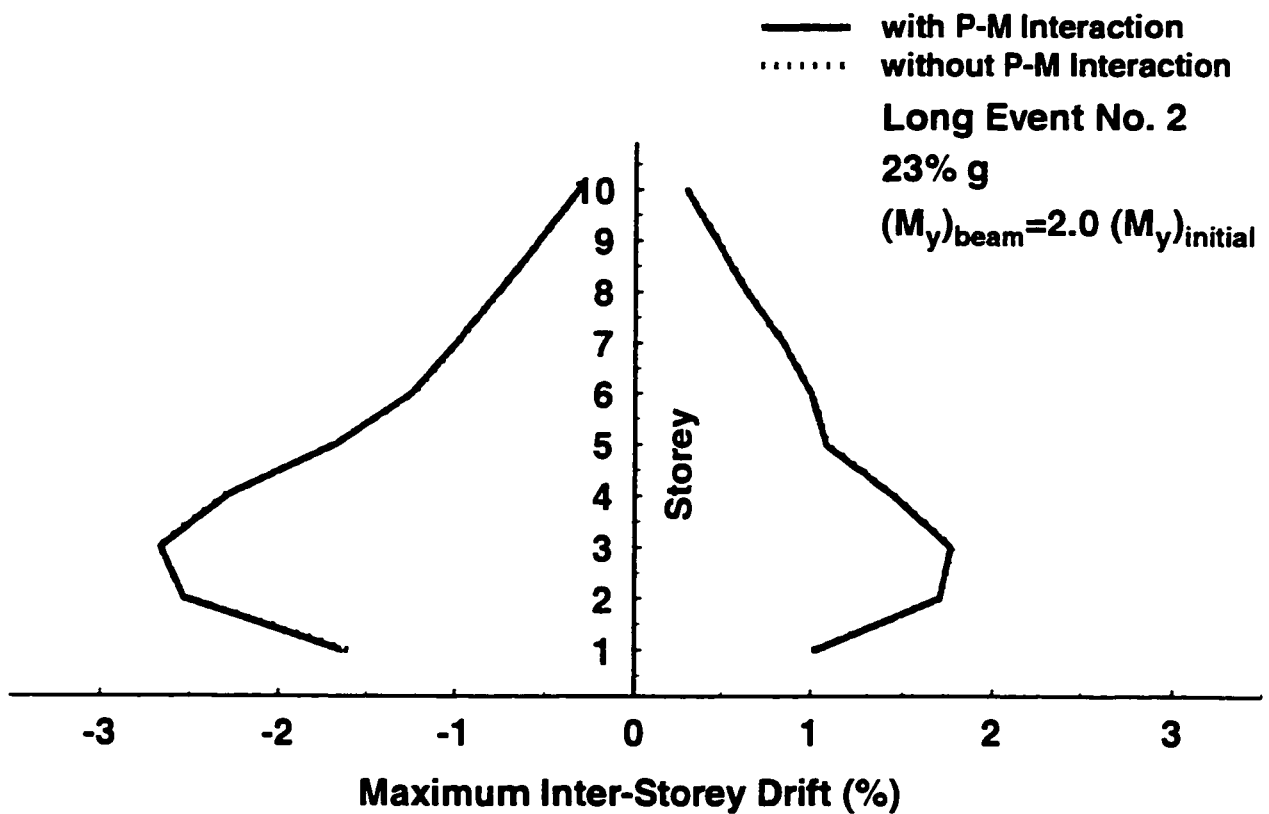


(b)

Fig. 4.37 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

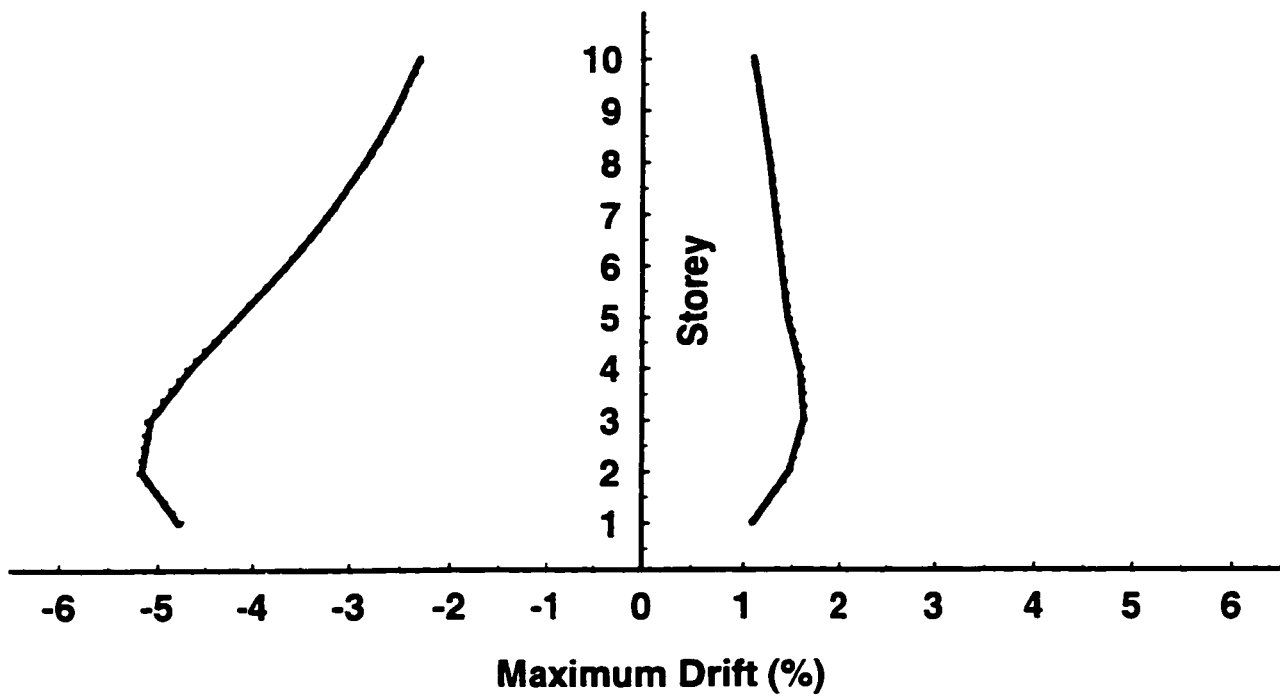


(a)



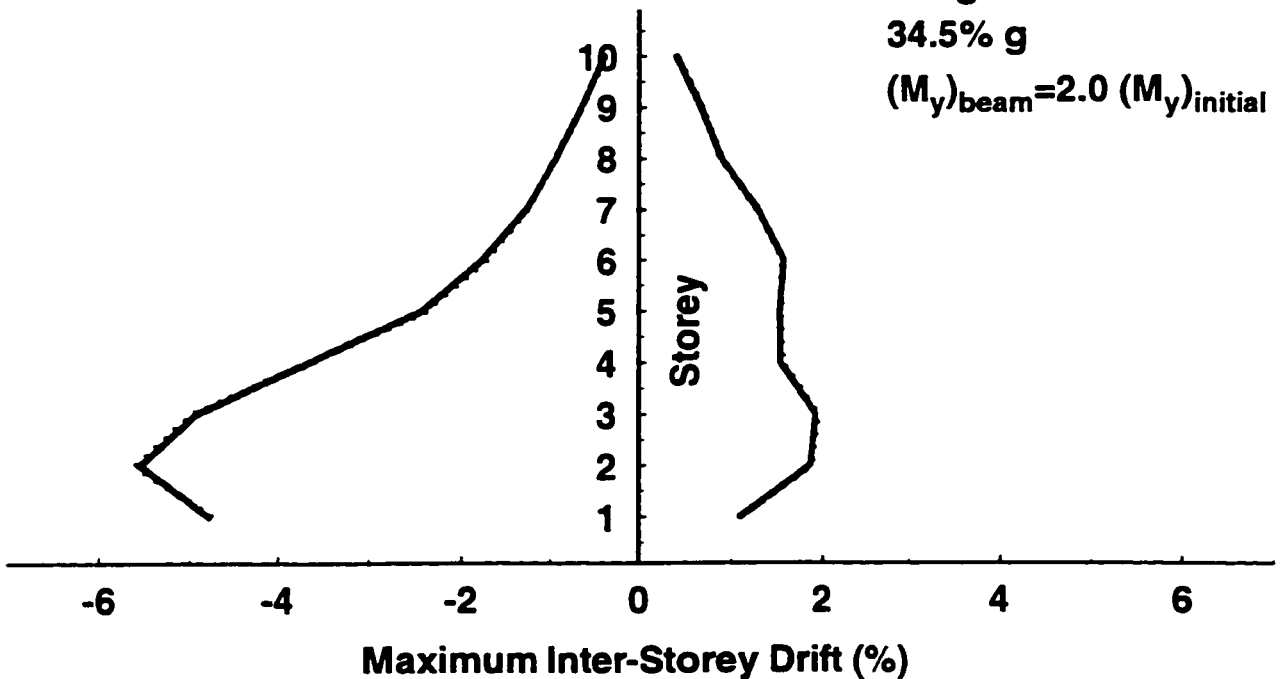
(b)

Fig. 4.38 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



(a)

— with P-M Interaction
 without P-M Interaction
Long Event No. 2
34.5% g
 $(M_y)_{\text{beam}} = 2.0 (M_y)_{\text{initial}}$



(b)

Fig. 4.39 Effect of P-M Interaction on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

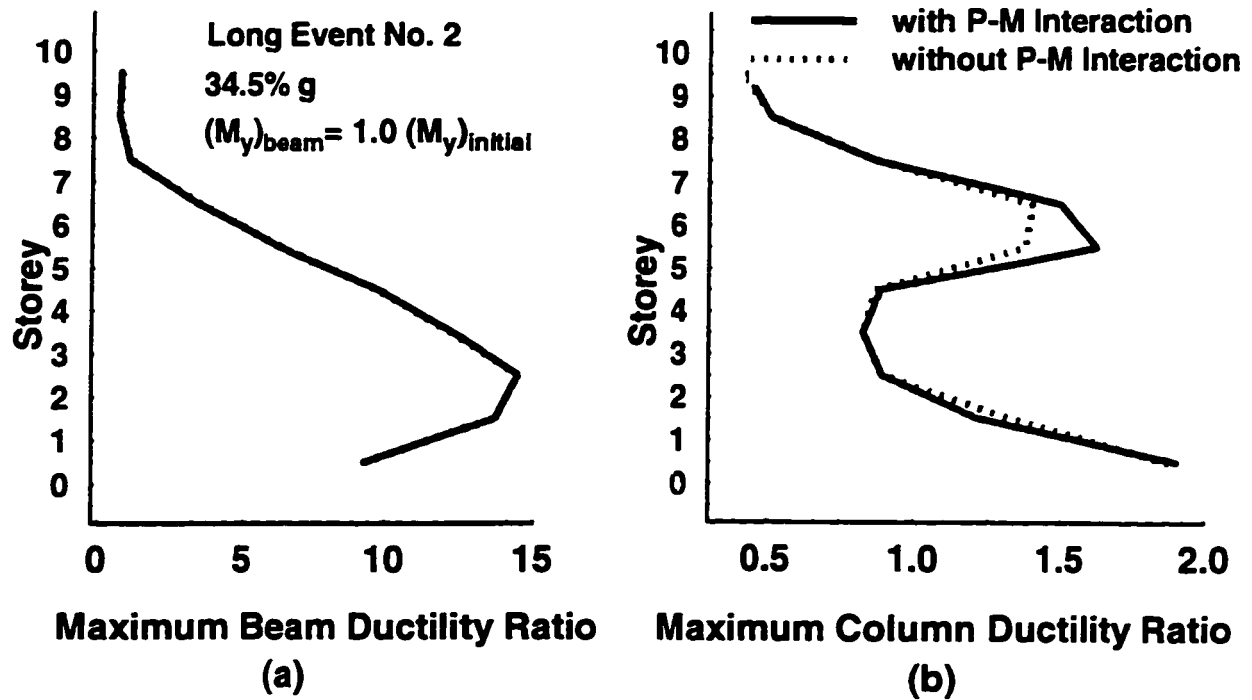


Fig. 4.40 Effect of P-M Interaction on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

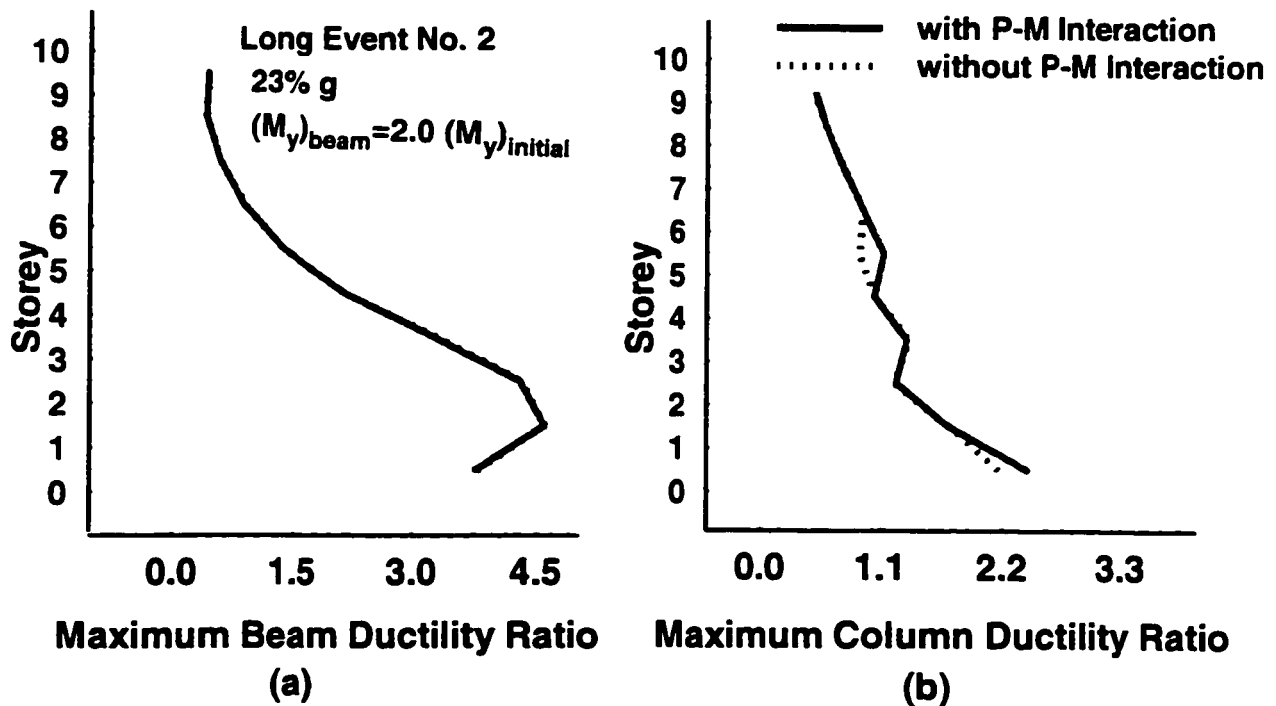


Fig. 4.41 Effect of P-M Interaction on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

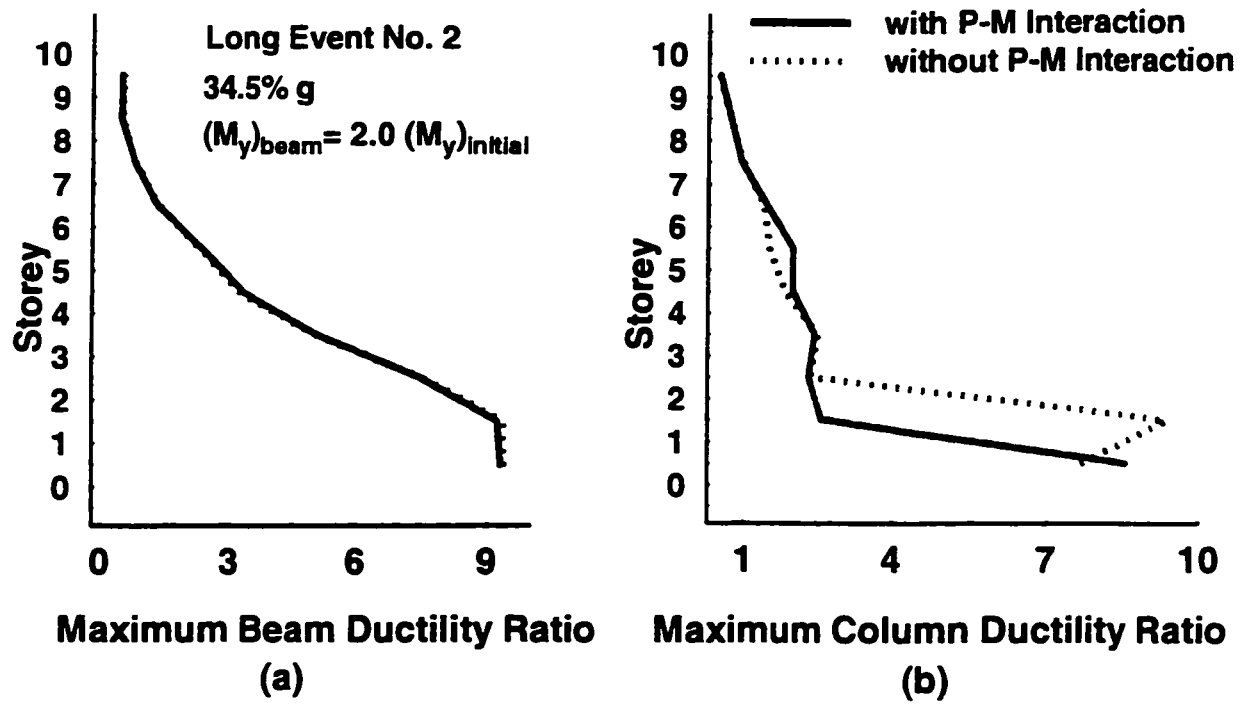


Fig. 4.42 Effect of P-M Interaction on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

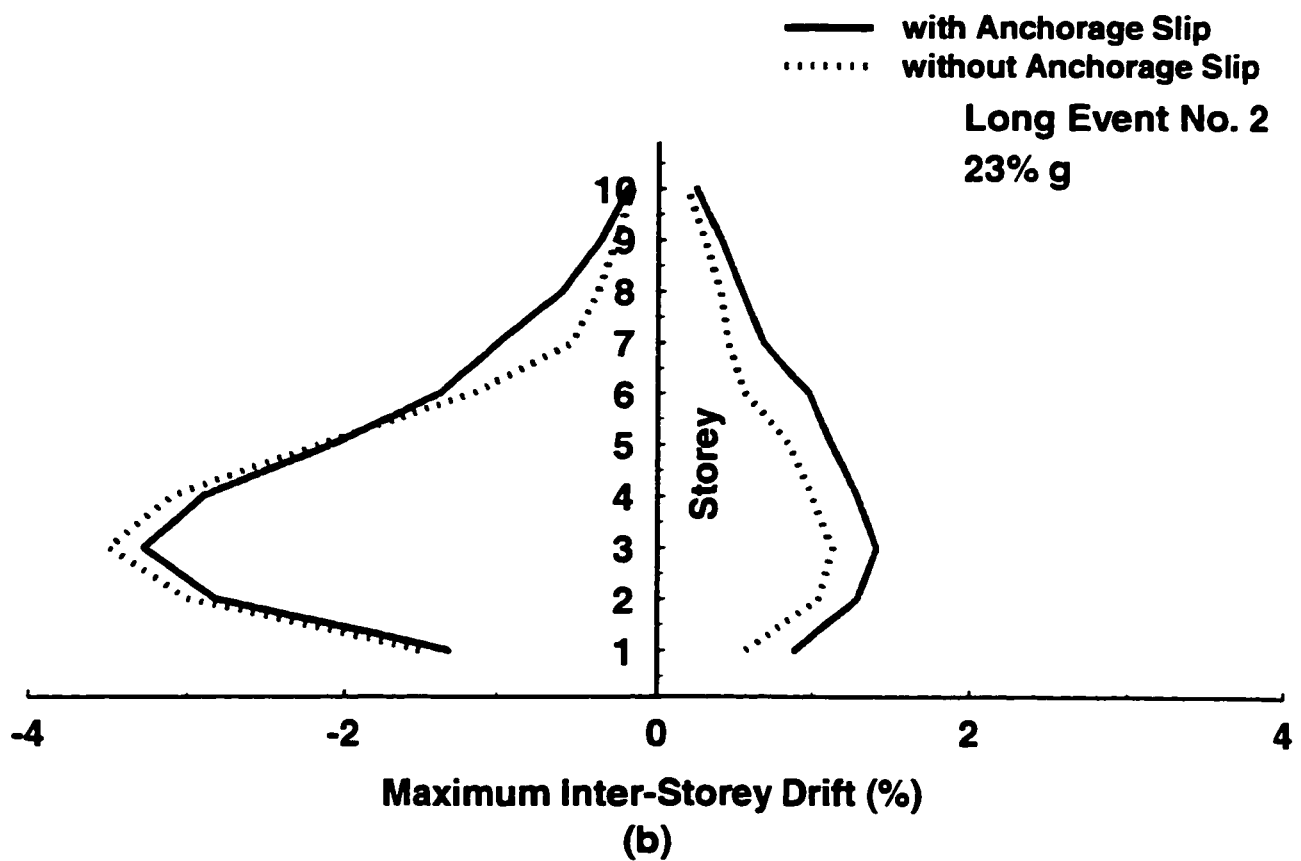
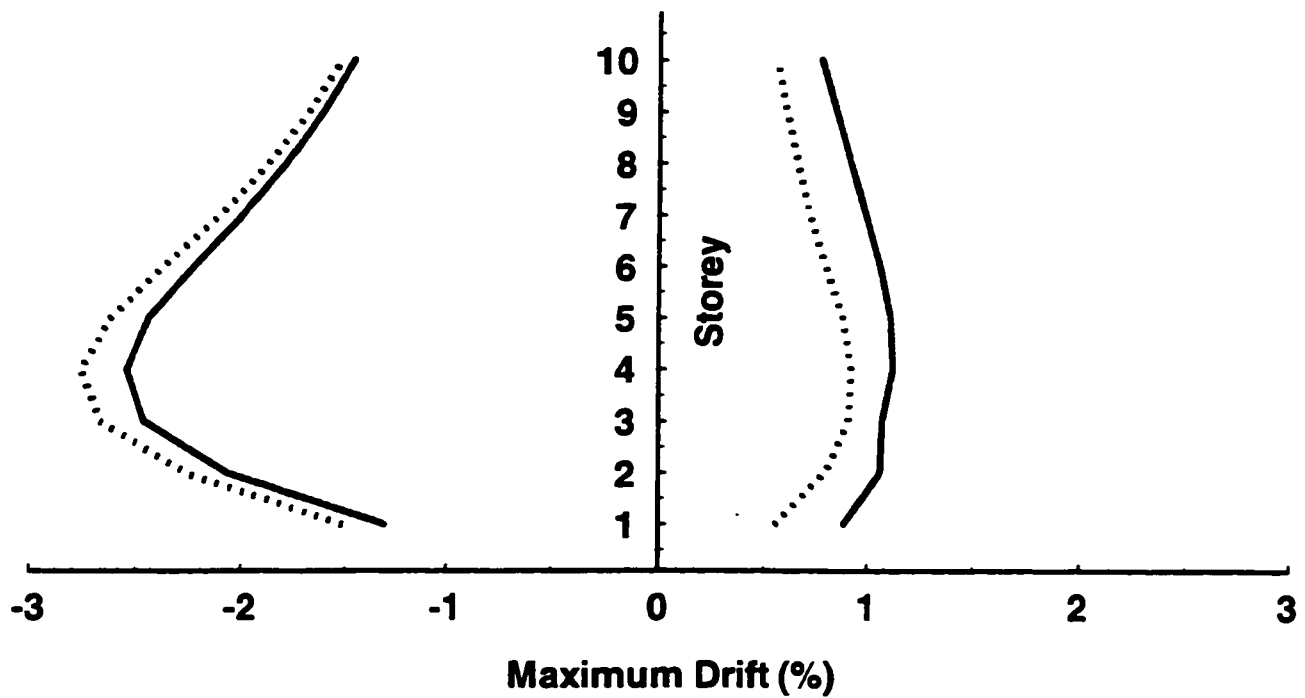
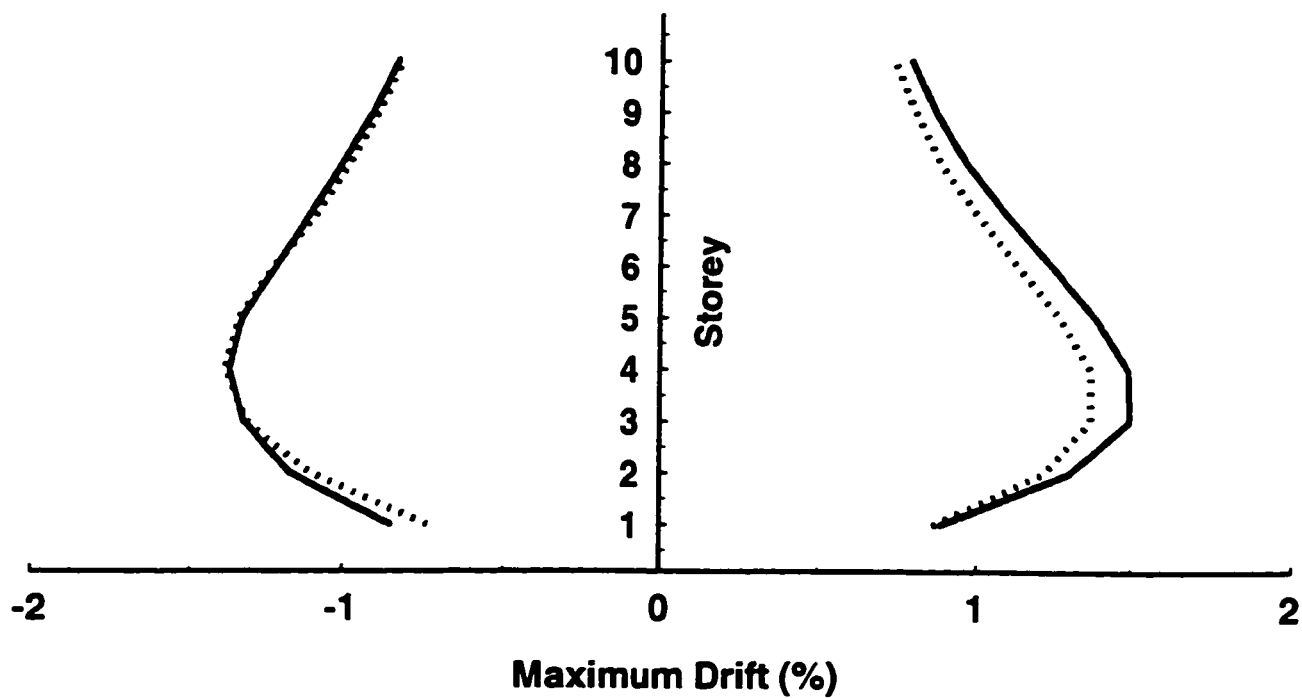
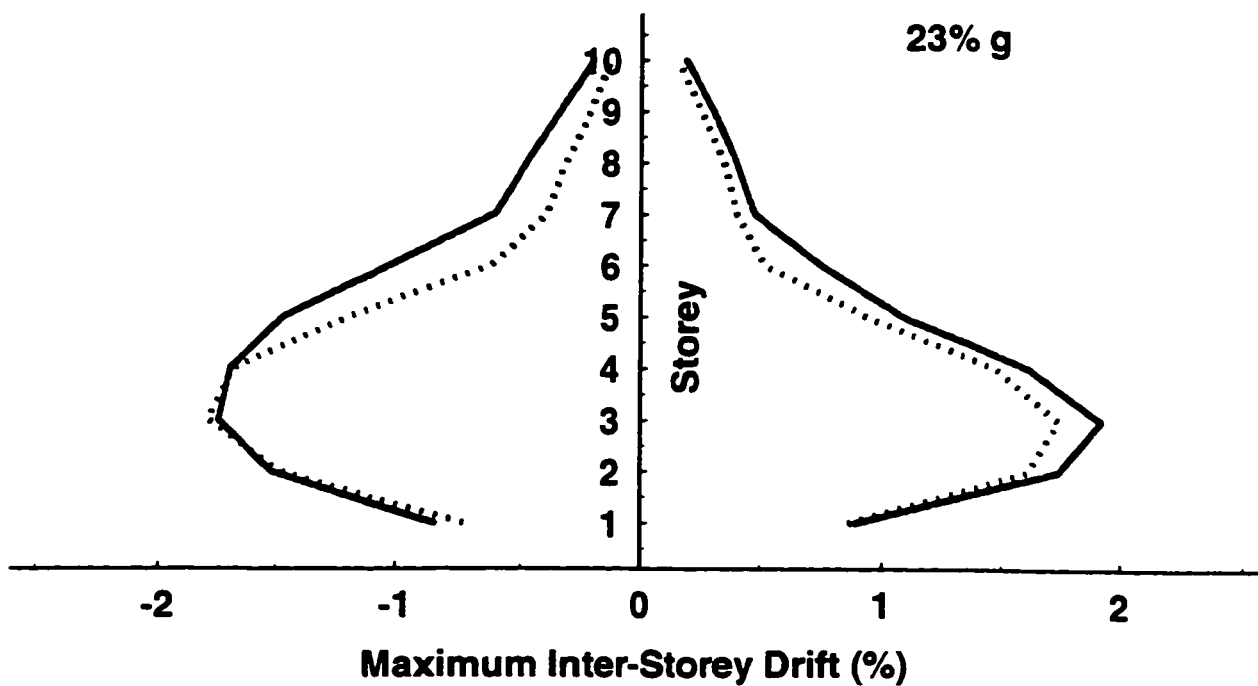


Fig. 4.43 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



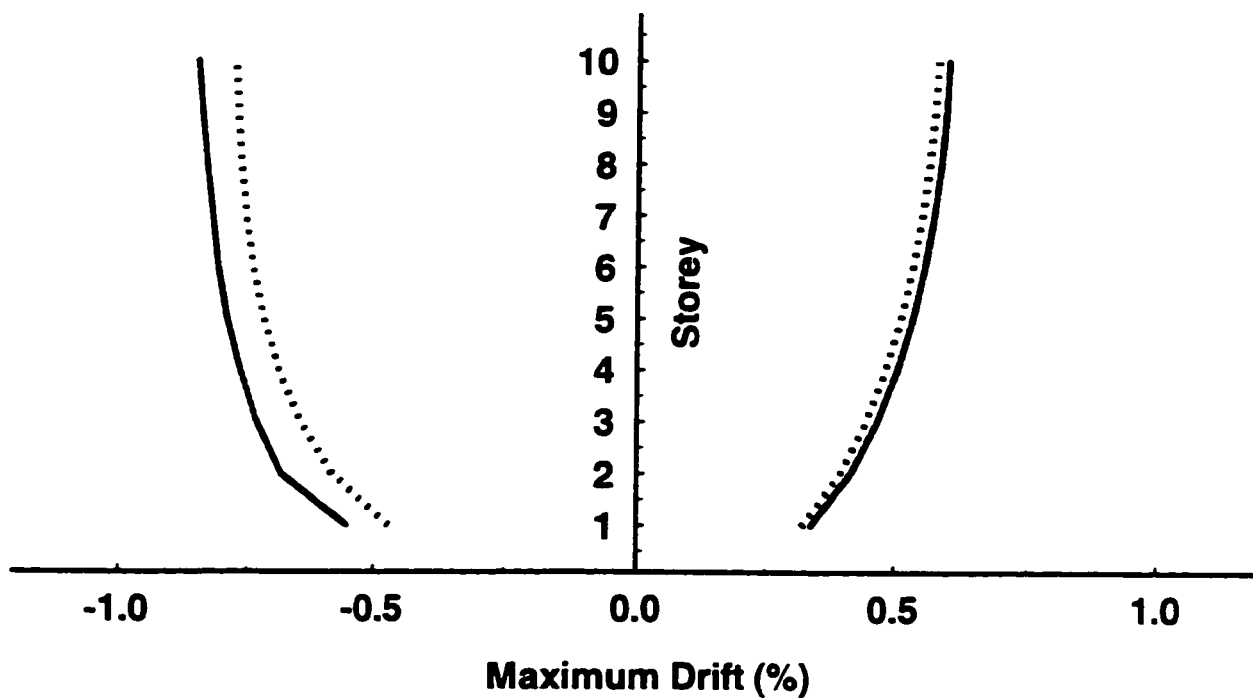
(a)

— with Anchorage Slip
 without Anchorage Slip
 El Centro 1940
 23% g



(b)

Fig. 4.44 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada

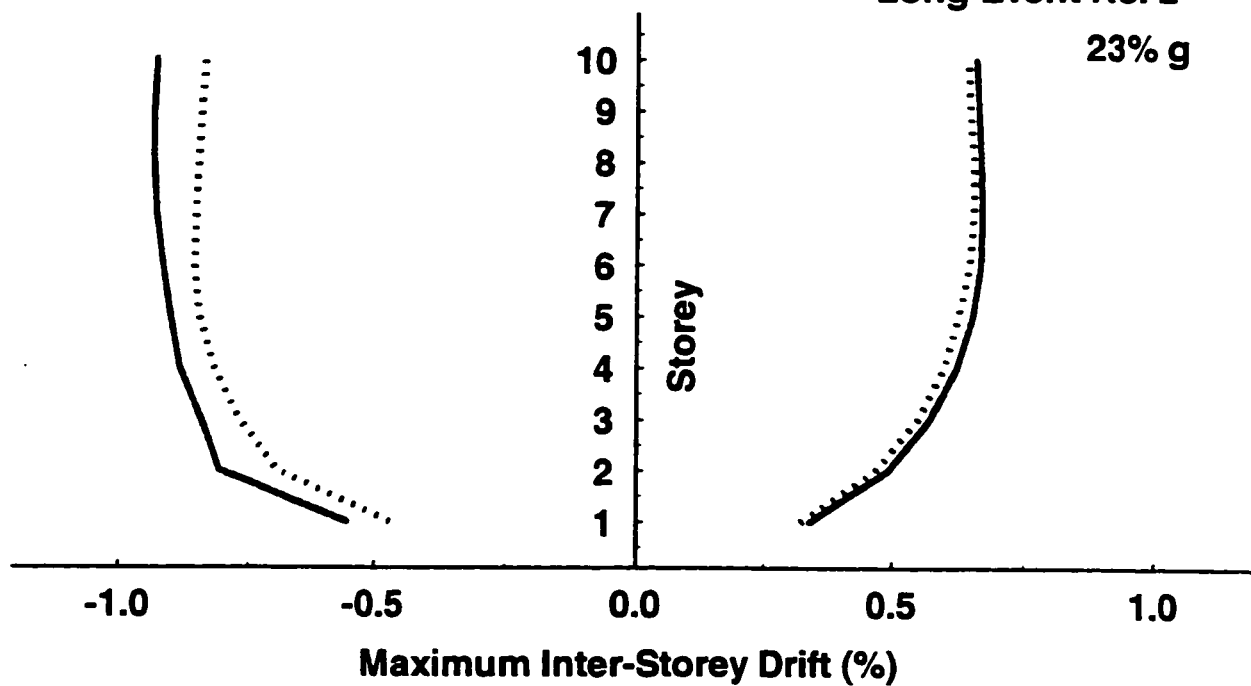


(a)

— with Anchorage Slip
 without Anchorage Slip

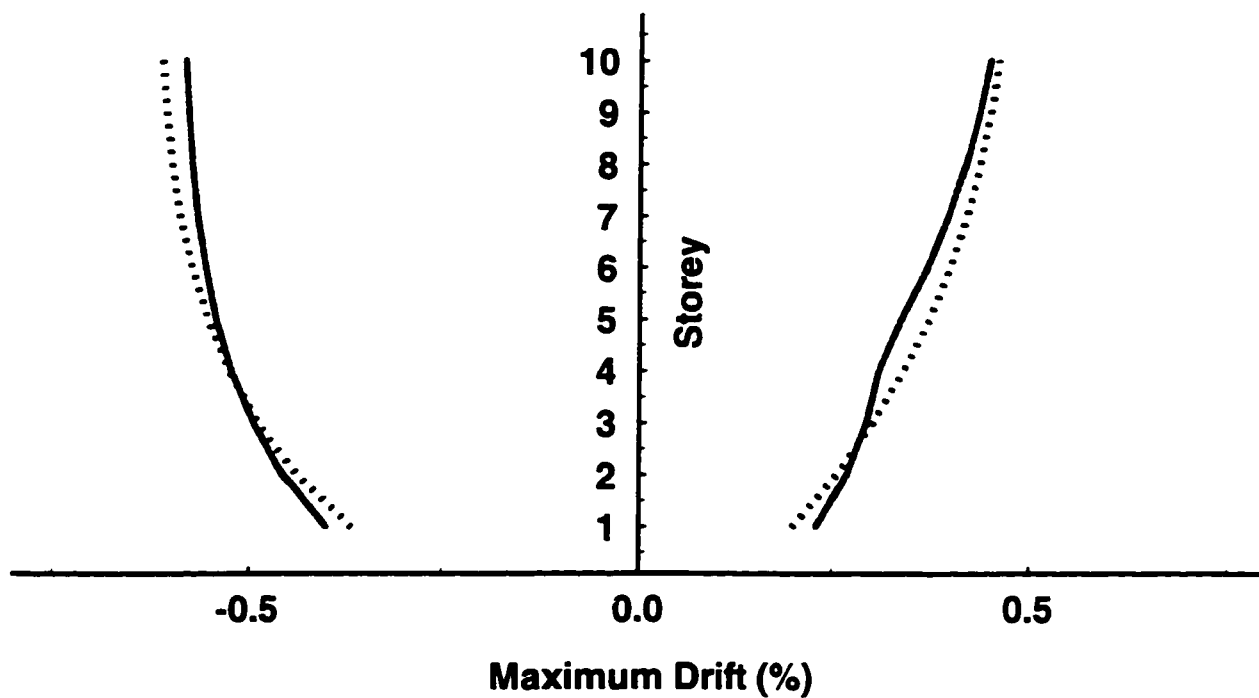
Long Event No. 2

23% g



(b)

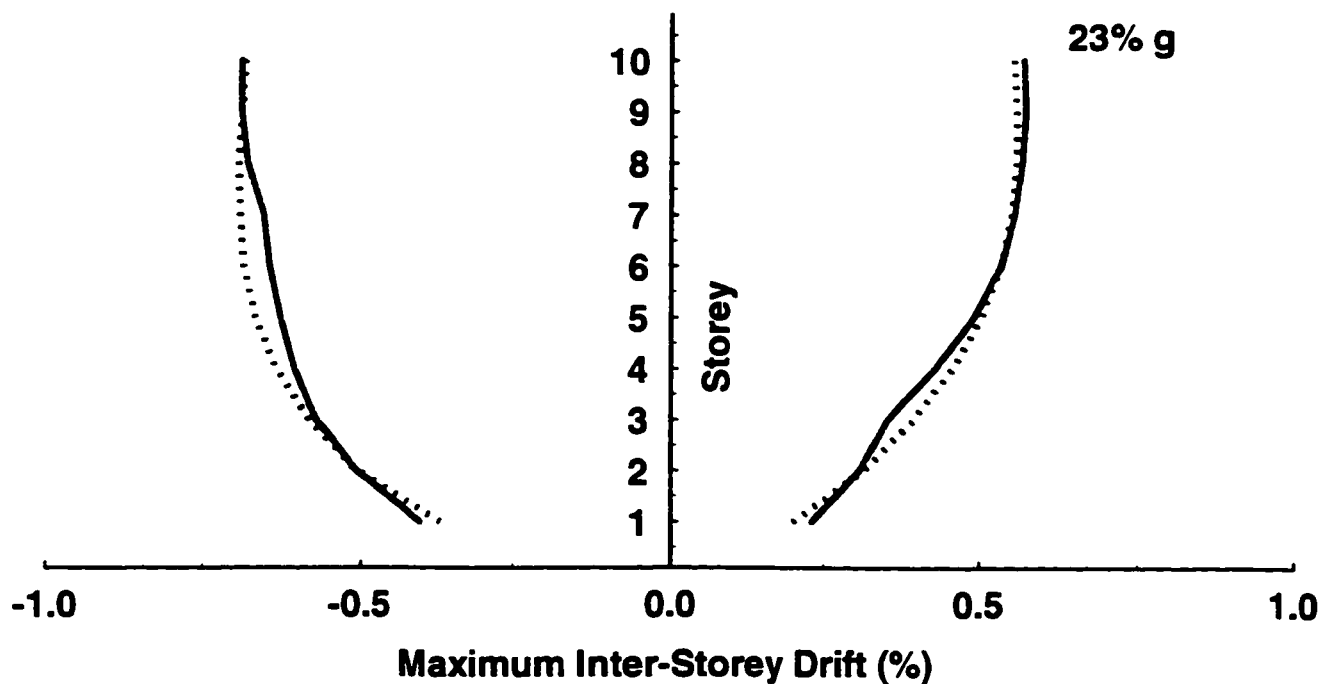
Fig. 4.45 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada



(a)

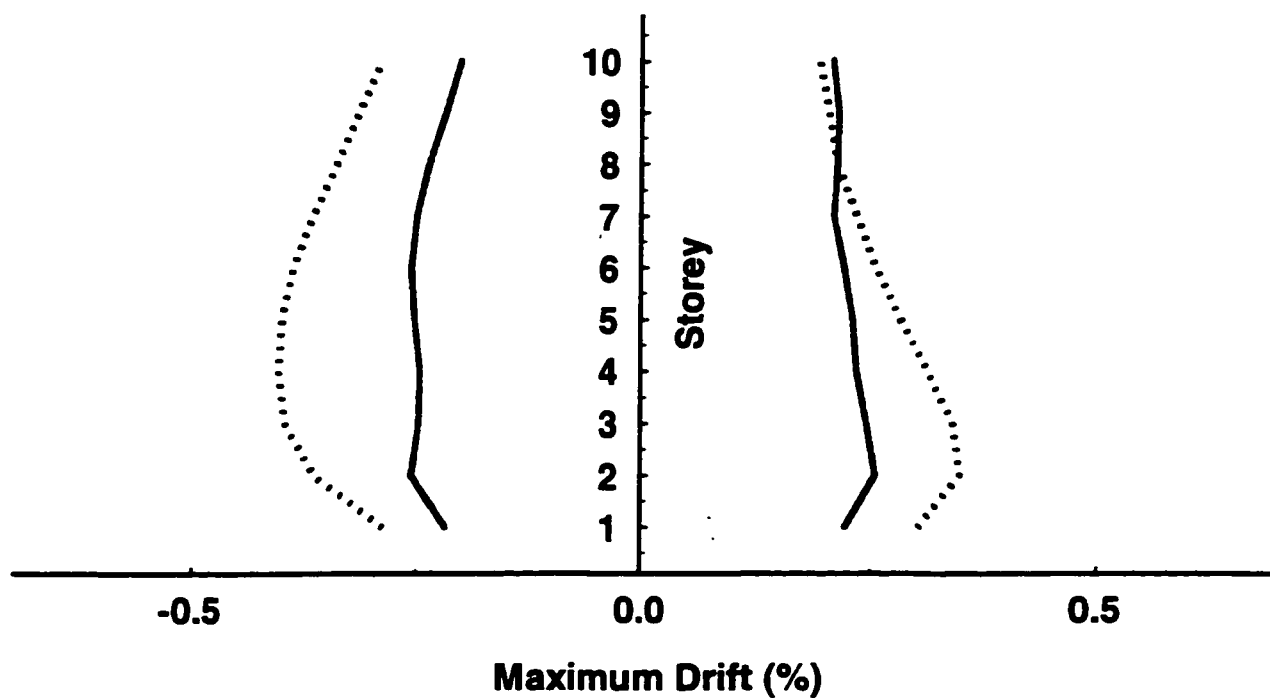
— with Anchorage Slip
 without Anchorage Slip
 El Centro 1940

23% g

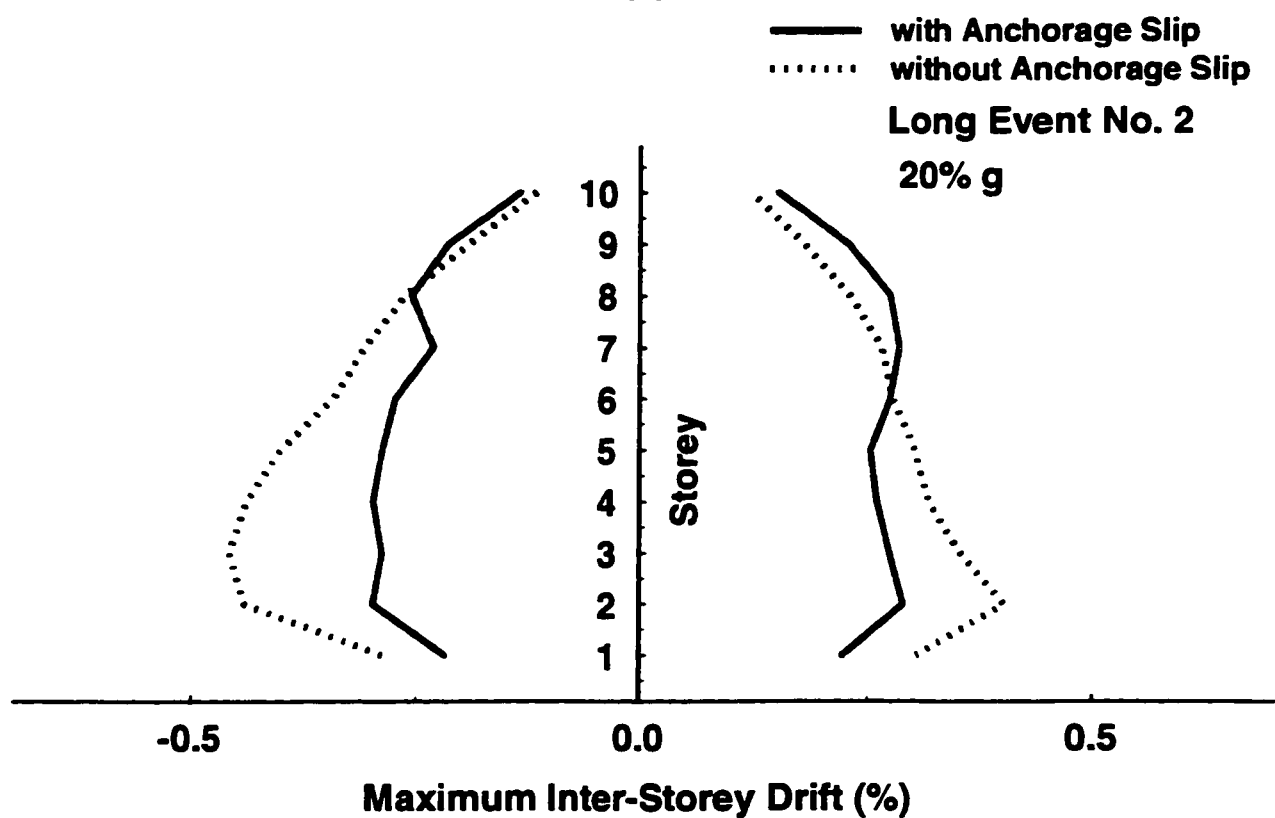


(b)

Fig. 4.46 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to 1940 El Centro for Western Canada

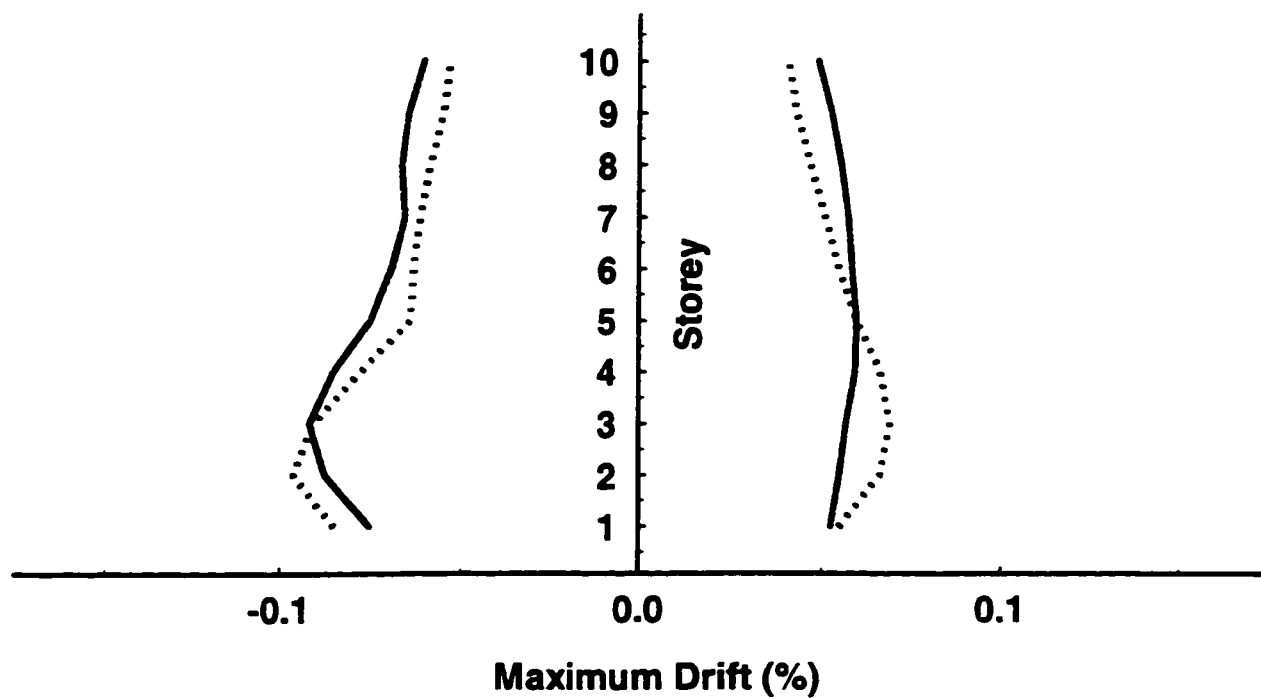


(a)



(b)

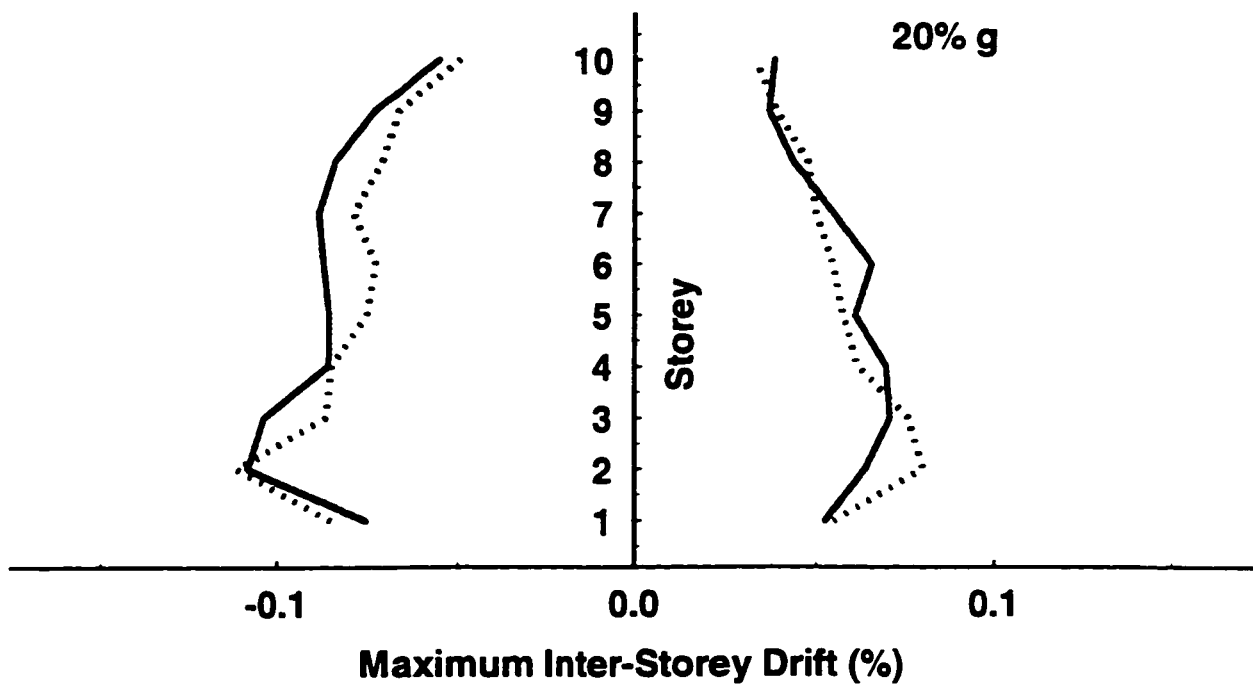
Fig. 4.47 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada



(a)

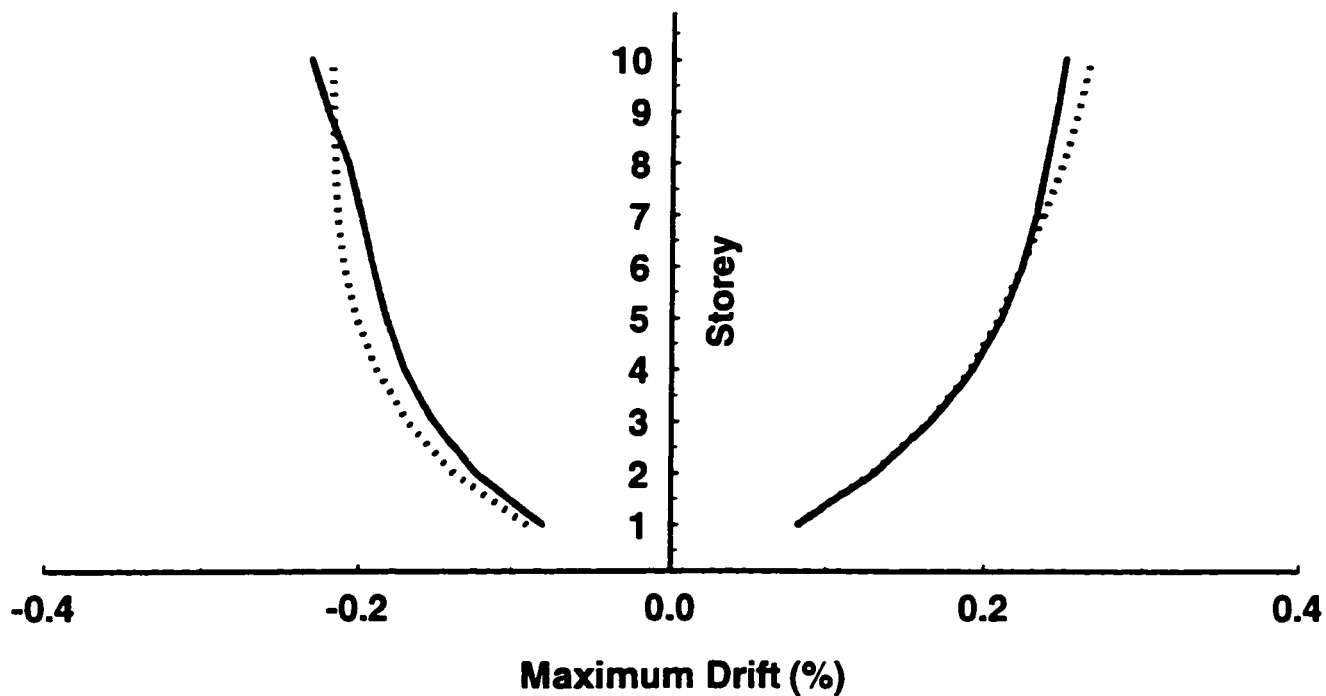
— with Anchorage Slip
 without Anchorage Slip
 Saguenay 1988

20% g

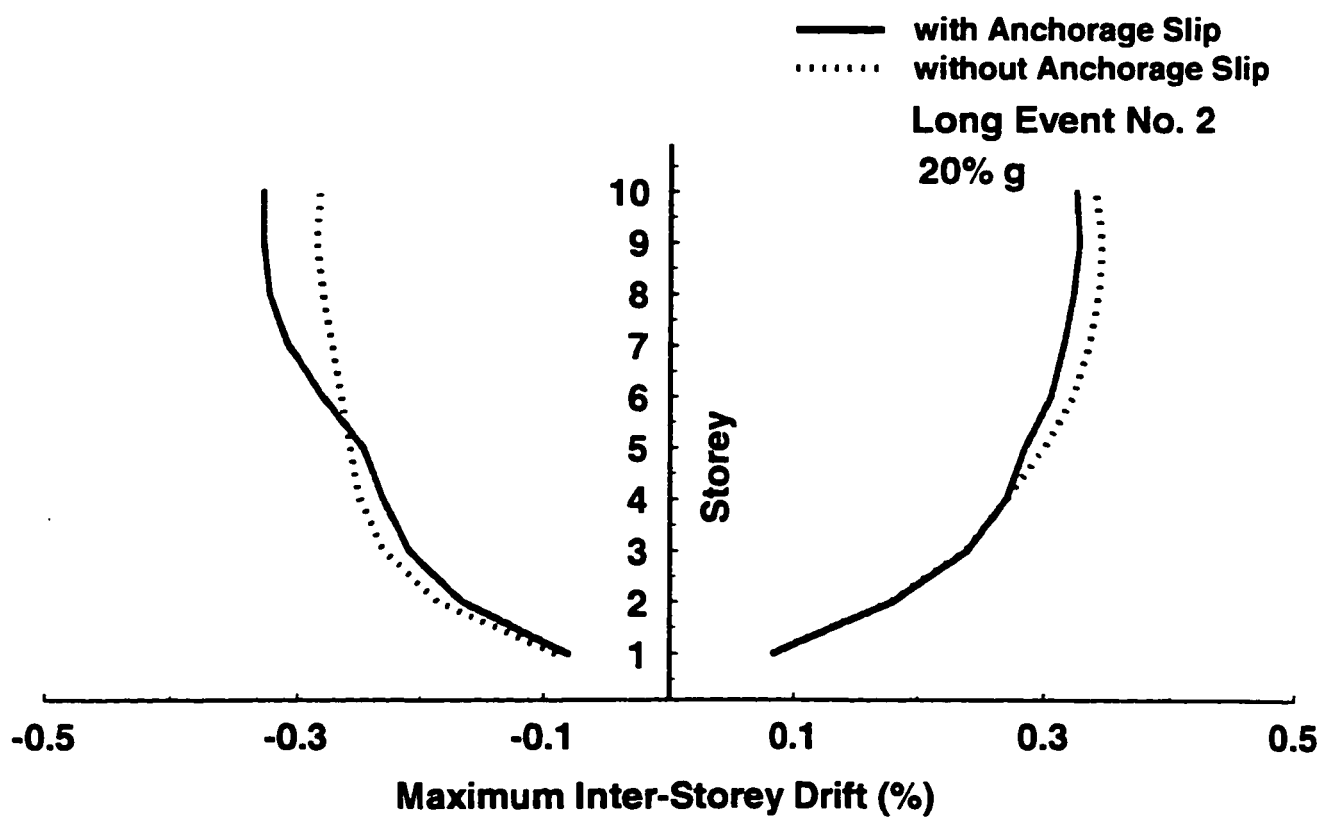


(b)

Fig. 4.48 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay for Eastern Canada

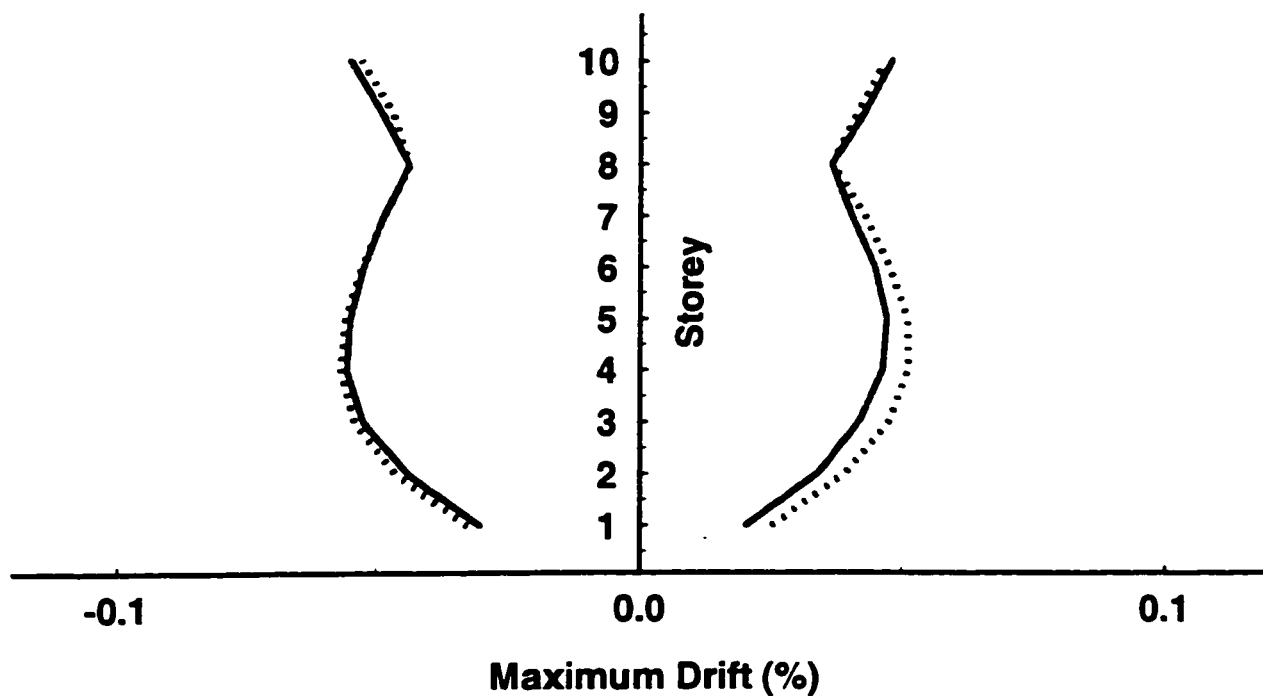


(a)

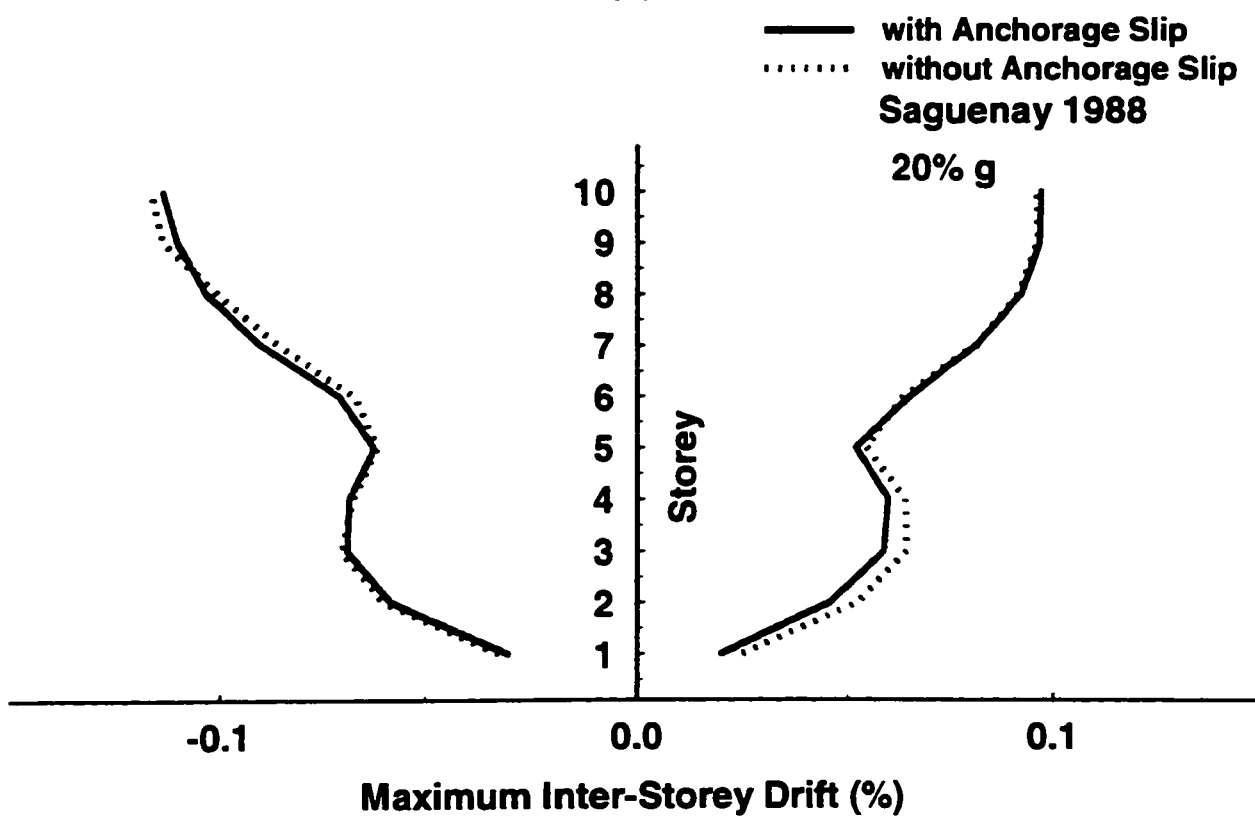


(b)

Fig. 4.49 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada



(a)



(b)

Fig. 4.50 Effect of Anchorage Slip on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay for Eastern Canada

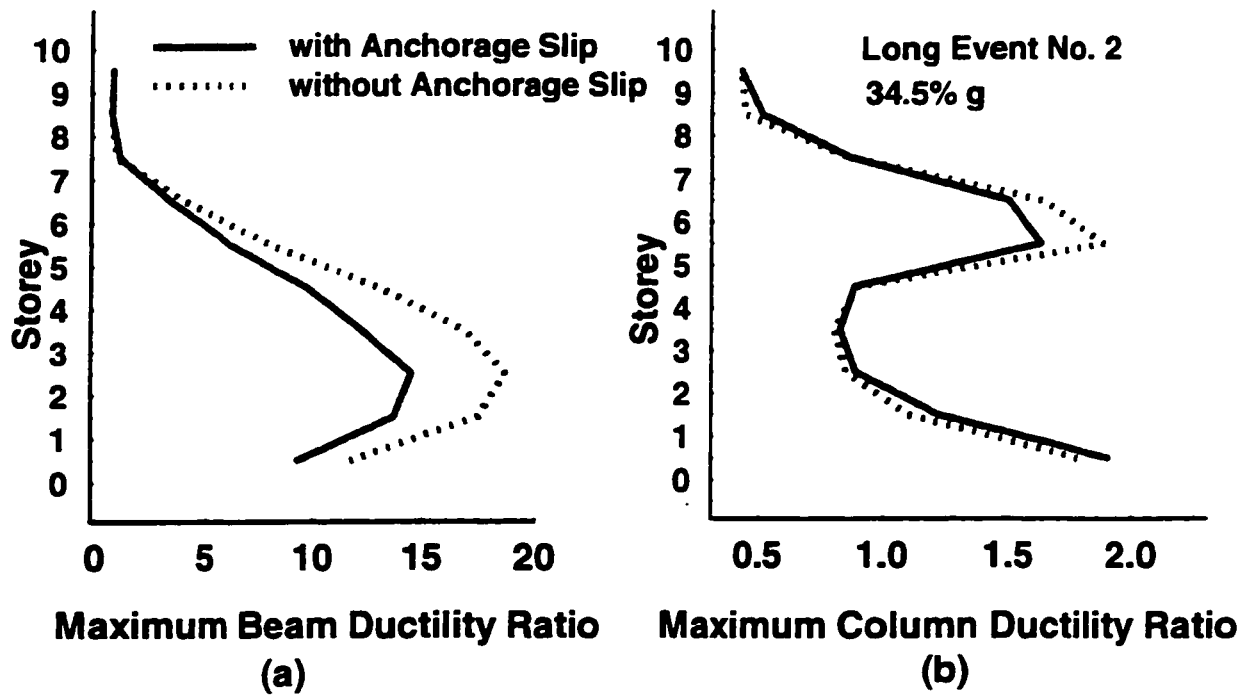
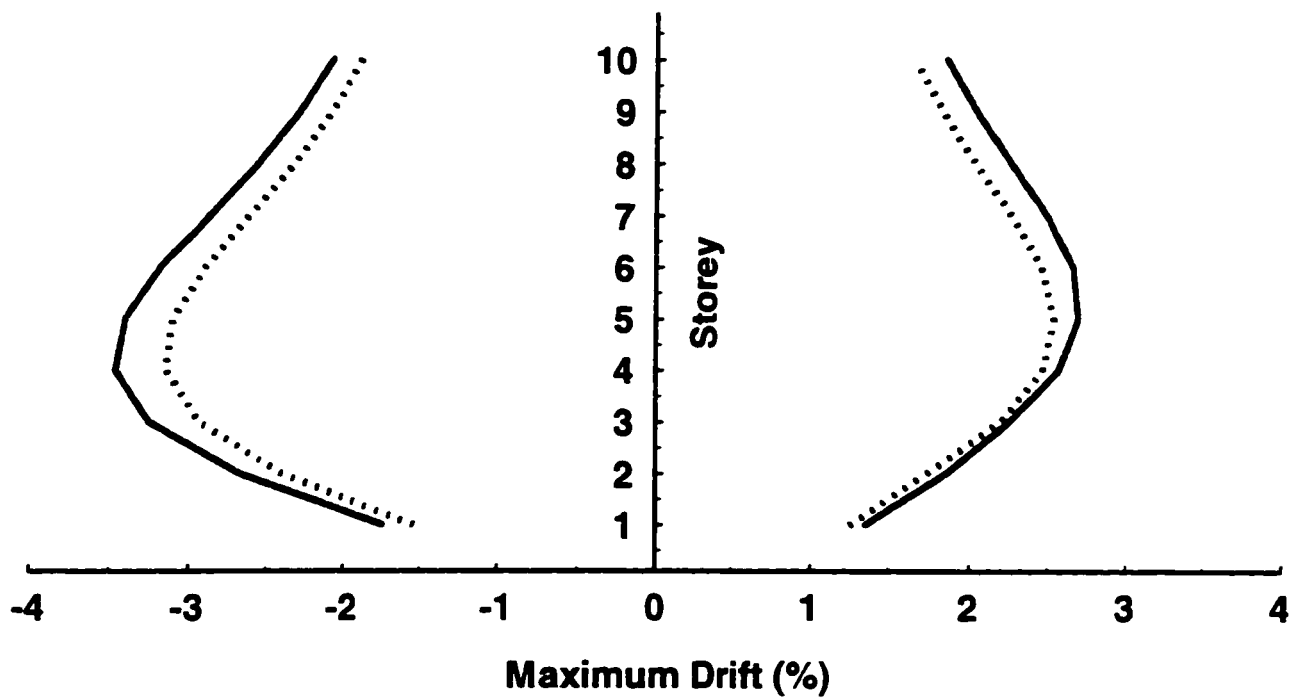


Fig. 4.51 Effect of Anchorage Slip on Ductility Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

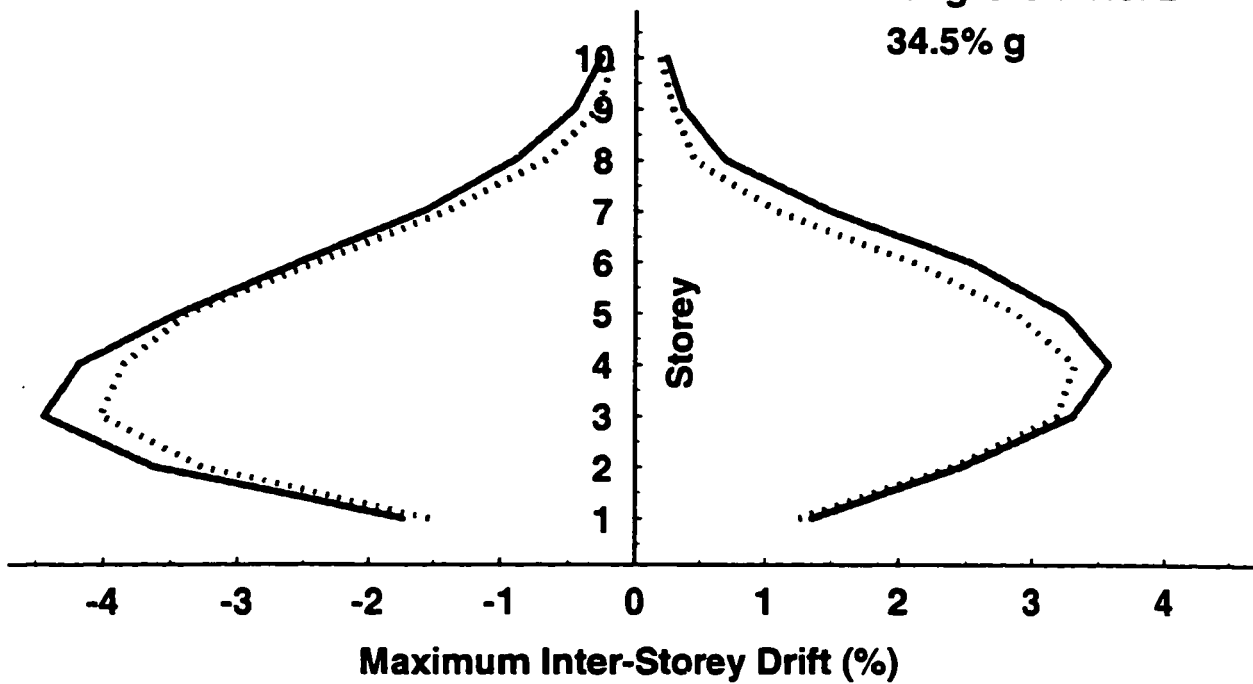


(a)

— with Anchorage Slip
 without Anchorage Slip

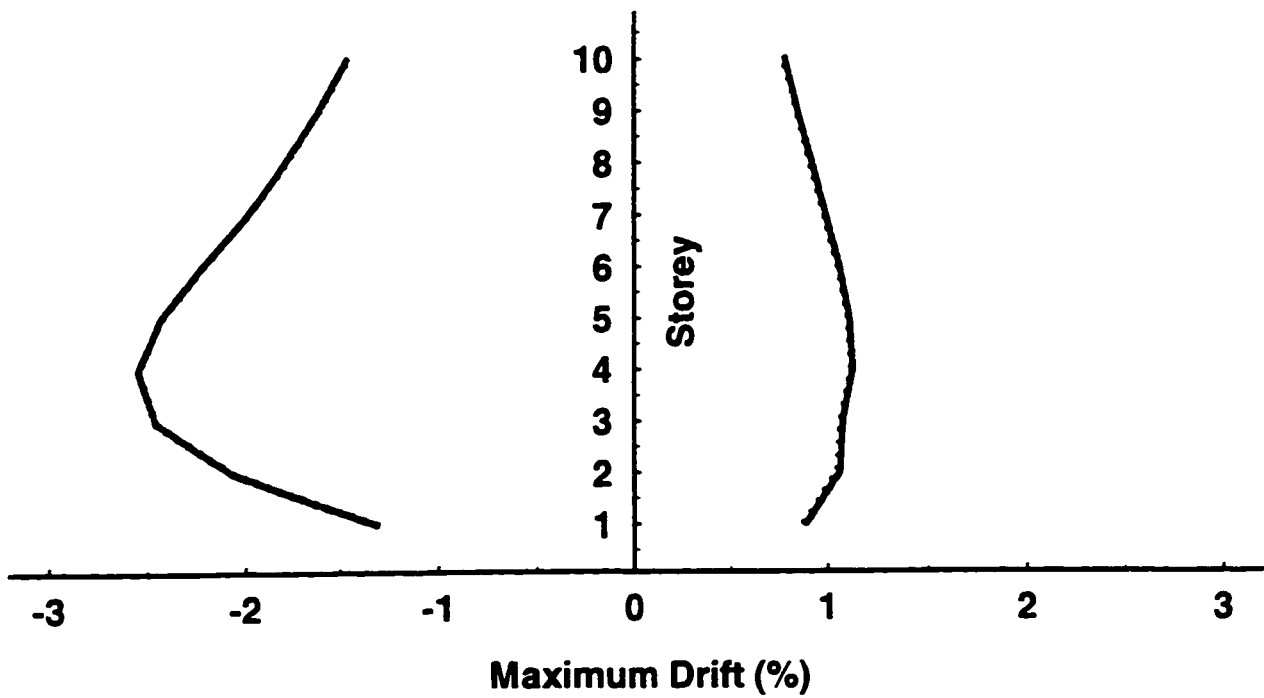
Long Event No. 2

34.5% g



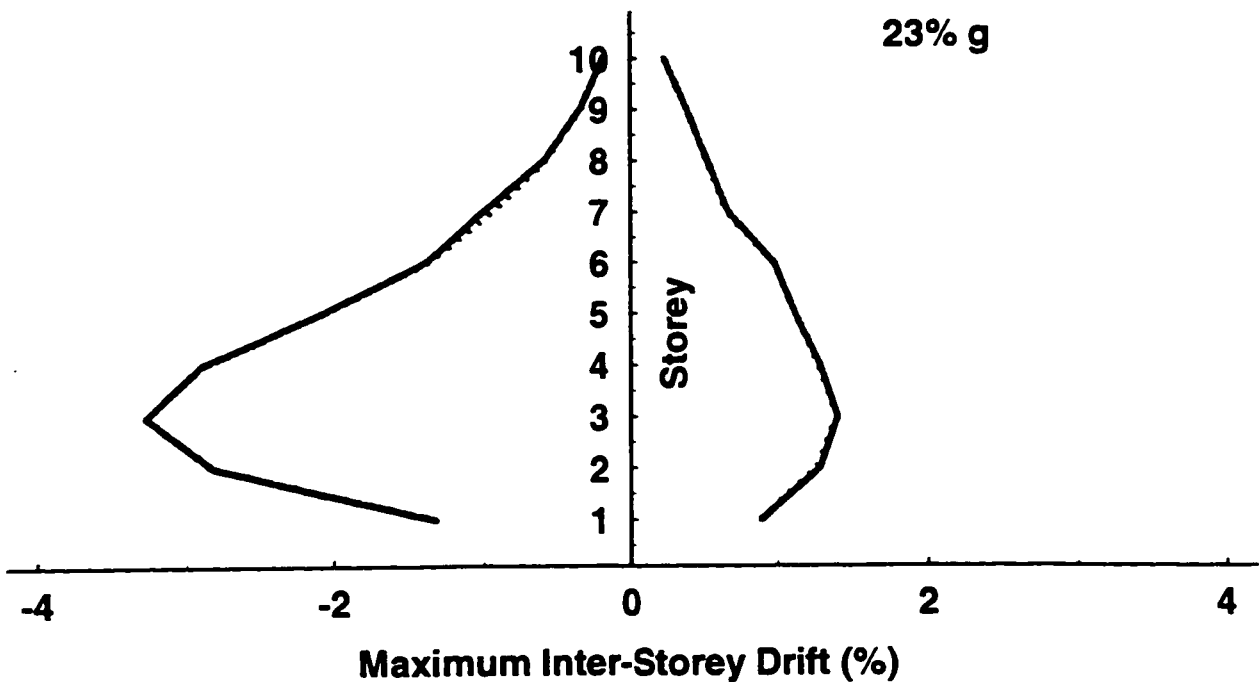
(b)

Fig. 4.52 Effect of Anchorage Slip on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



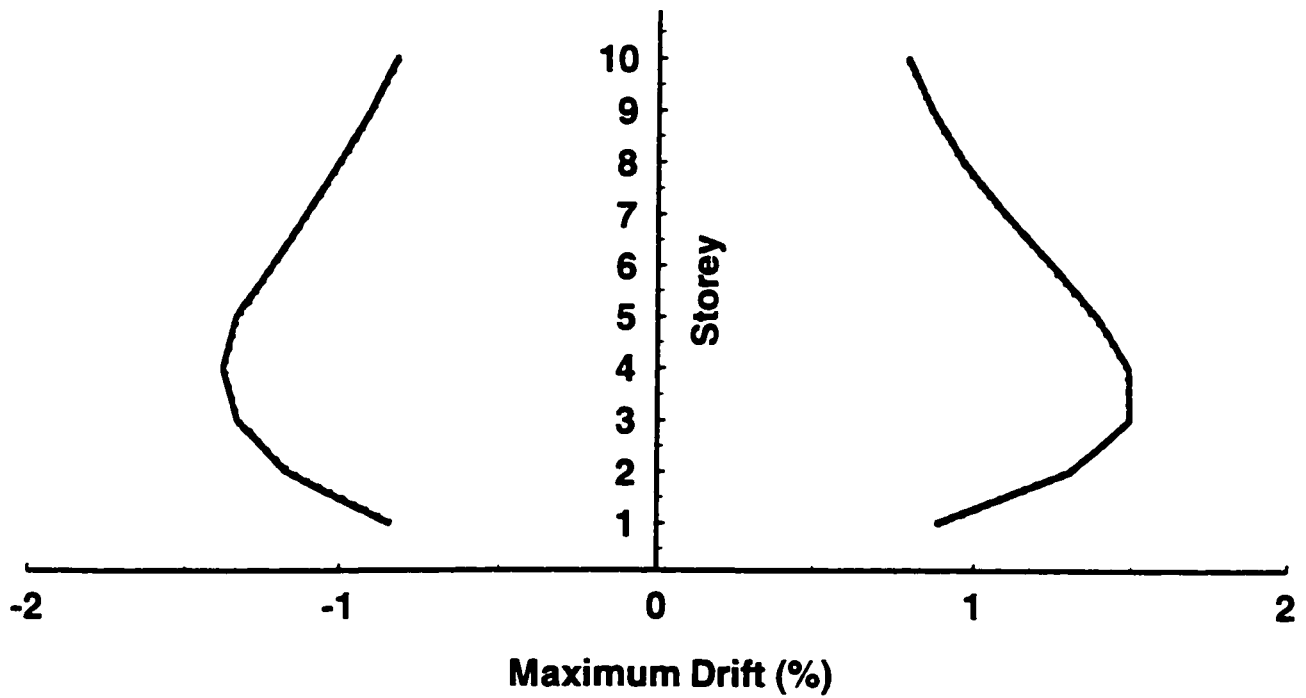
(a)

— with Inelastic Shear
 without Inelastic Shear
 Long Event No. 2
 23% g

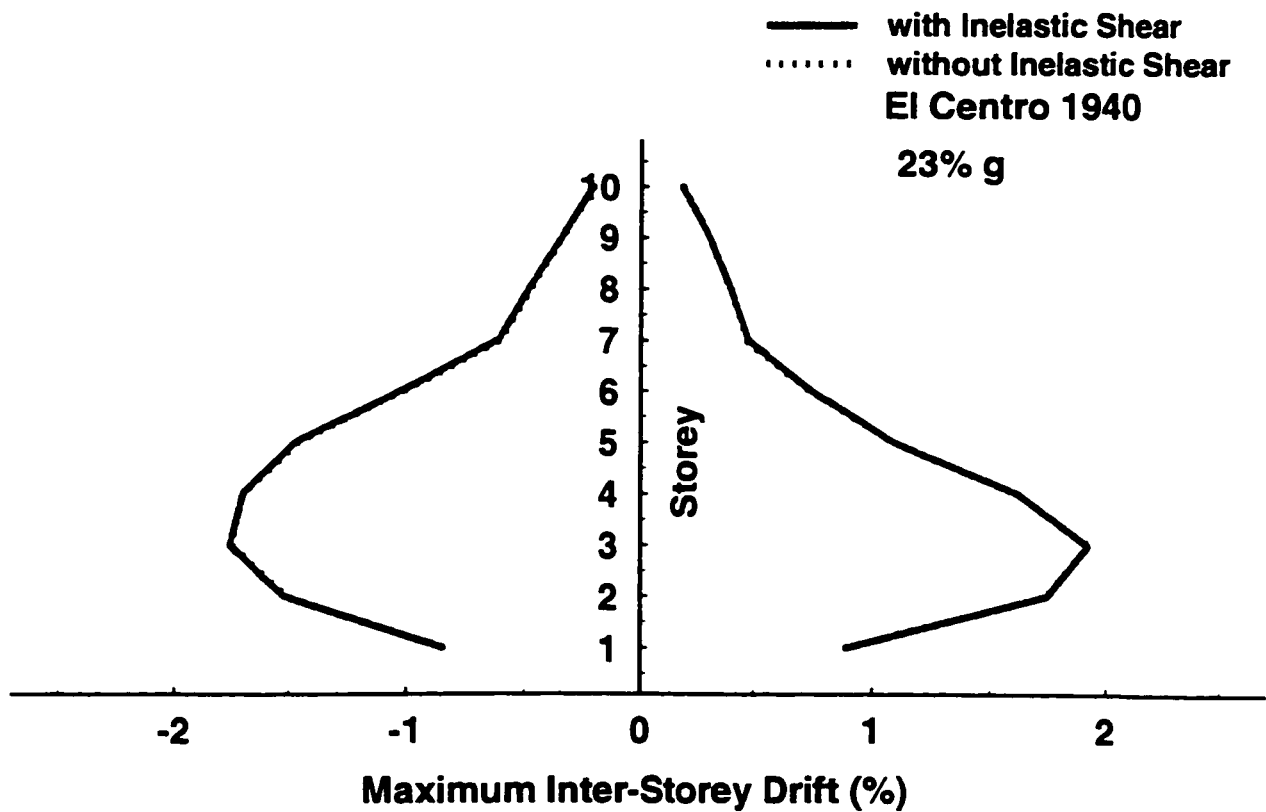


(b)

**Fig. 4.53 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF
 Subjected to Long Event No. 2 for Western Canada**

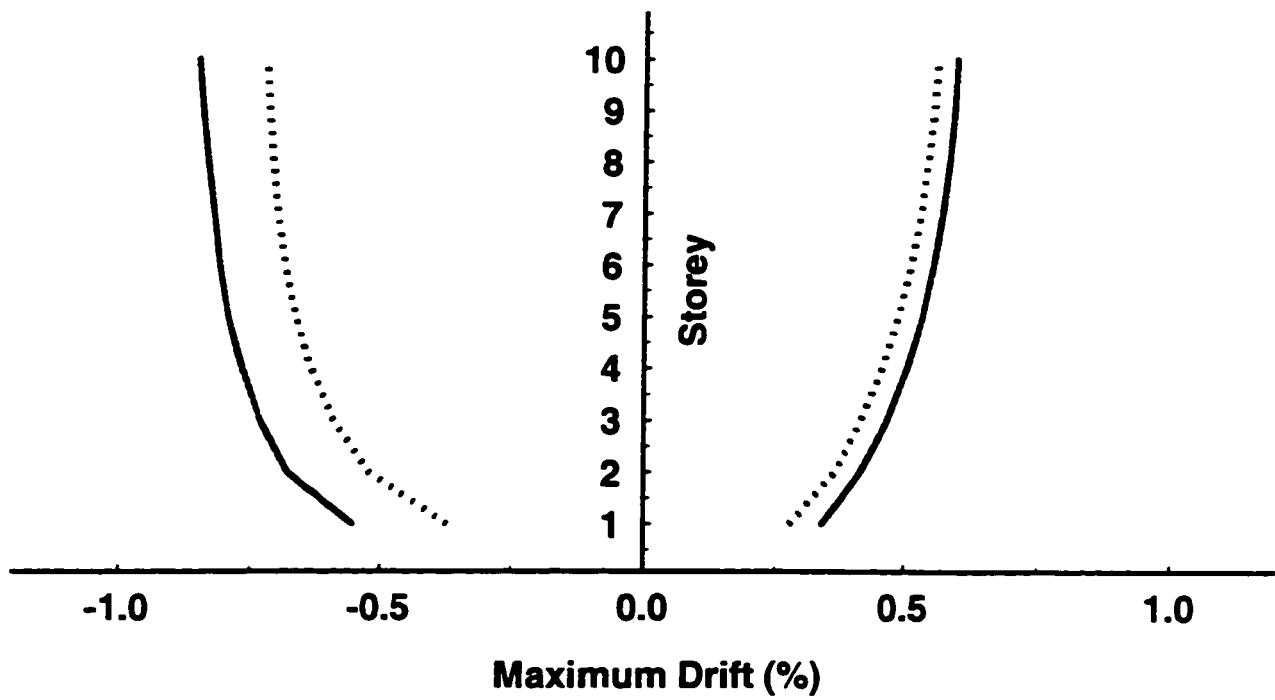


(a)



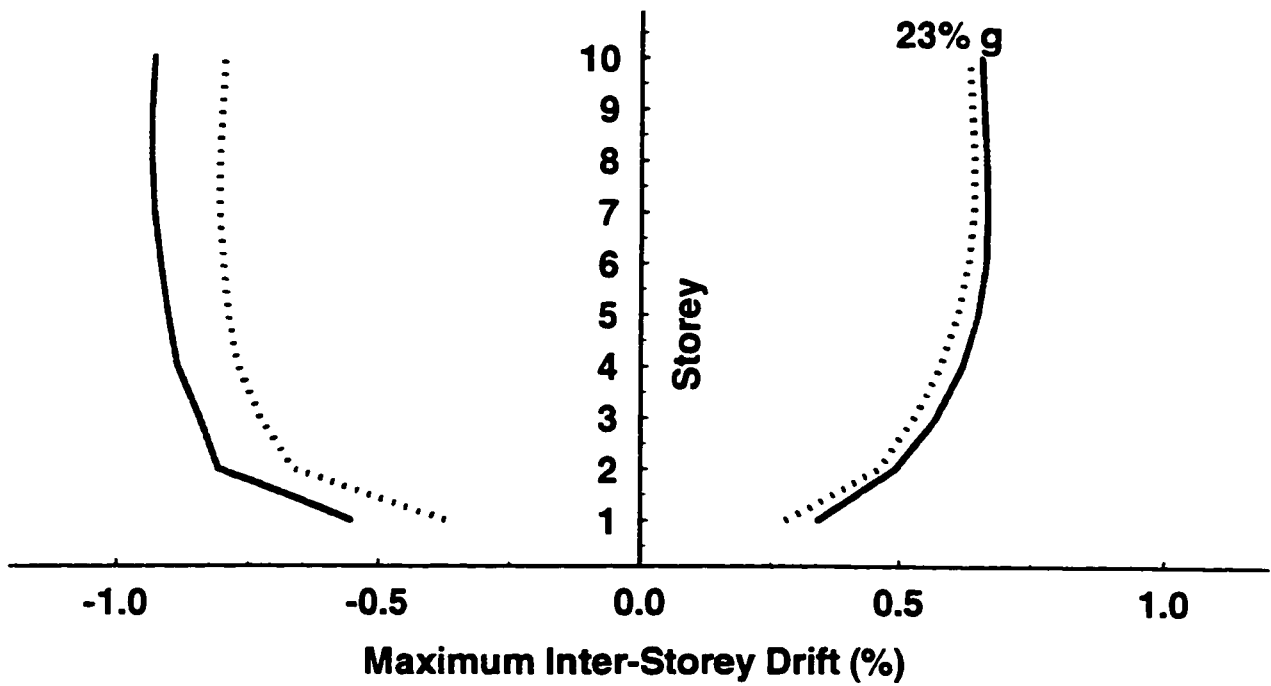
(b)

**Fig. 4.54 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF
Subjected to 1940 El Centro for Western Canada**



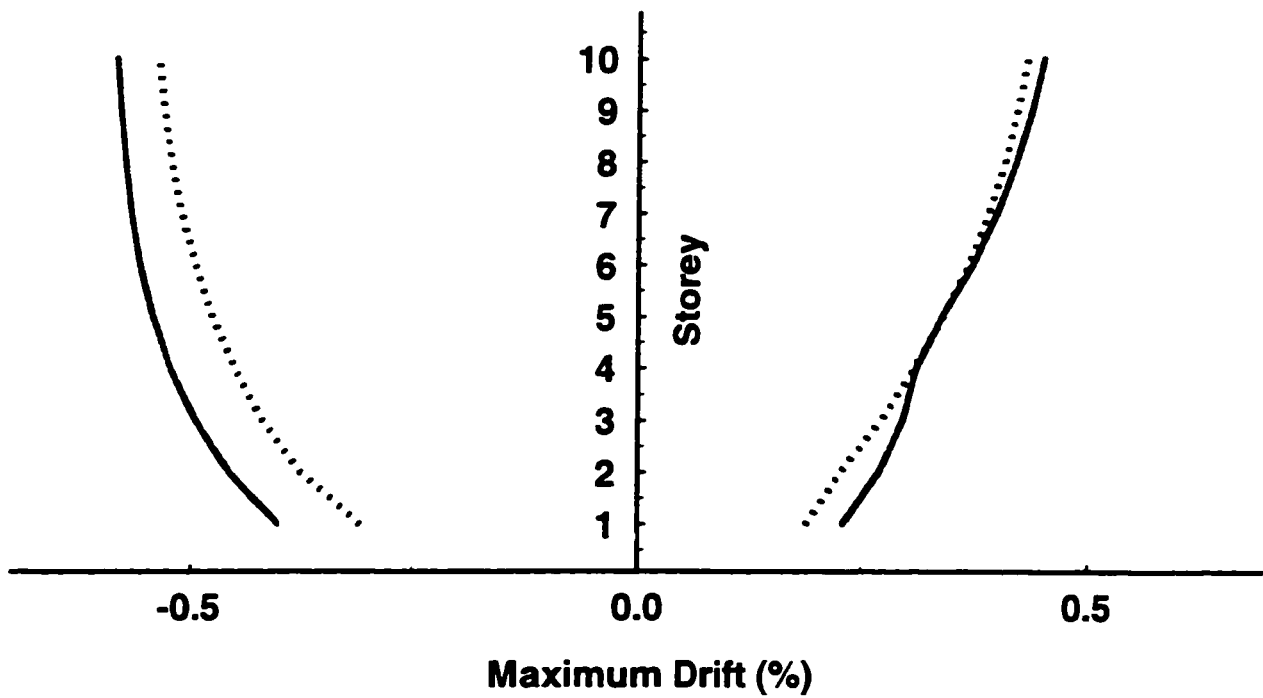
(a)

— with Inelastic Shear
 without Inelastic Shear
 Long Event No. 2

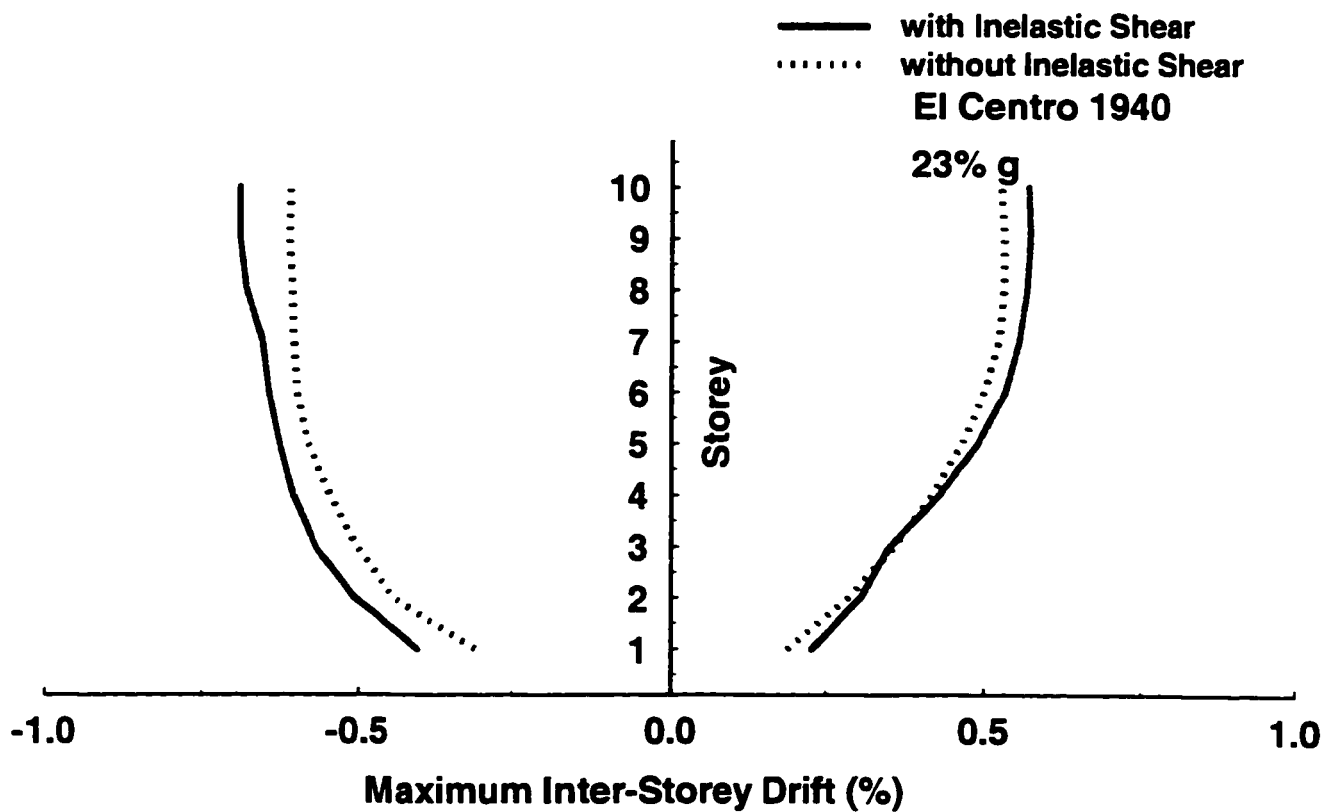


(b)

**Fig. 4.55 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW
 Subjected to Long Event No. 2 for Western Canada**

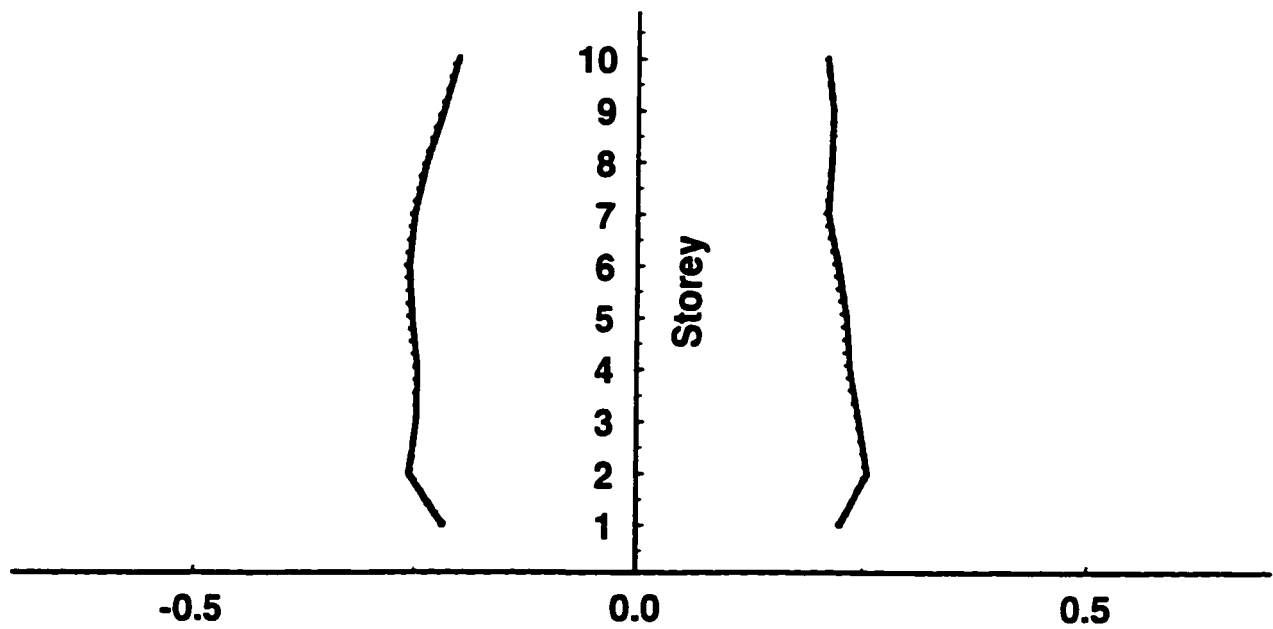


(a)



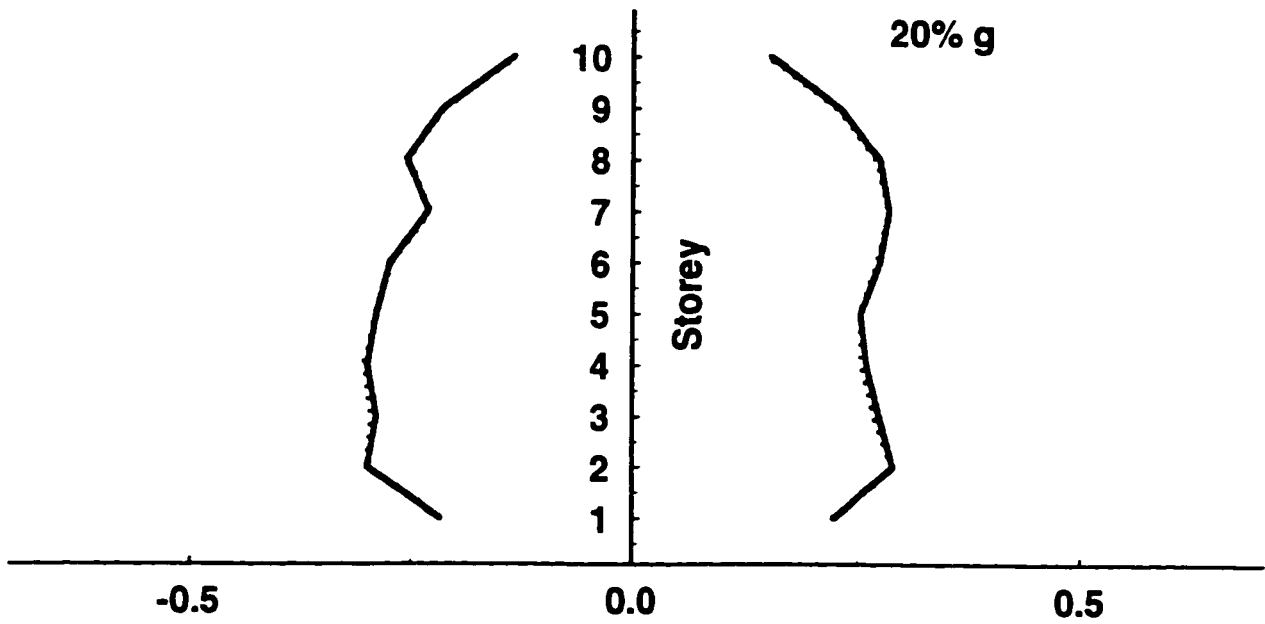
(b)

**Fig. 4.56 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW
Subjected to 1940 El Centro for Western Canada**



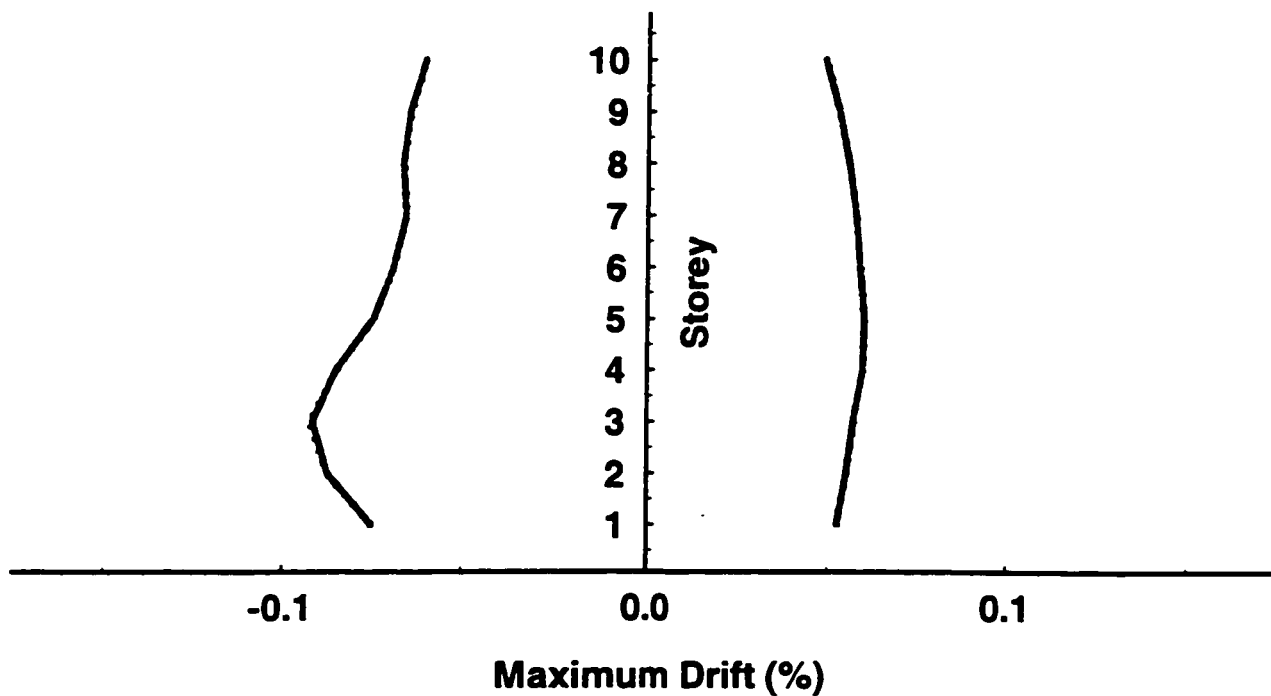
(a)

— with Inelastic Shear
 without Inelastic Shear
 Long Event No. 2
 20% g

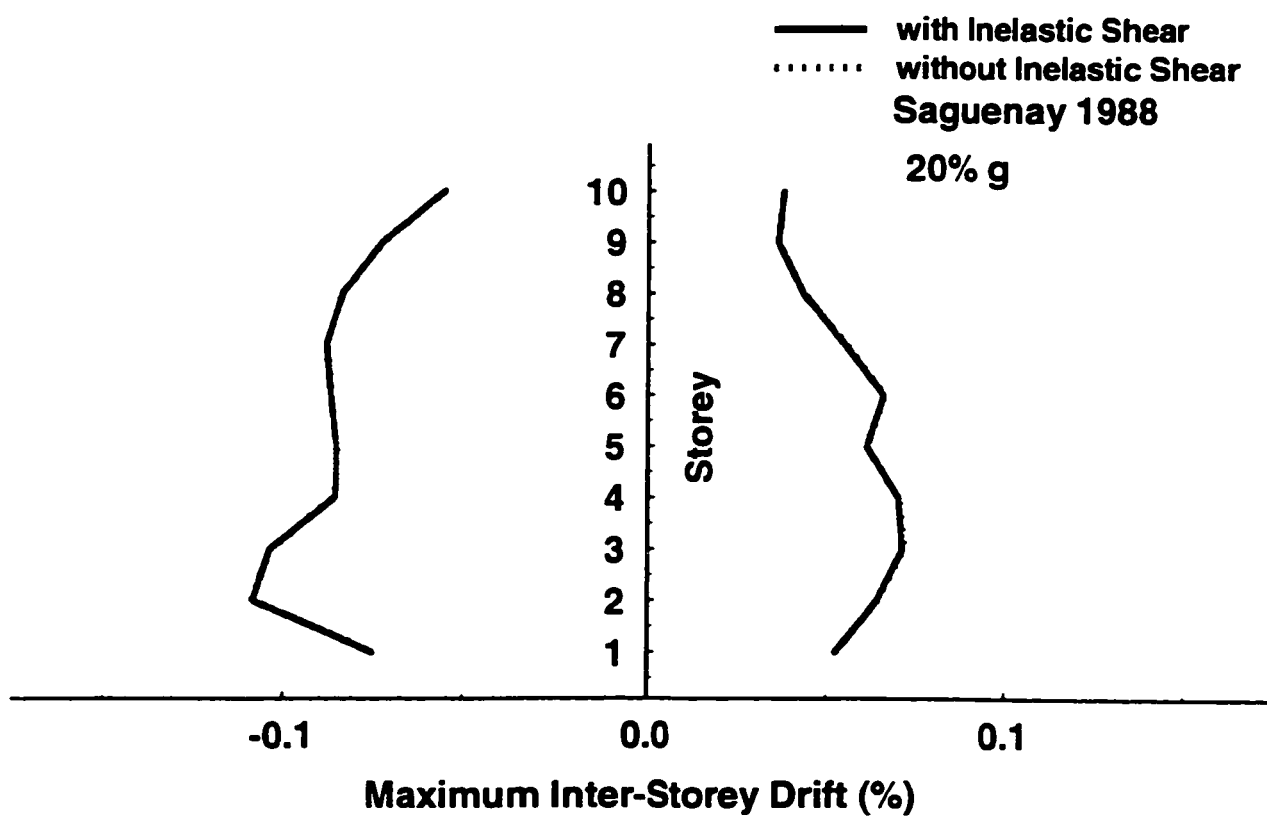


(b)

**Fig. 4.57 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF
 Subjected to Long Event No. 2 for Eastern Canada**

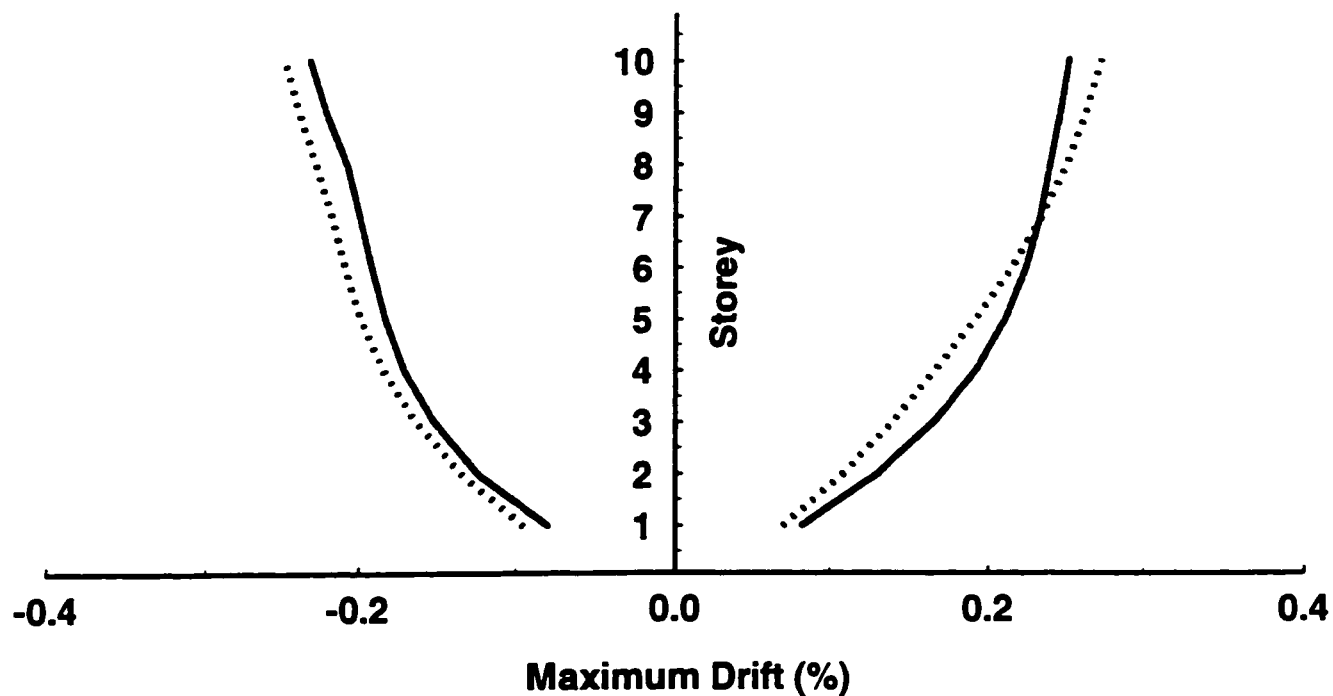


(a)

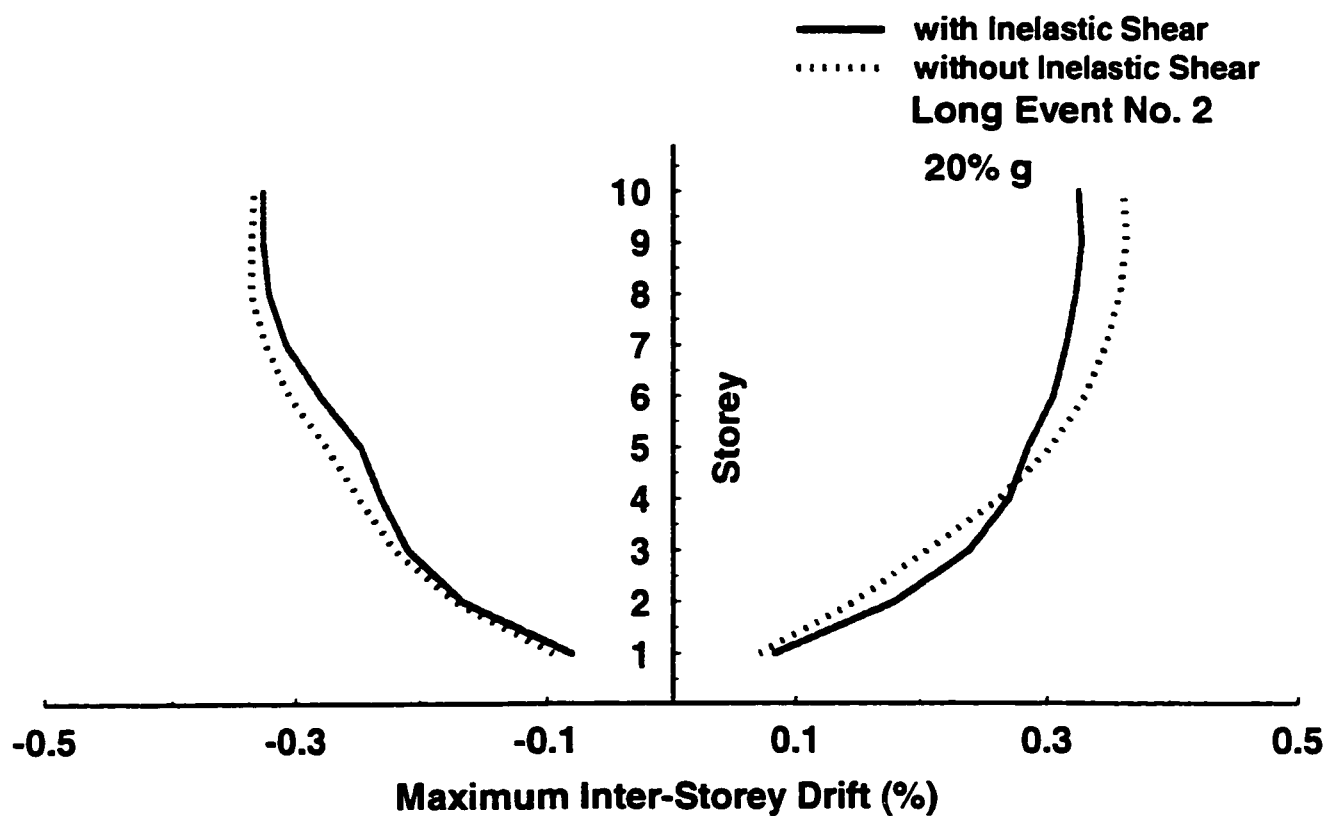


(b)

**Fig. 4.58 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey MRF
Subjected to 1988 Saguenay for Eastern Canada**

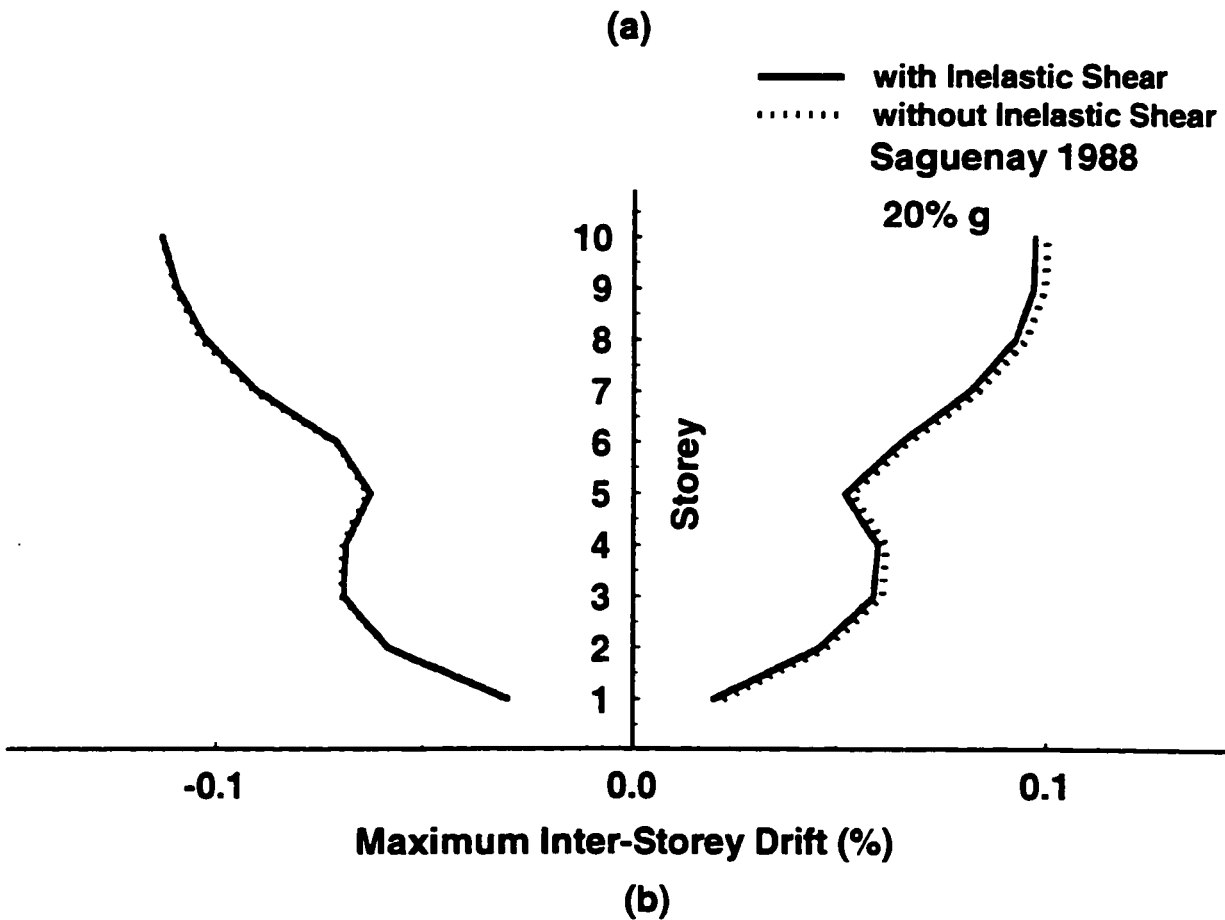
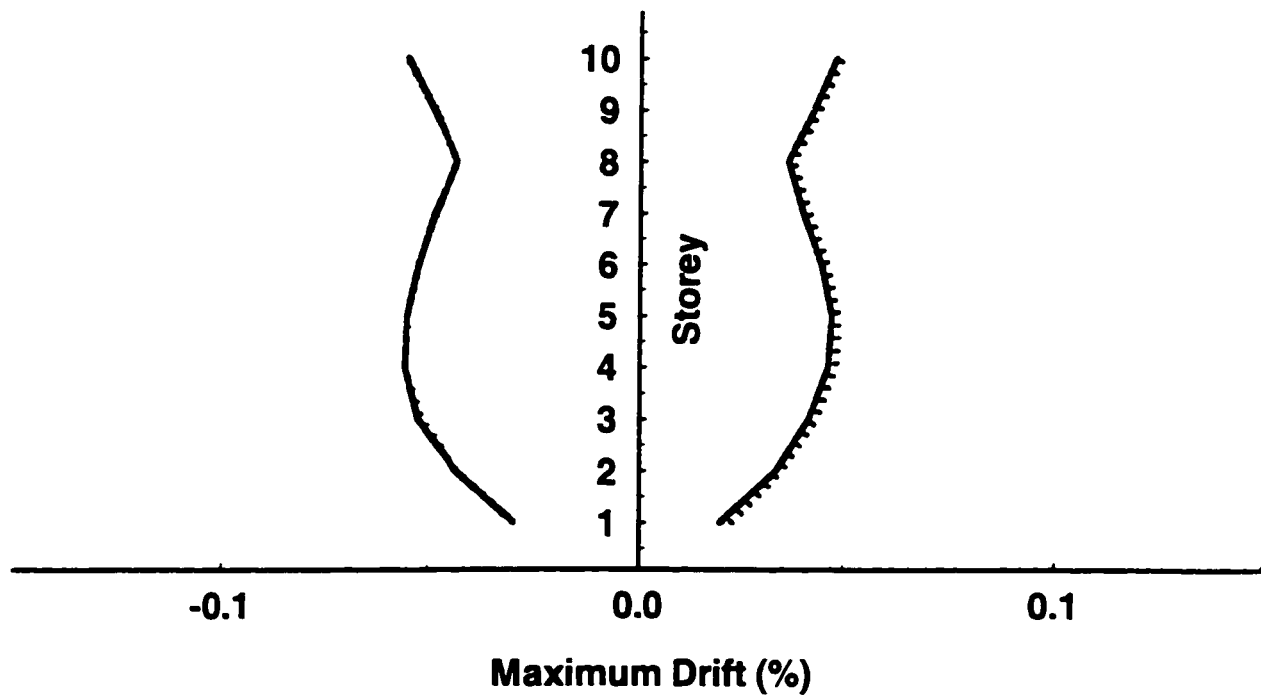


(a)

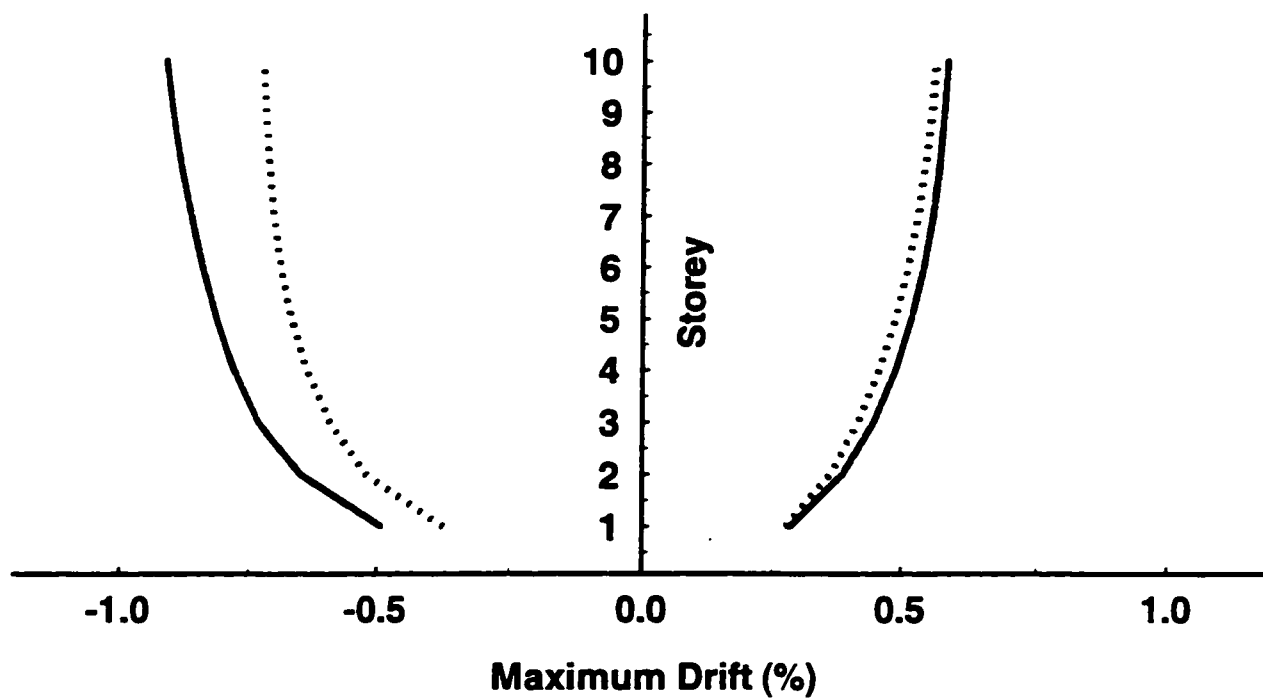


(b)

**Fig. 4.59 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW
Subjected to Long Event No. 2 for Eastern Canada**



**Fig. 4.60 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW
Subjected to 1988 Saguenay for Eastern Canada**



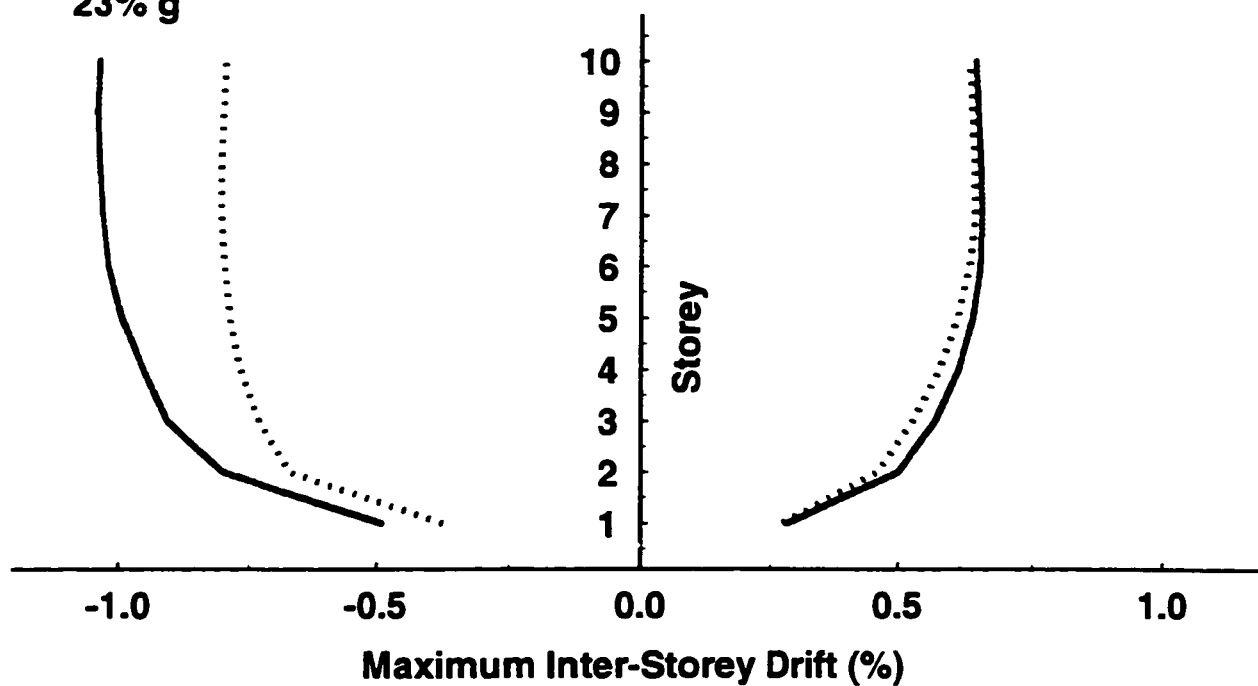
(a)

$(M_y)_{\text{shear}} = 0.75 (M_y)_{\text{flexur}}$ for shear wall

Long Event No. 2

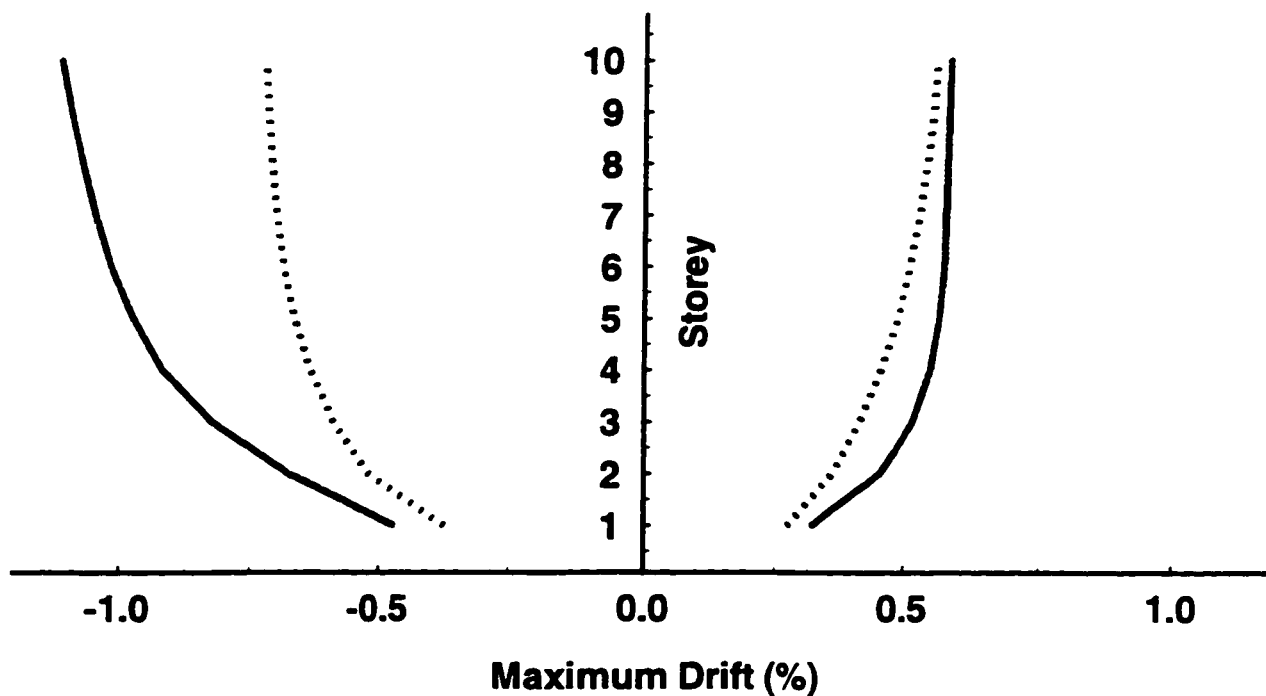
23% g

— with Inelastic Shear
 without Inelastic Shear



(b)

**Fig. 4.61 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW
 Subjected to Long Event No. 2 for Western Canada**



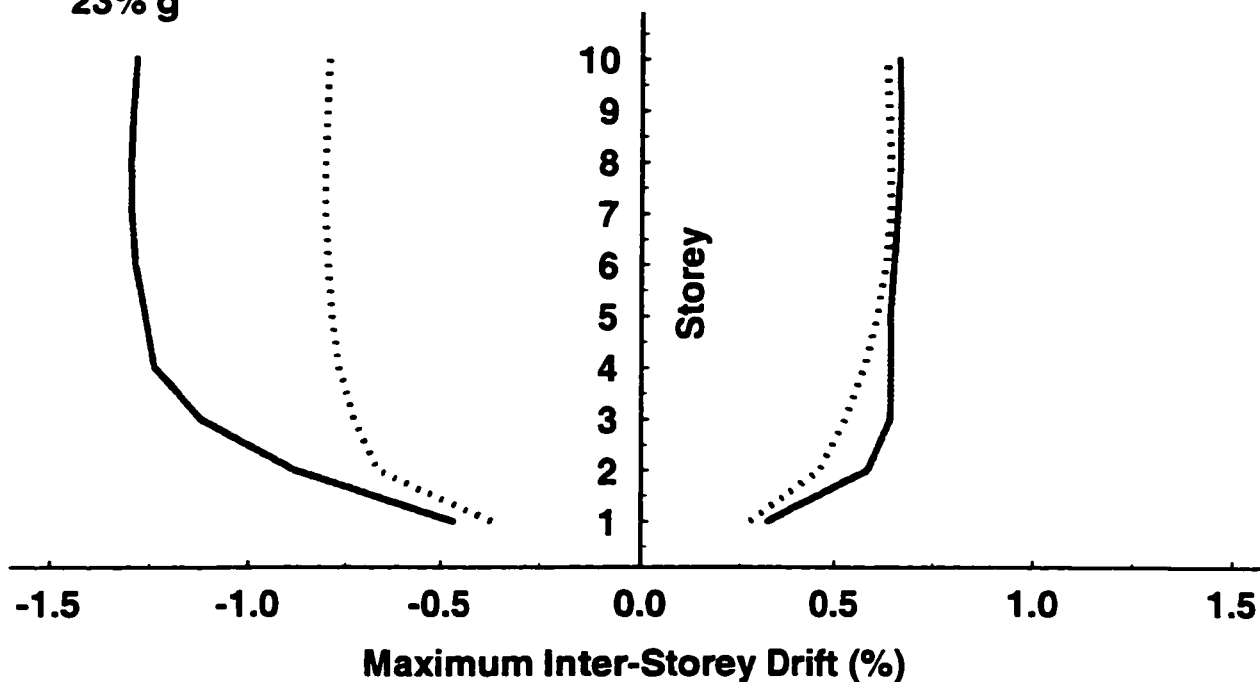
(a)

$(M_y)_{\text{shear}} = 0.50 (M_y)_{\text{flexur}}$ for shear wall

Long Event No. 2

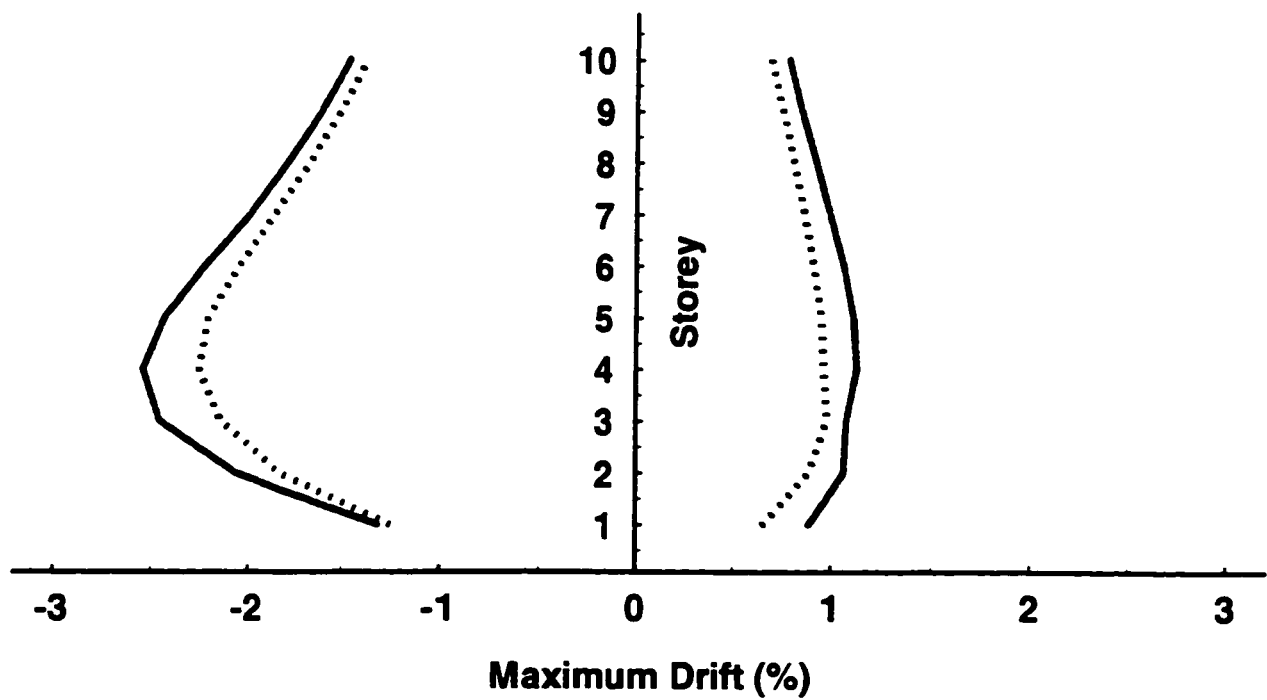
23% g

— with Inelastic Shear
 without Inelastic Shear

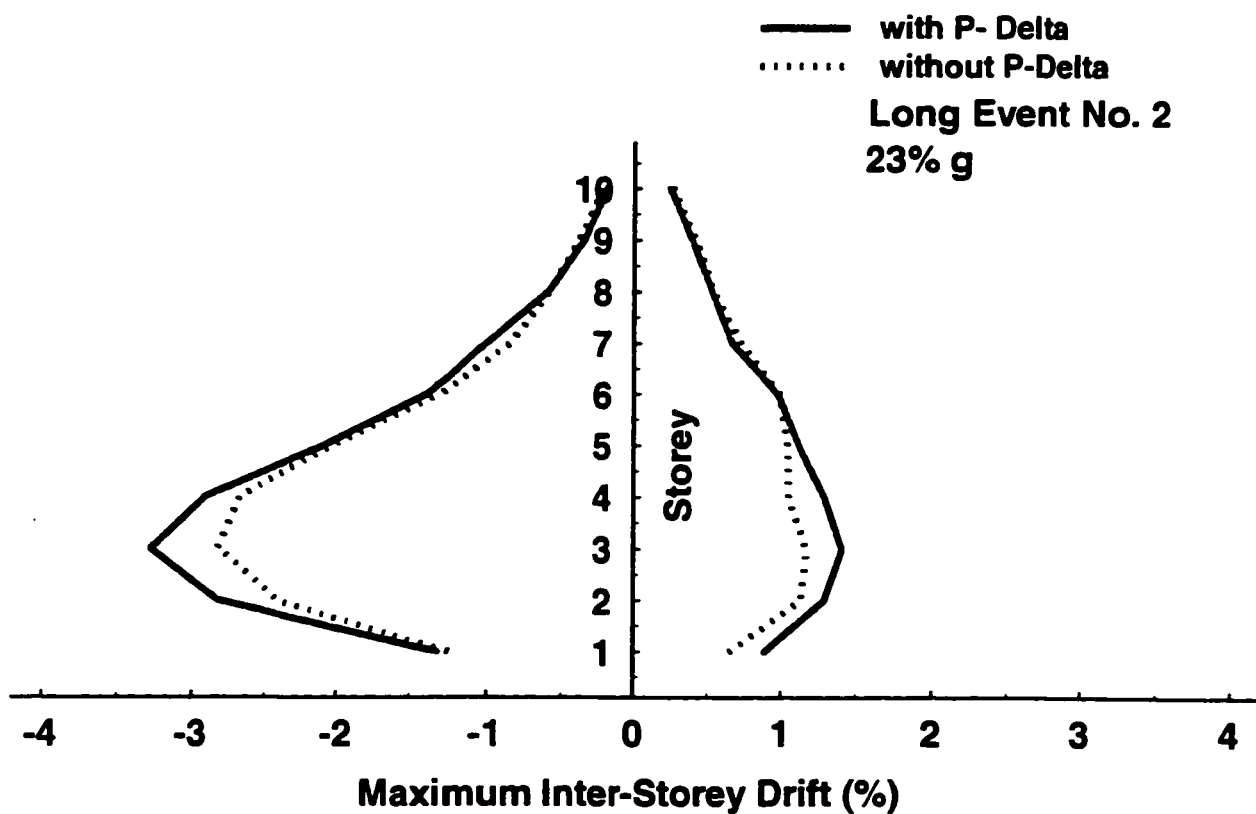


(b)

**Fig. 4.62 Effect of Inelastic Shear Deformation on Drift Response. 10-Storey SHW
 Subjected to Long Event No. 2 for Western Canada**

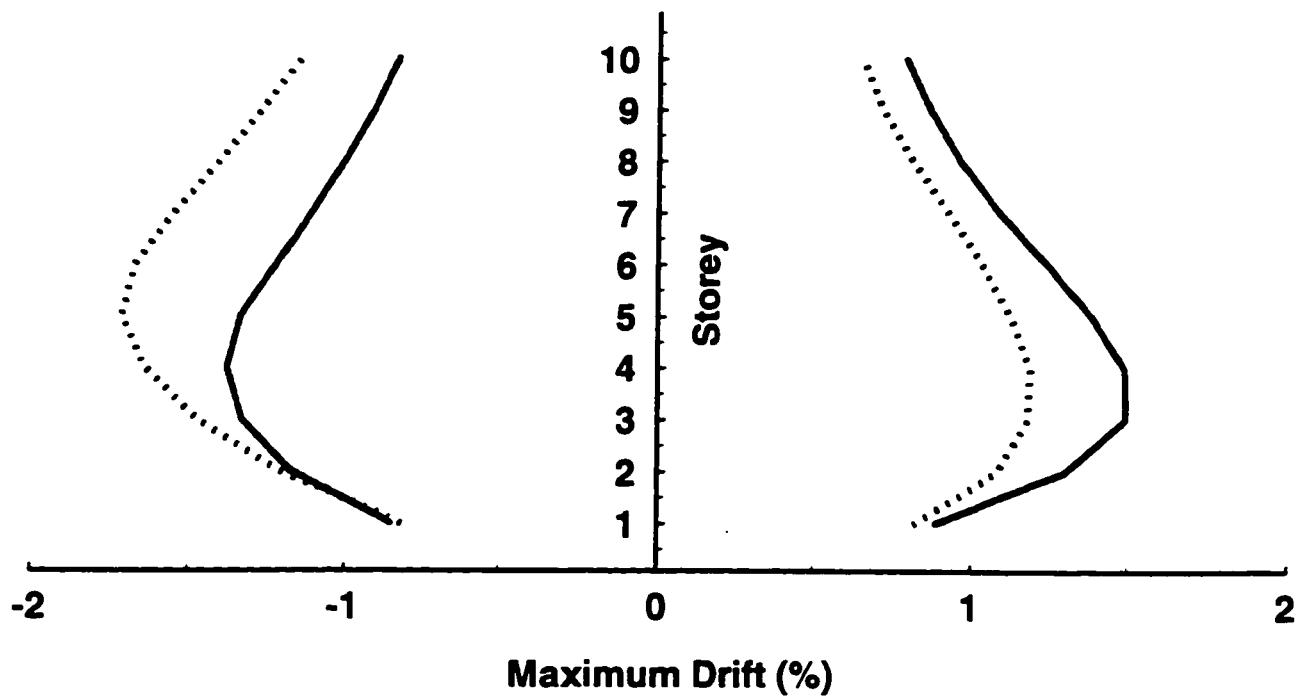


(a)

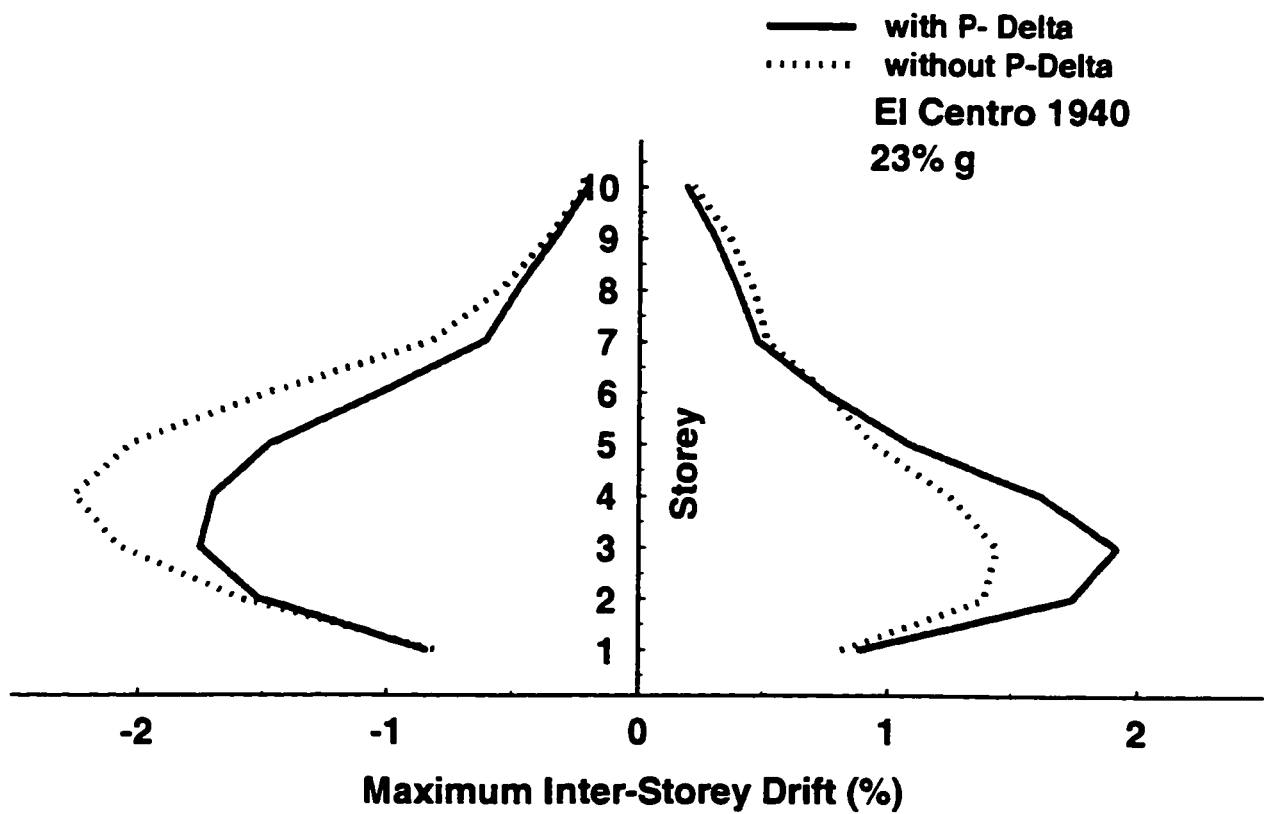


(b)

Fig. 4.63 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

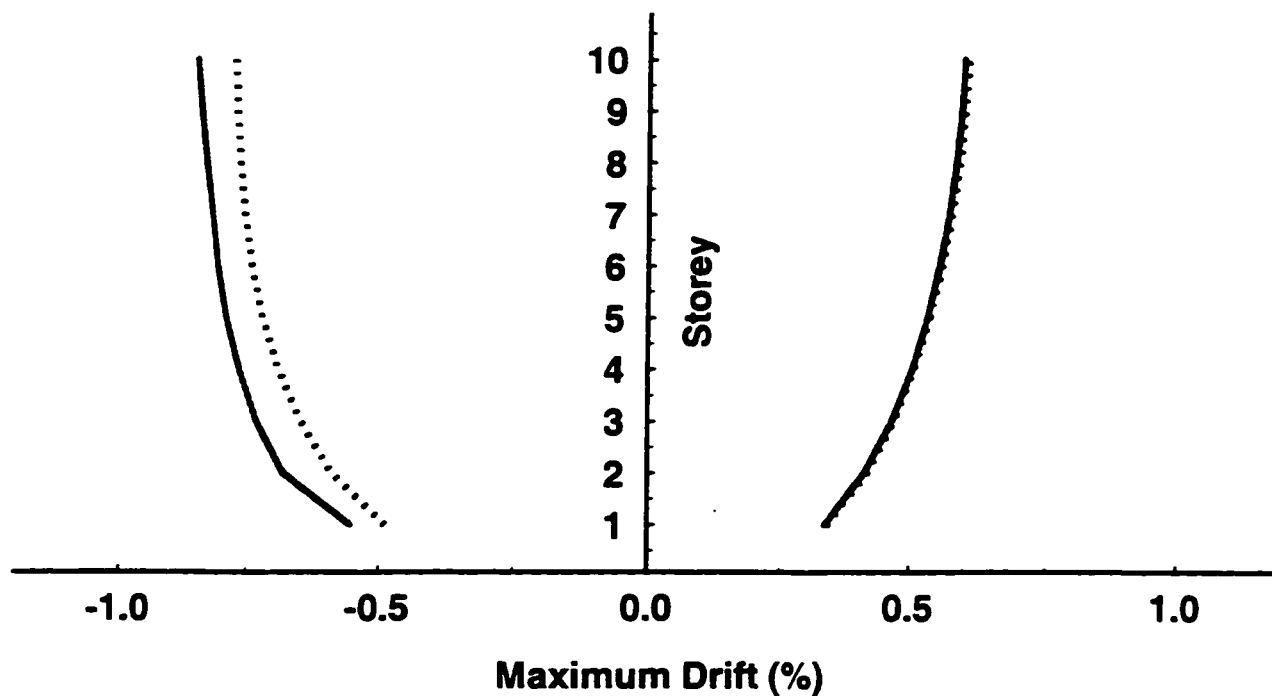


(a)

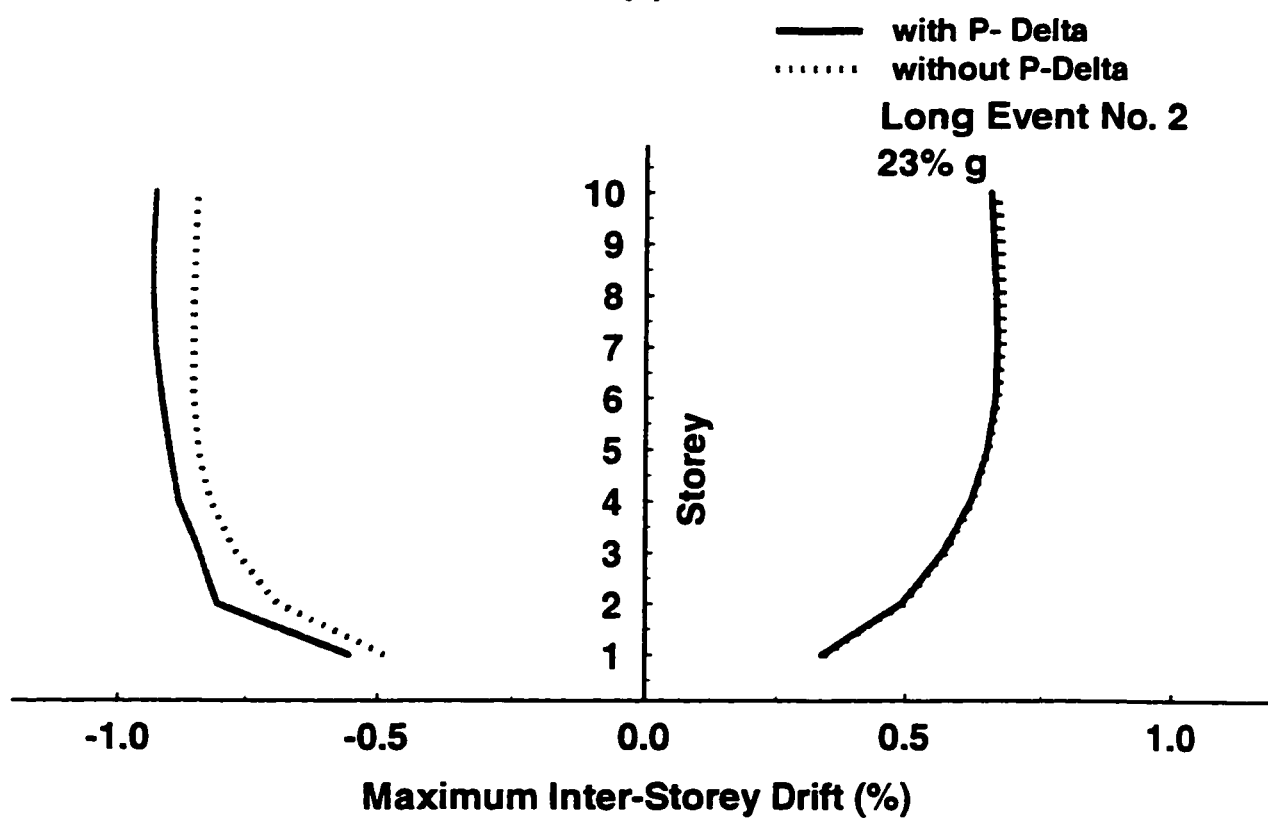


(b)

Fig. 4.64 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada

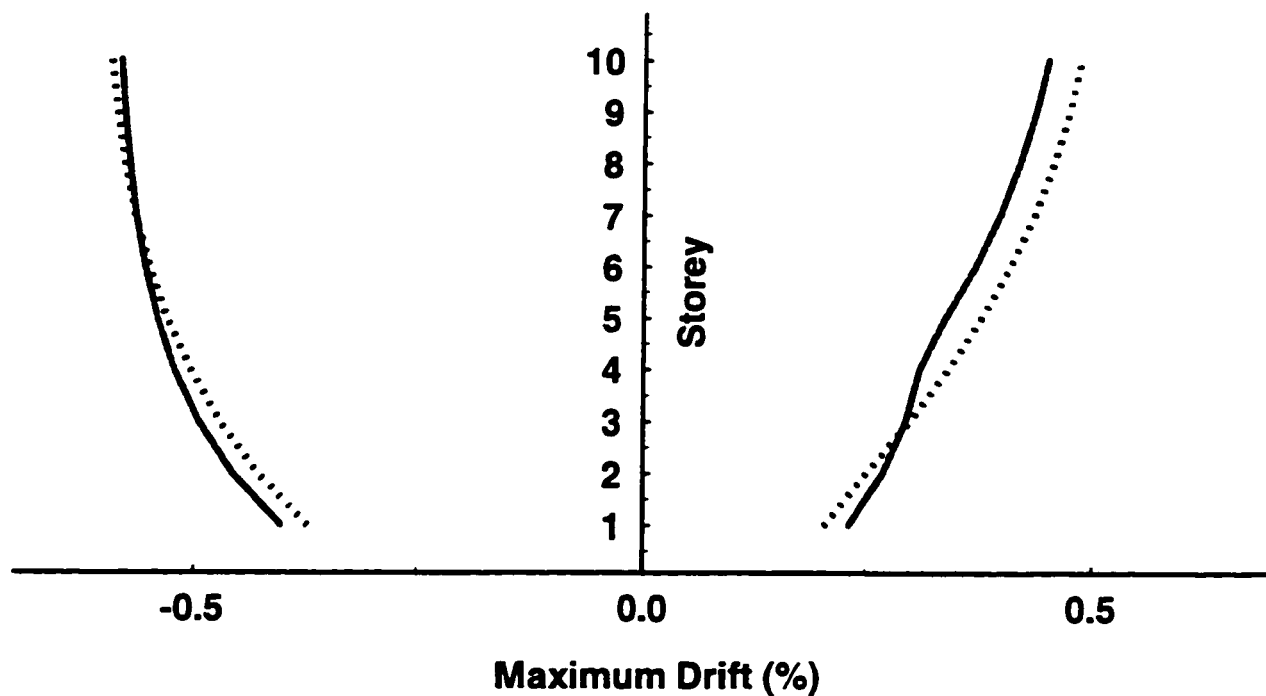


(a)

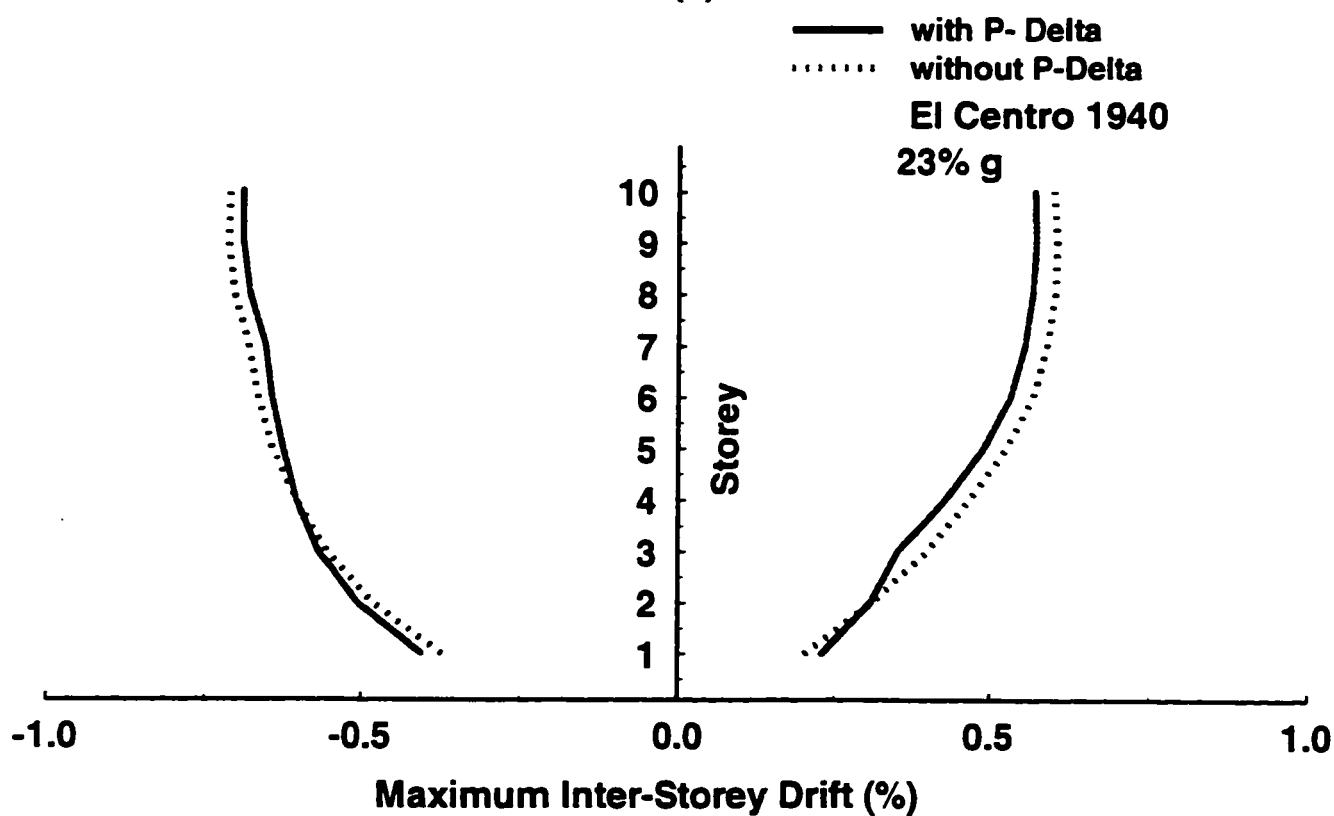


(b)

Fig. 4.65 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Western Canada

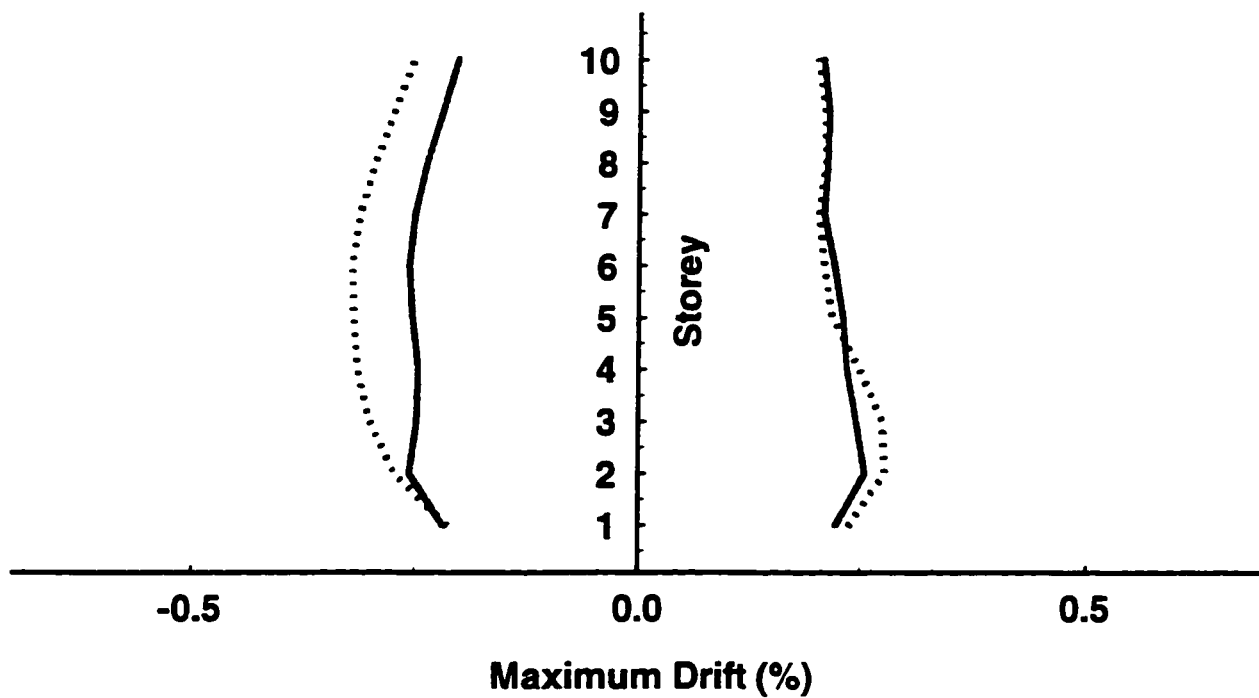


(a)

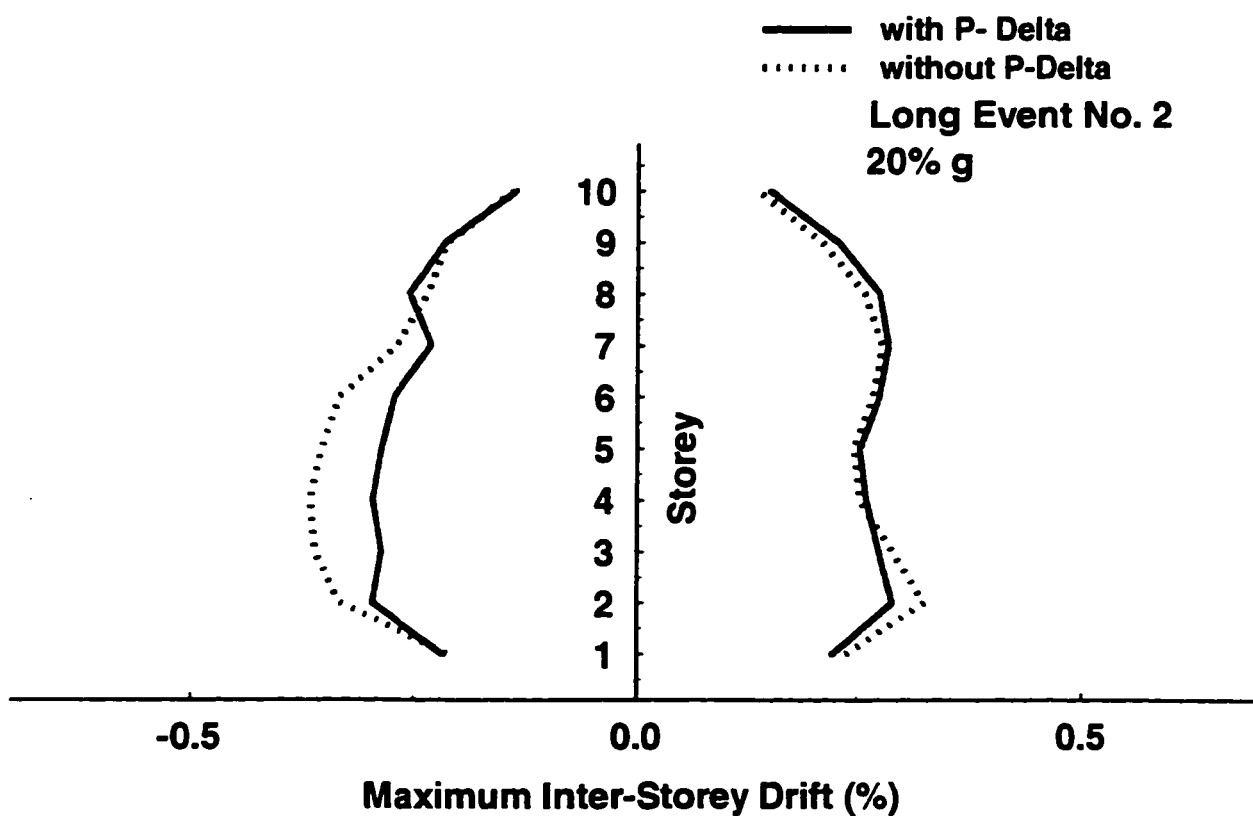


(b)

Fig. 4.66 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to 1940 El Centro for Western Canada

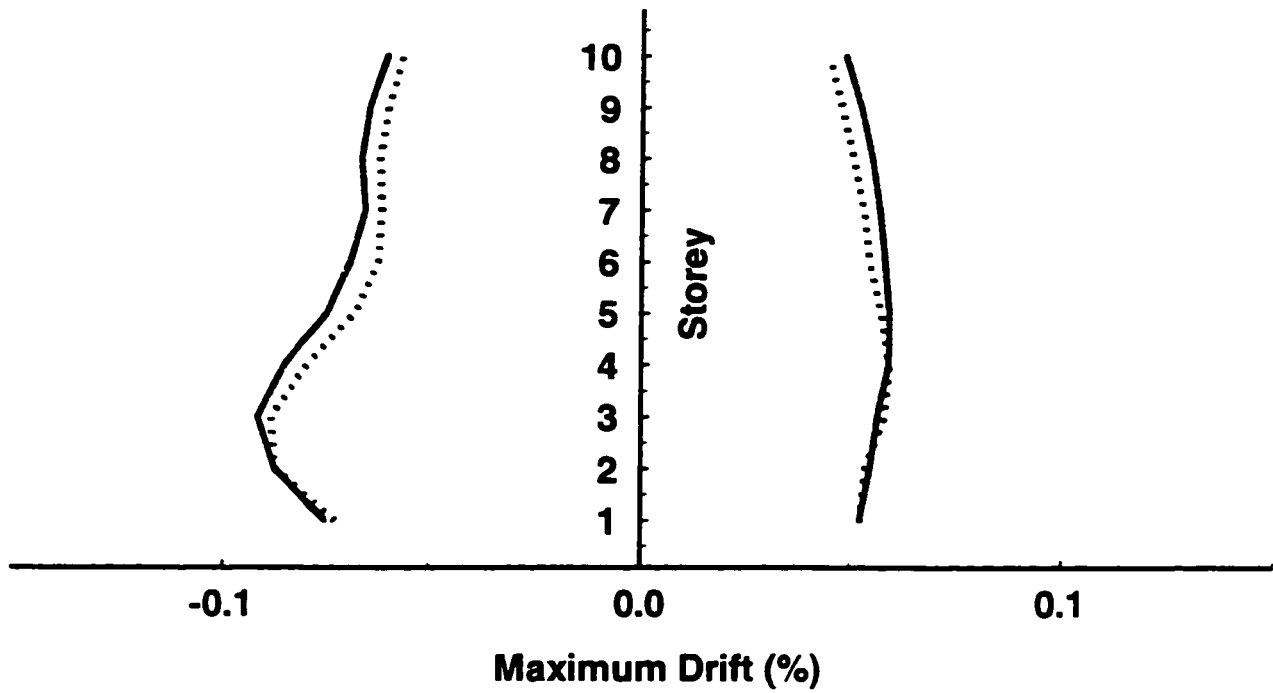


(a)

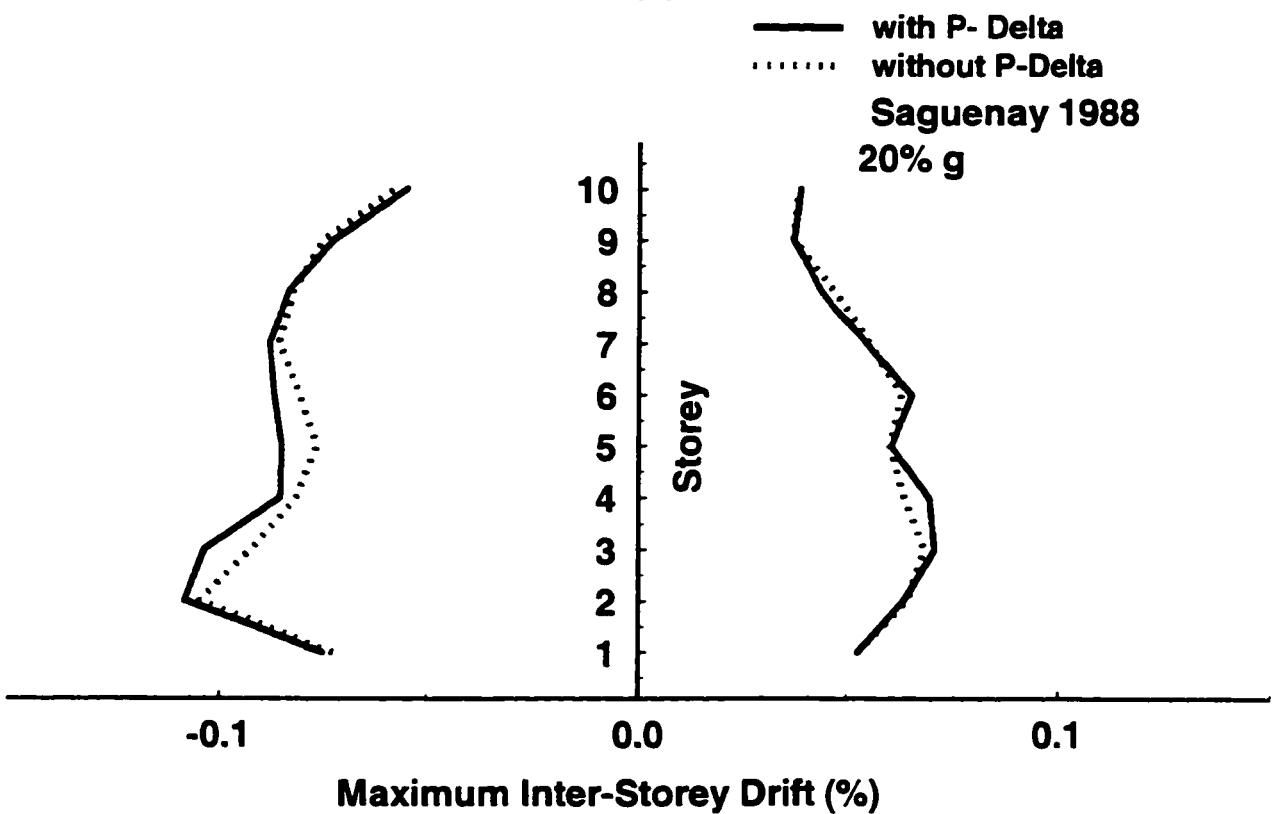


(b)

Fig. 4.67 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Eastern Canada

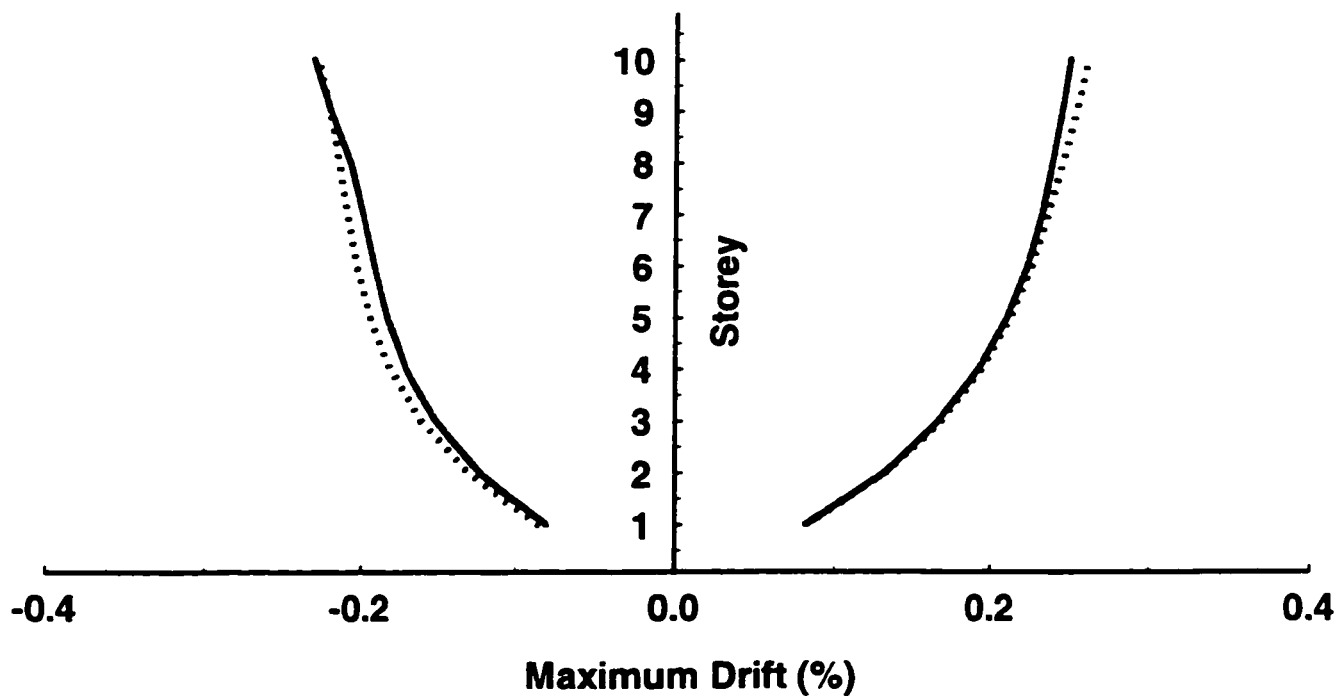


(a)

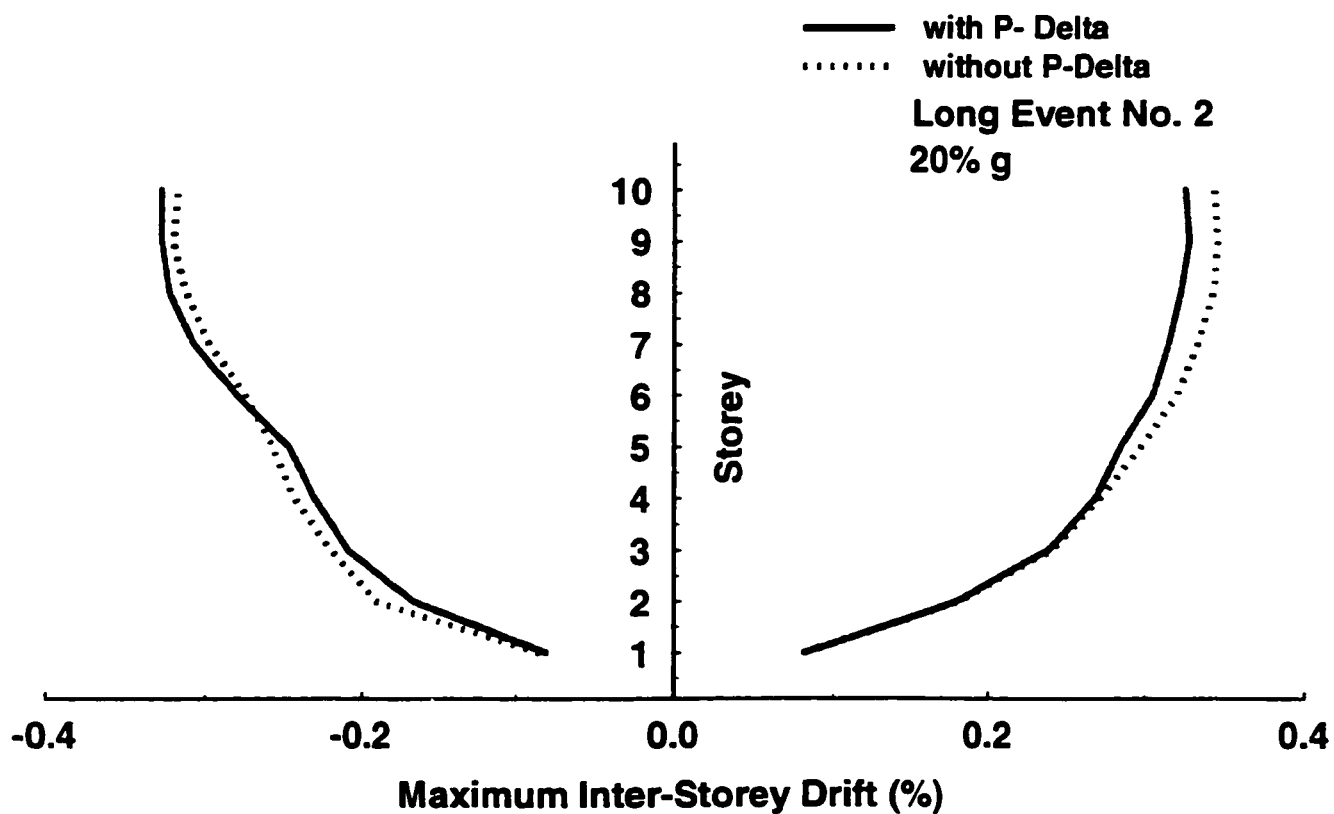


(b)

Fig. 4.68 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to 1988 Saguenay for Eastern Canada



(a)



(b)

Fig. 4.69 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to Long Event No. 2 for Eastern Canada

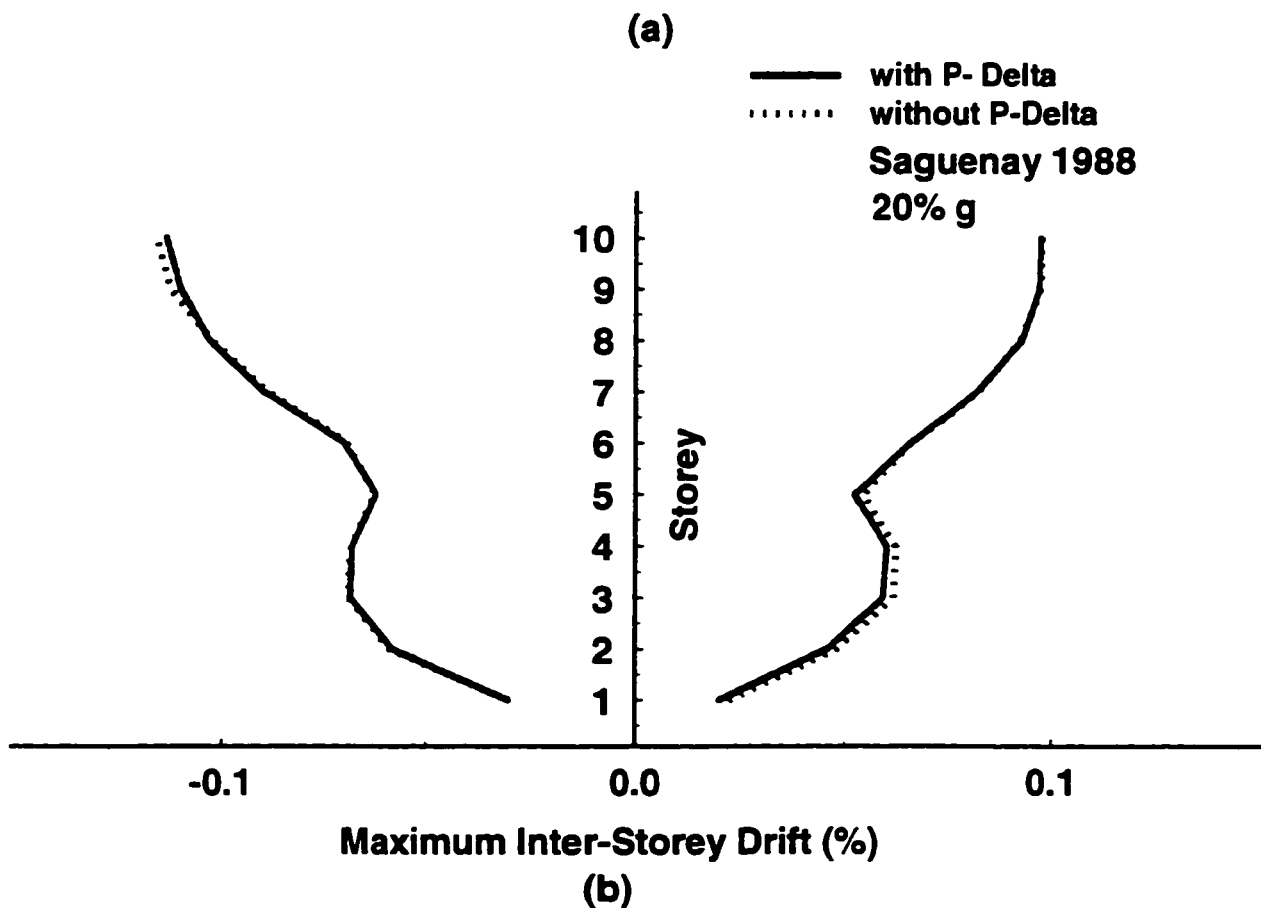
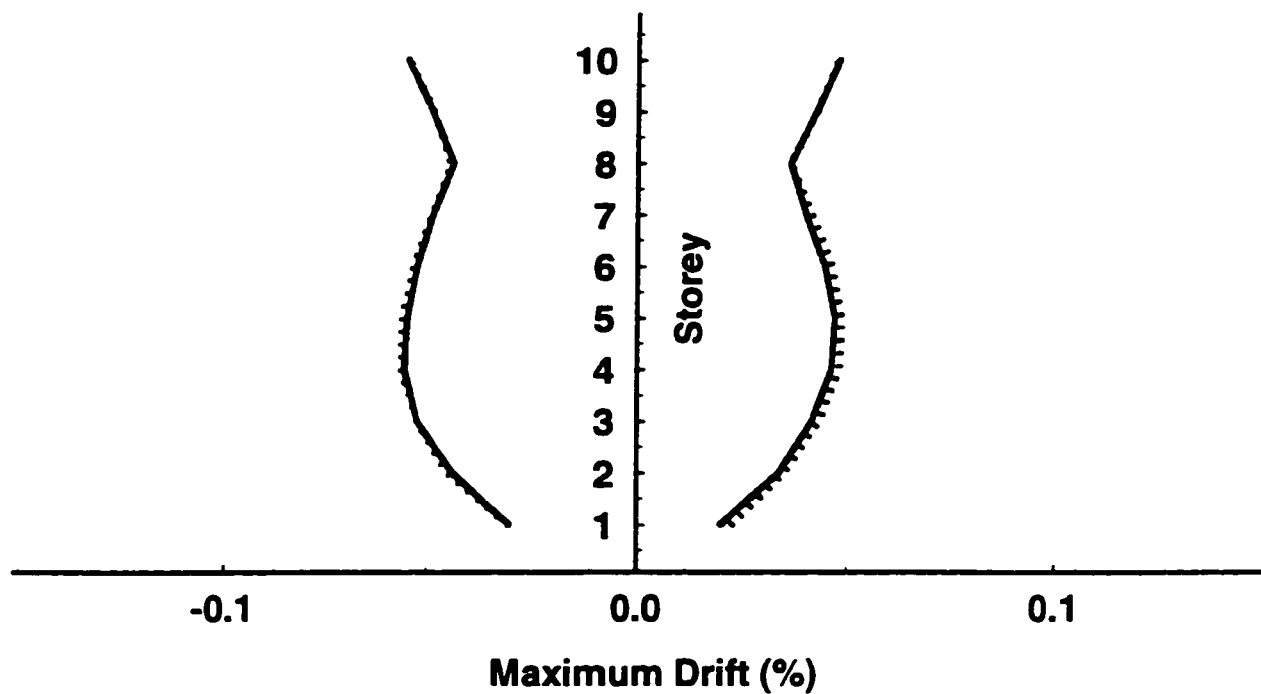


Fig. 4.70 Effect of P- Δ on Drift Response. 10-Storey SHW Subjected to 1988 Saguenay for Eastern Canada

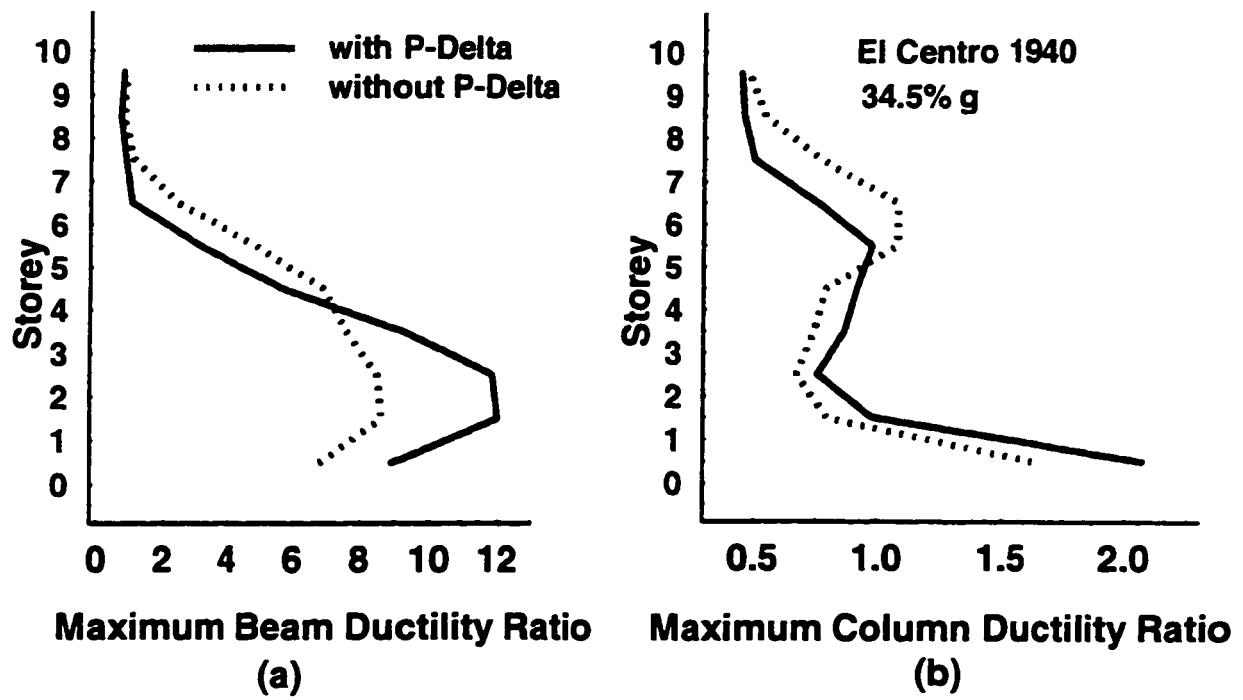
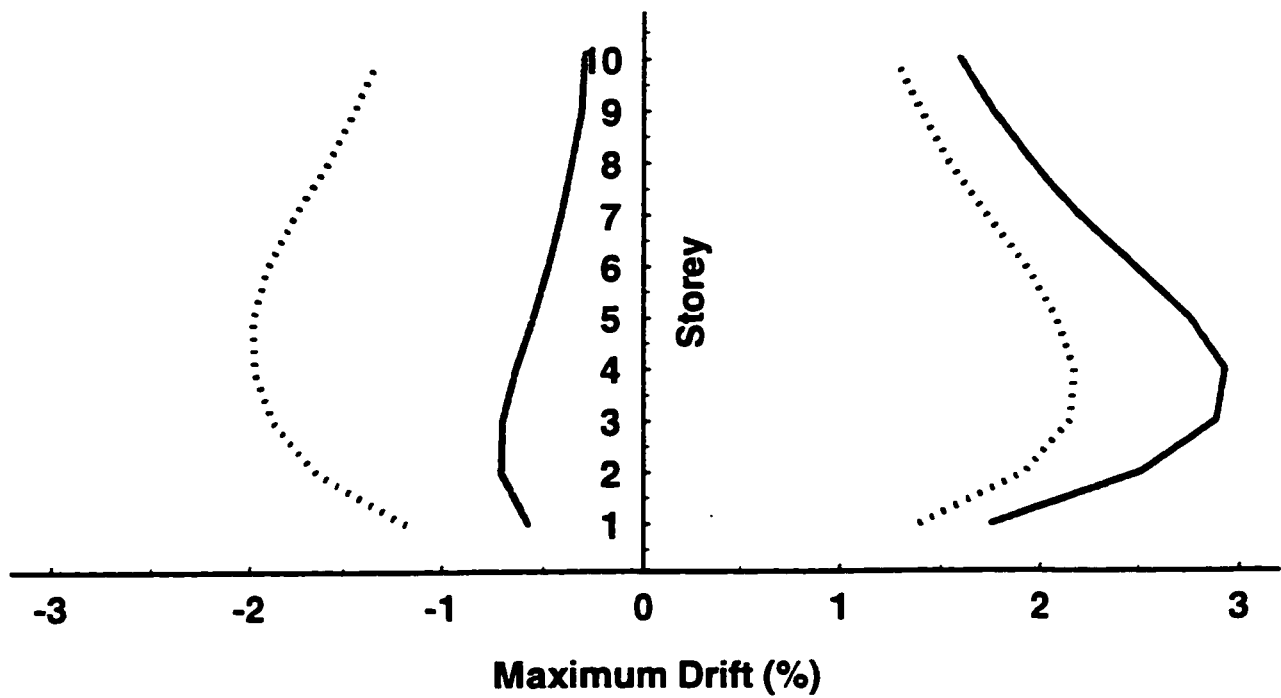
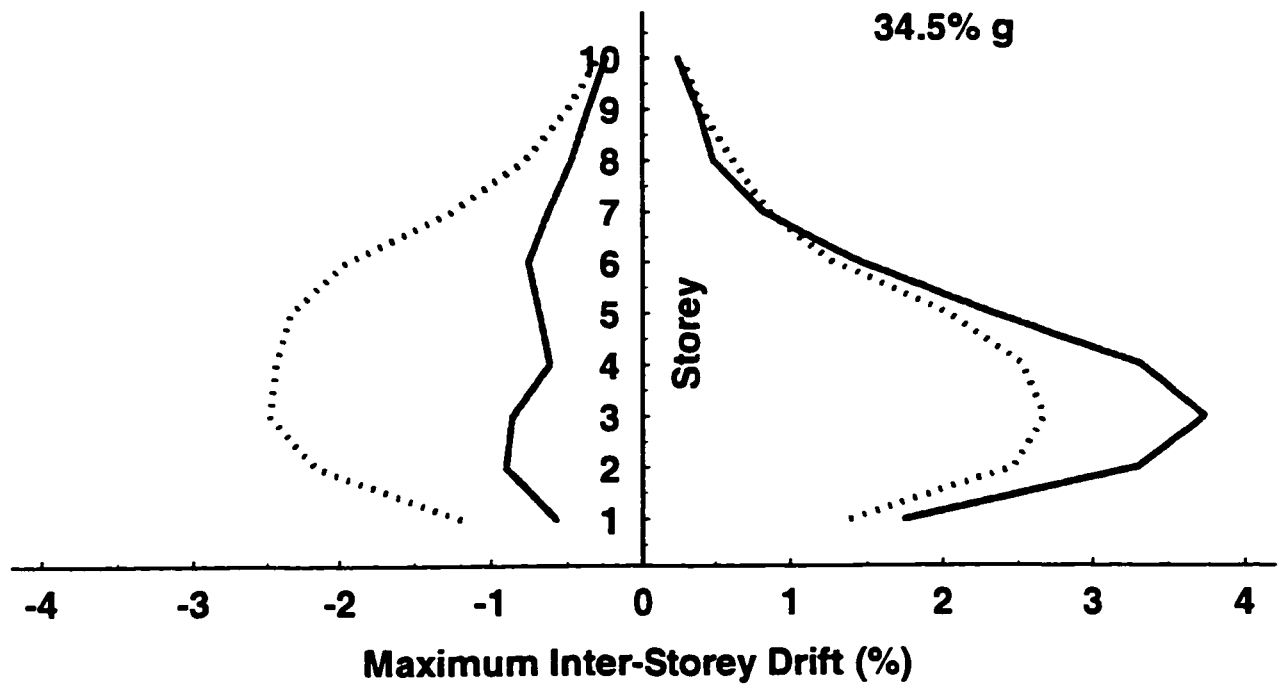


Fig. 4.71 Effect of P- Δ on Ductility Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada



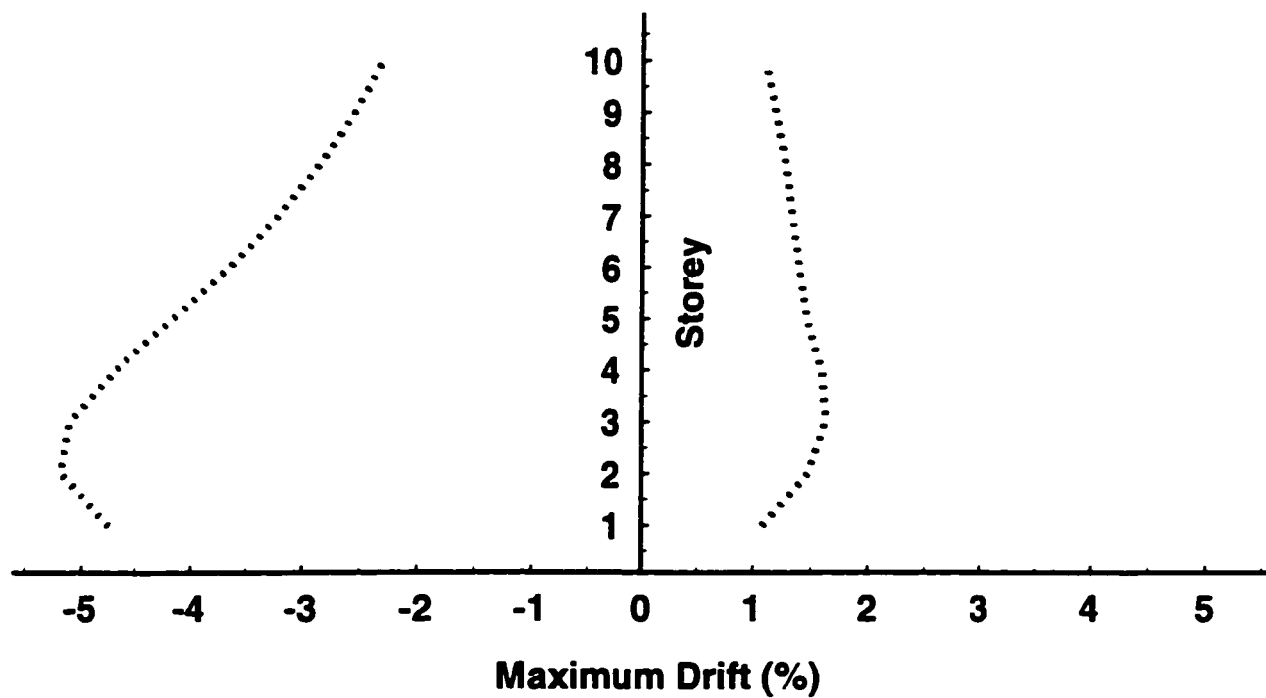
(a)

— with P- Delta
 without P-Delta
 El Centro 1940
 34.5% g



(b)

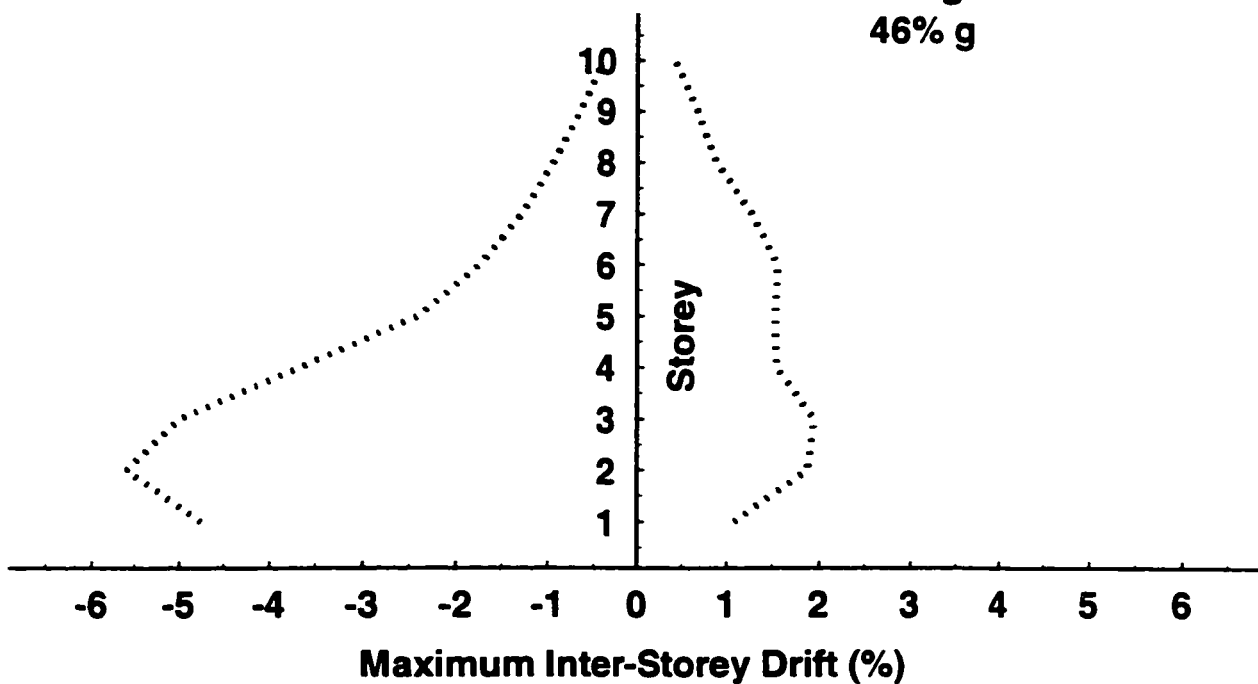
Fig. 4.72 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to 1940 El Centro for Western Canada



(a)

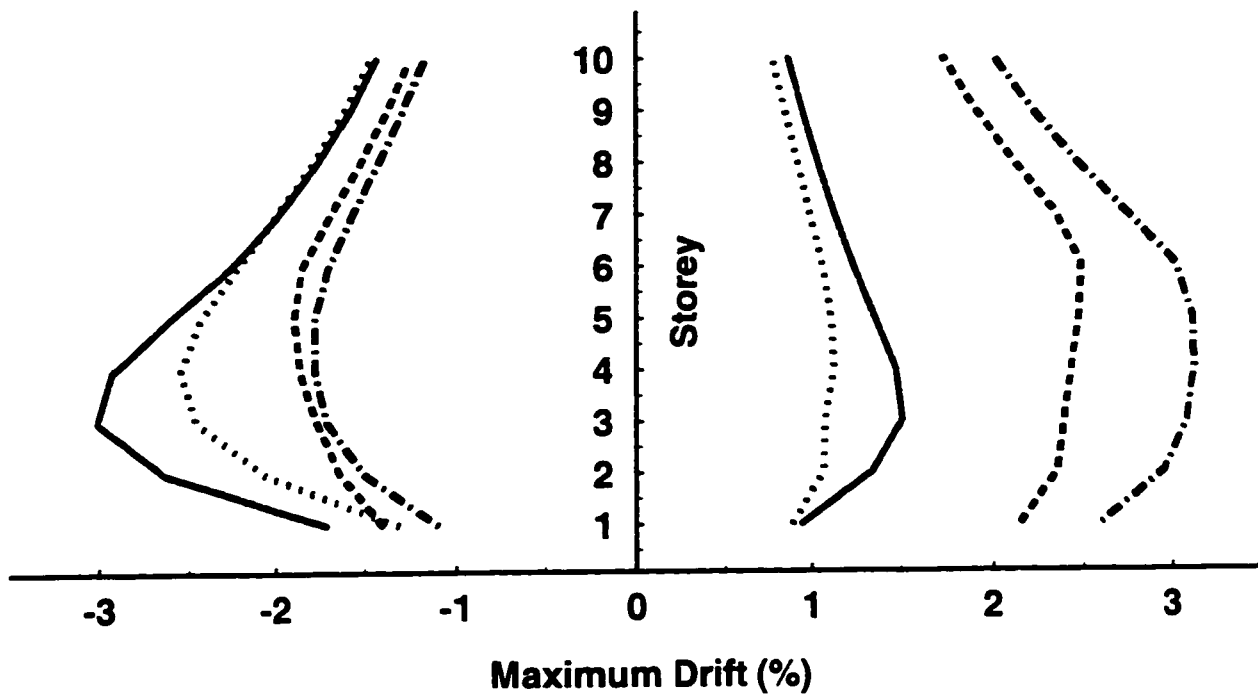
— with P- Delta (collapsed)
 without P-Delta

Long Event No. 2
 46% g

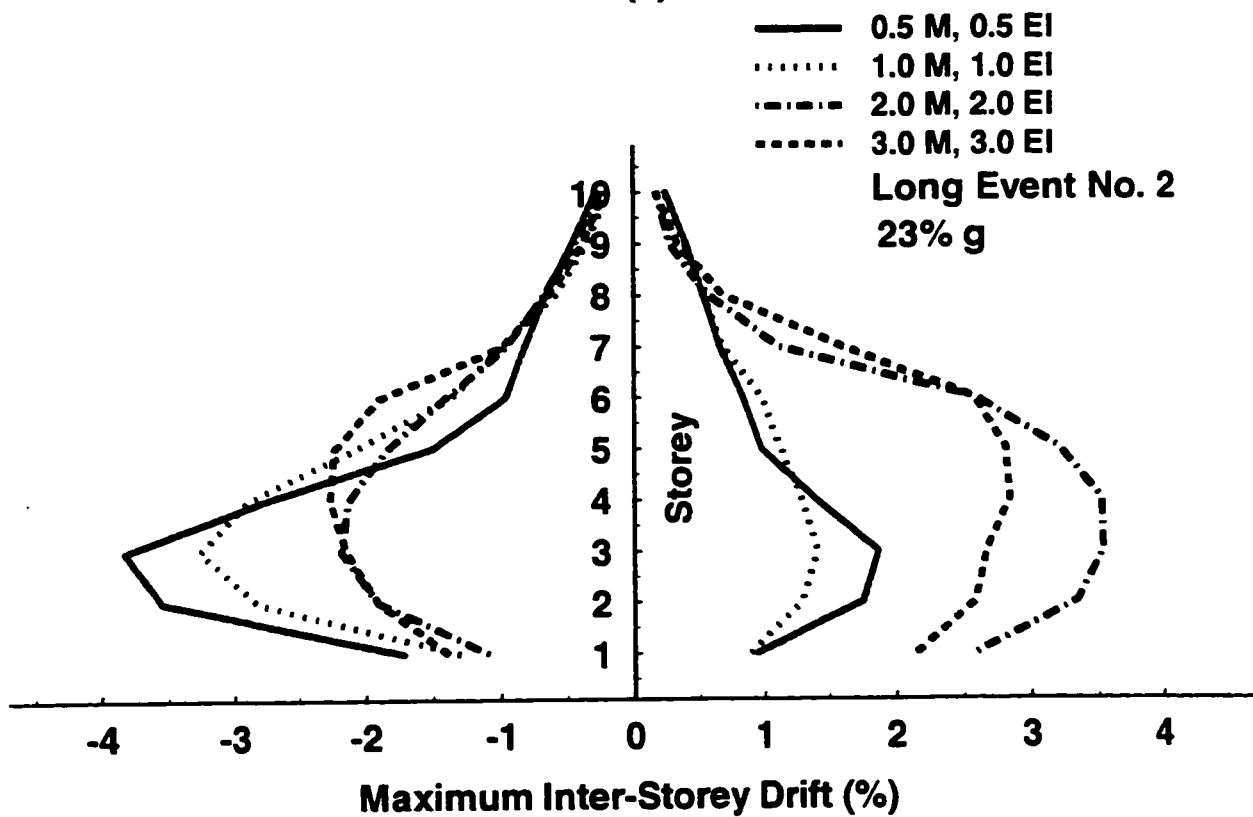


(b)

Fig. 4.73 Effect of P- Δ on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

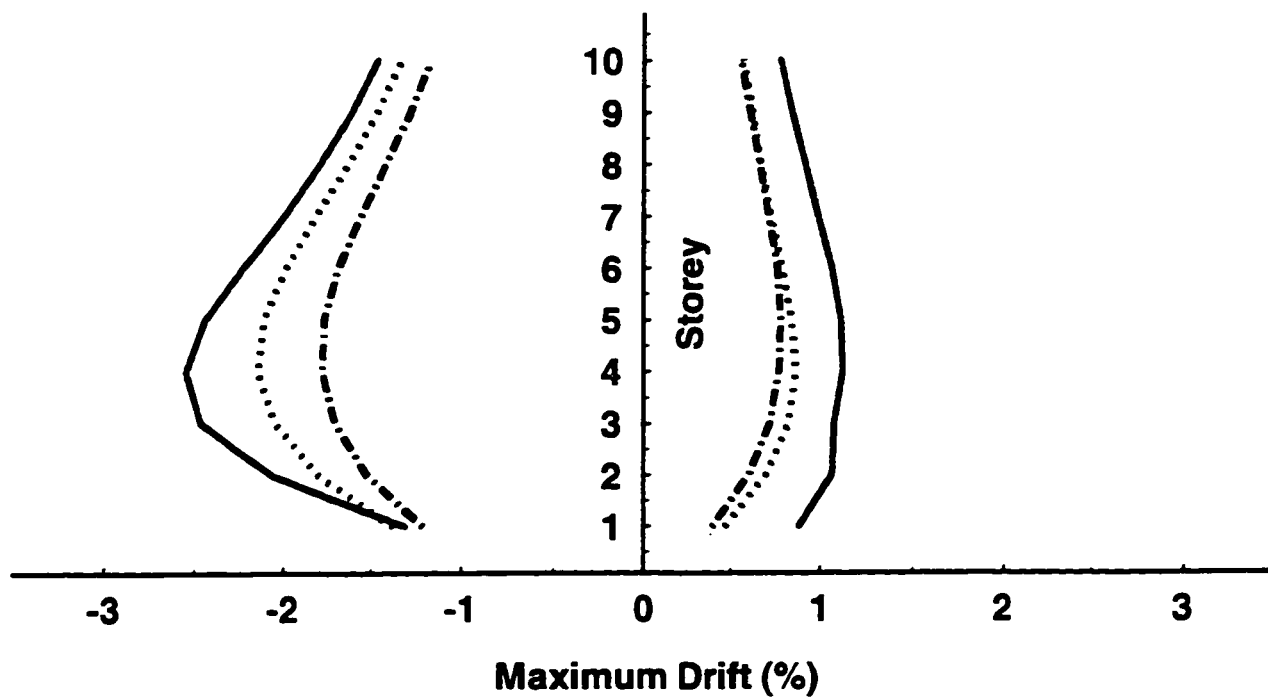


(a)



(b)

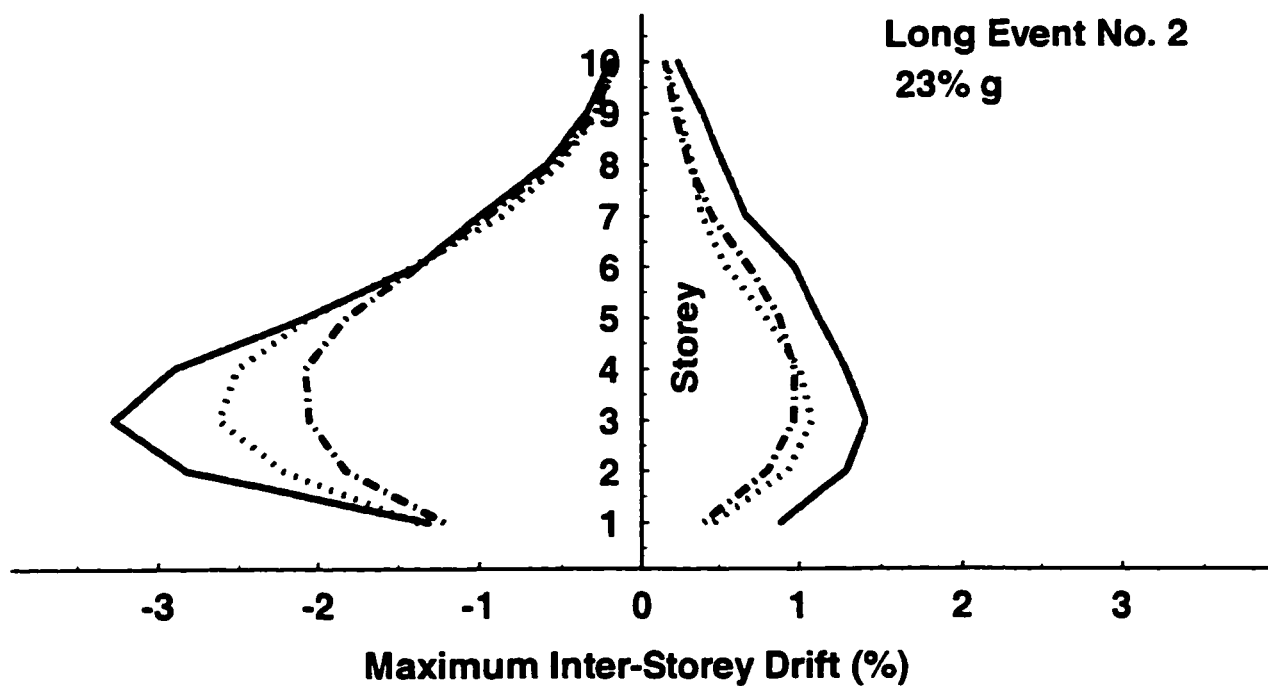
Fig. 4.74 Effect of Period on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



(a)

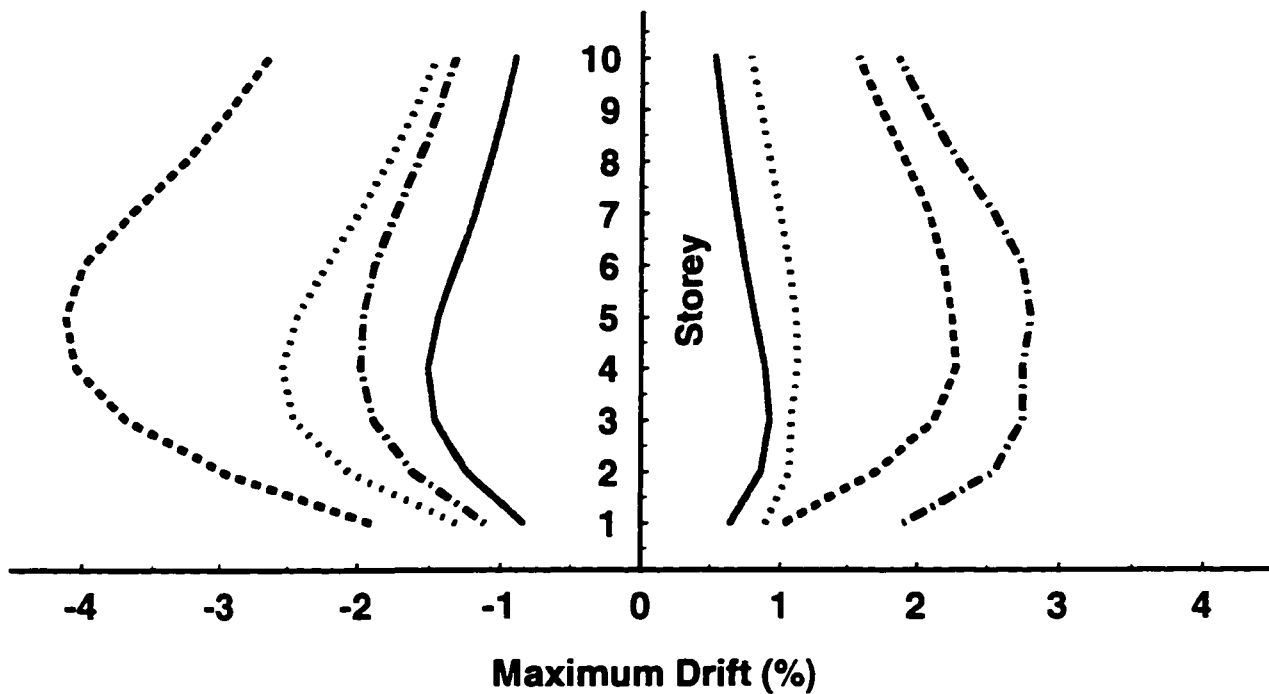
- 1.0 M, 1.0 EI
- 1.0 M, 2.0 EI
- . - . 1.0 M, 3.0 EI

Long Event No. 2
23% g

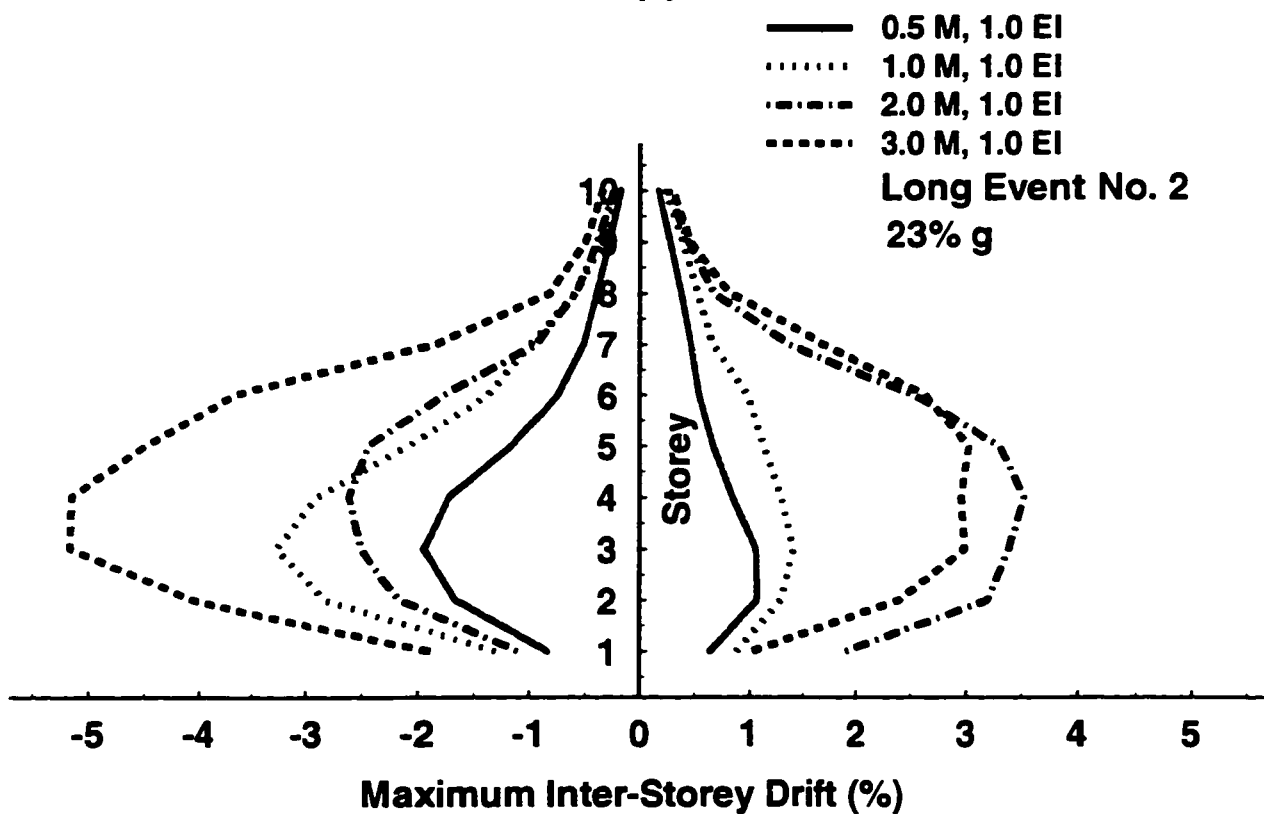


(b)

Fig. 4.75 Effect of Stiffness on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

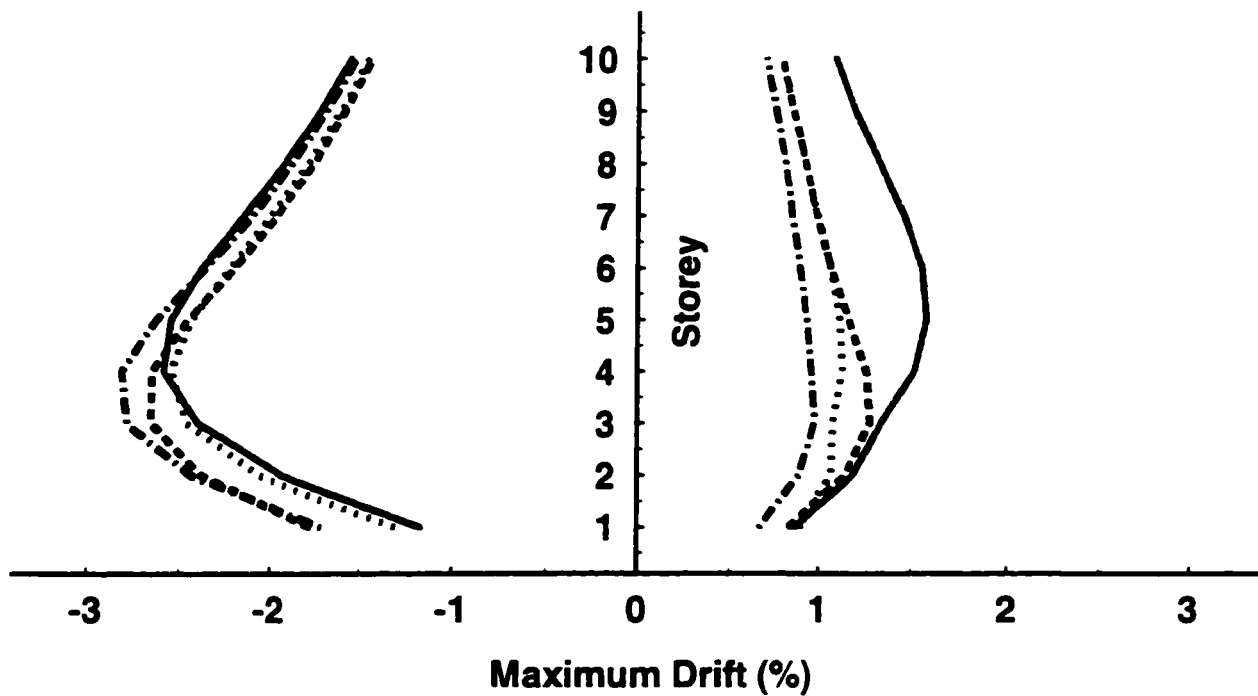


(a)

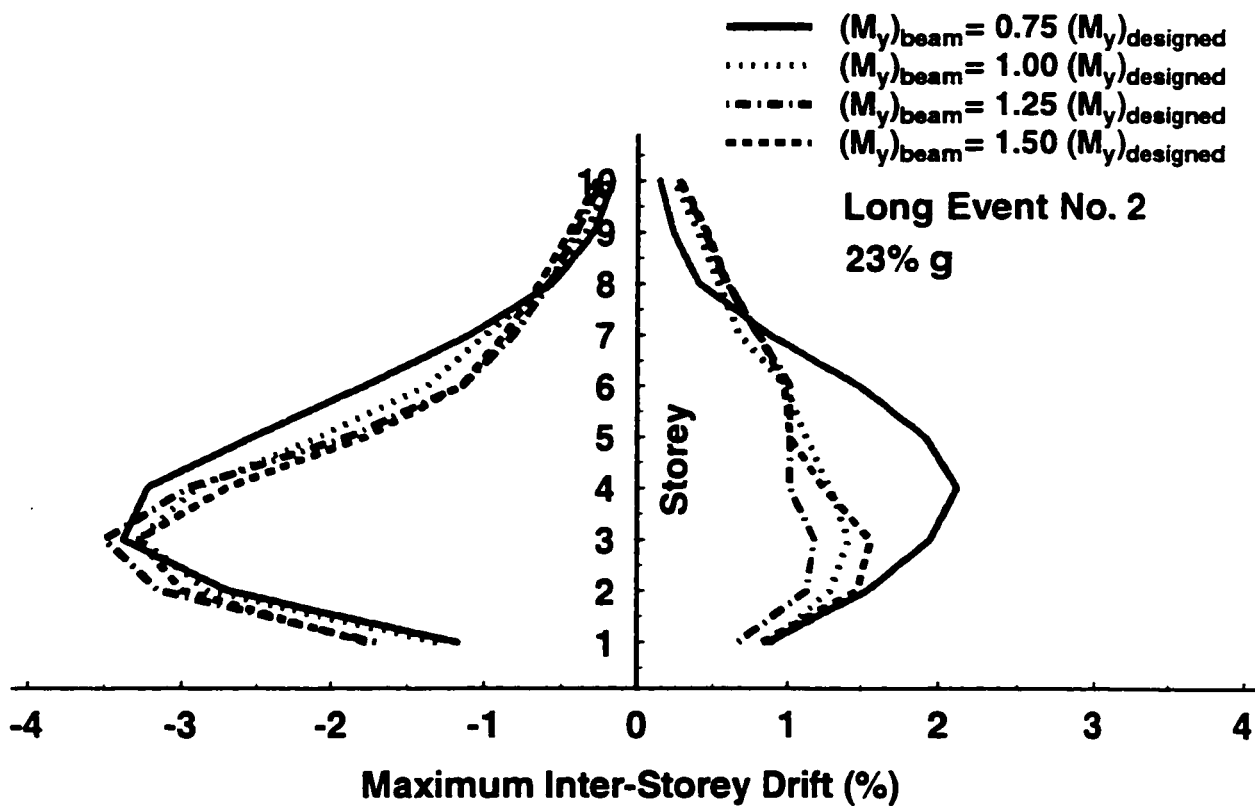


(b)

Fig. 4.76 Effect of Mass on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada



(a)



(b)

Fig. 4.77 Effect of Flexural Strength on Drift Response. 10-Storey MRF Subjected to Long Event No. 2 for Western Canada

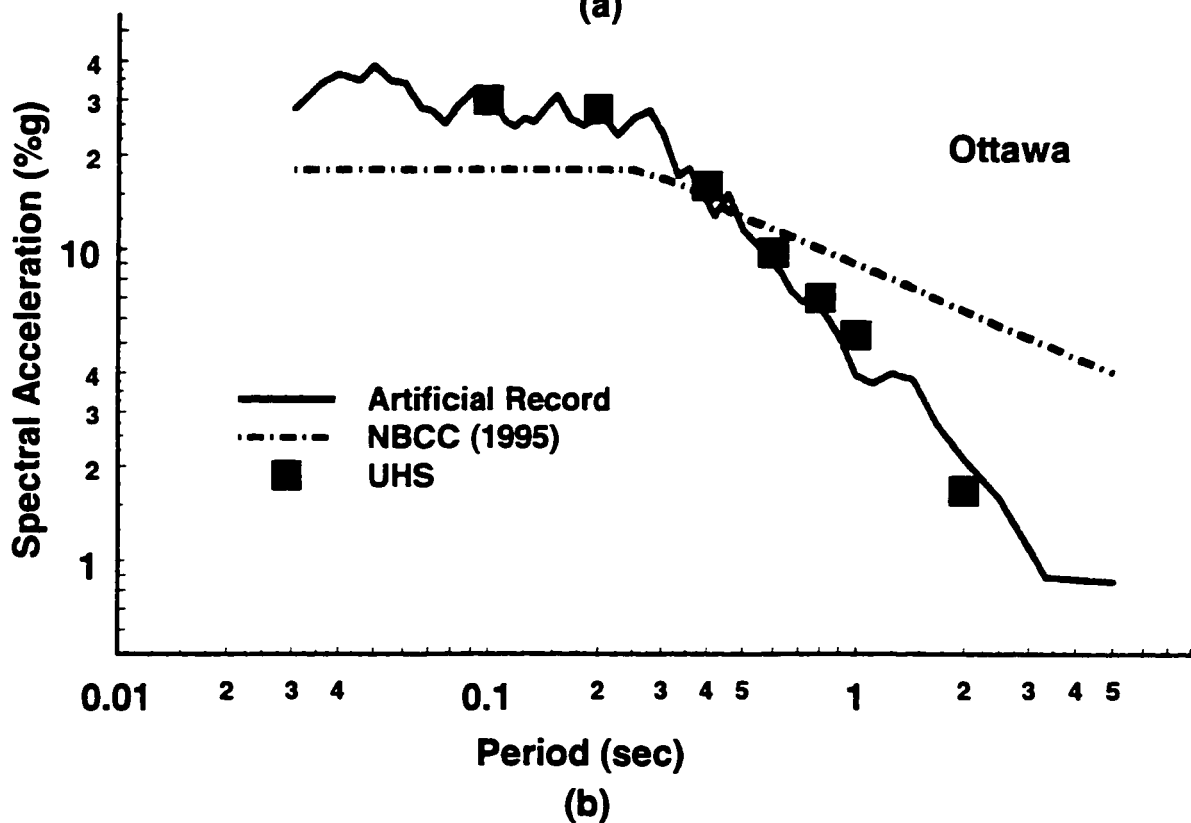
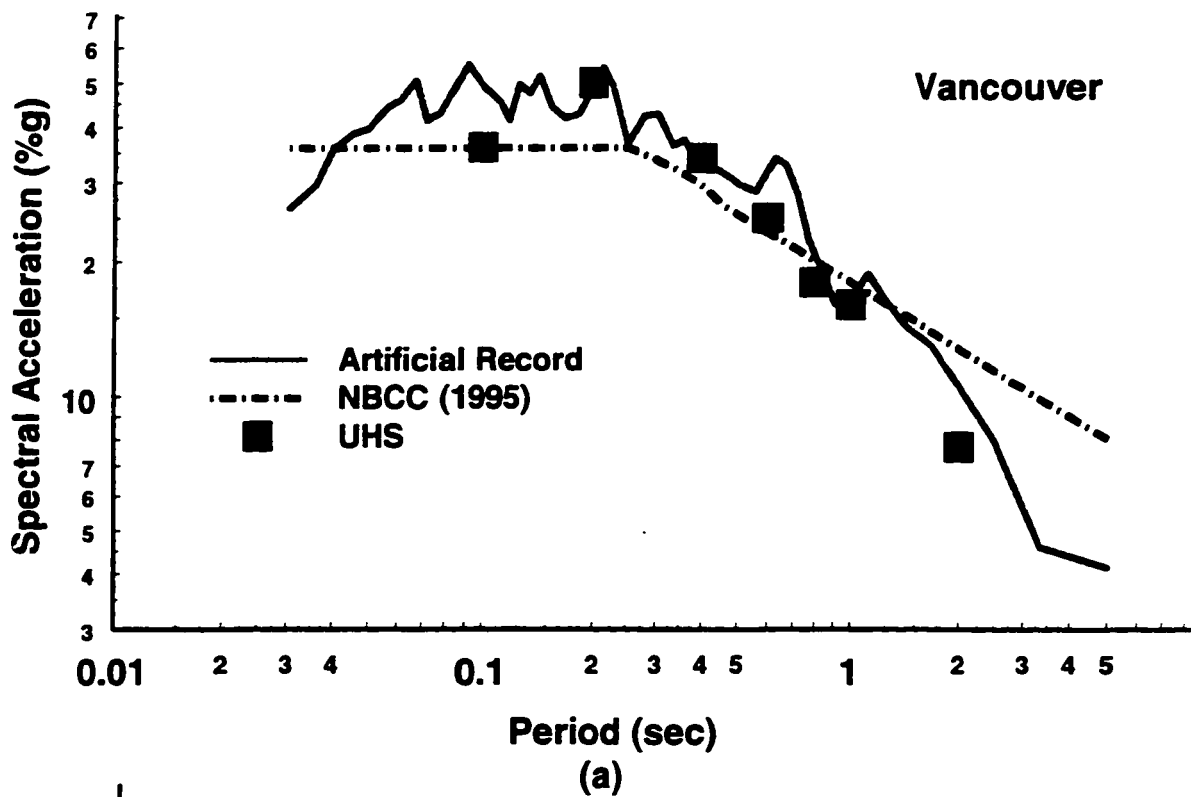


Fig. 5.1 Comparisons of NBCC-95 Design Response Spectra, UHS, and the Spectra for Artificial Records.

Western Canada

- Short Event No.1
- Short Event No.2
- .-.-. Long Event No.1
- Long Event No.2

Used for 5-Storey Frame
Shear Wall Building (T=0.59 sec)

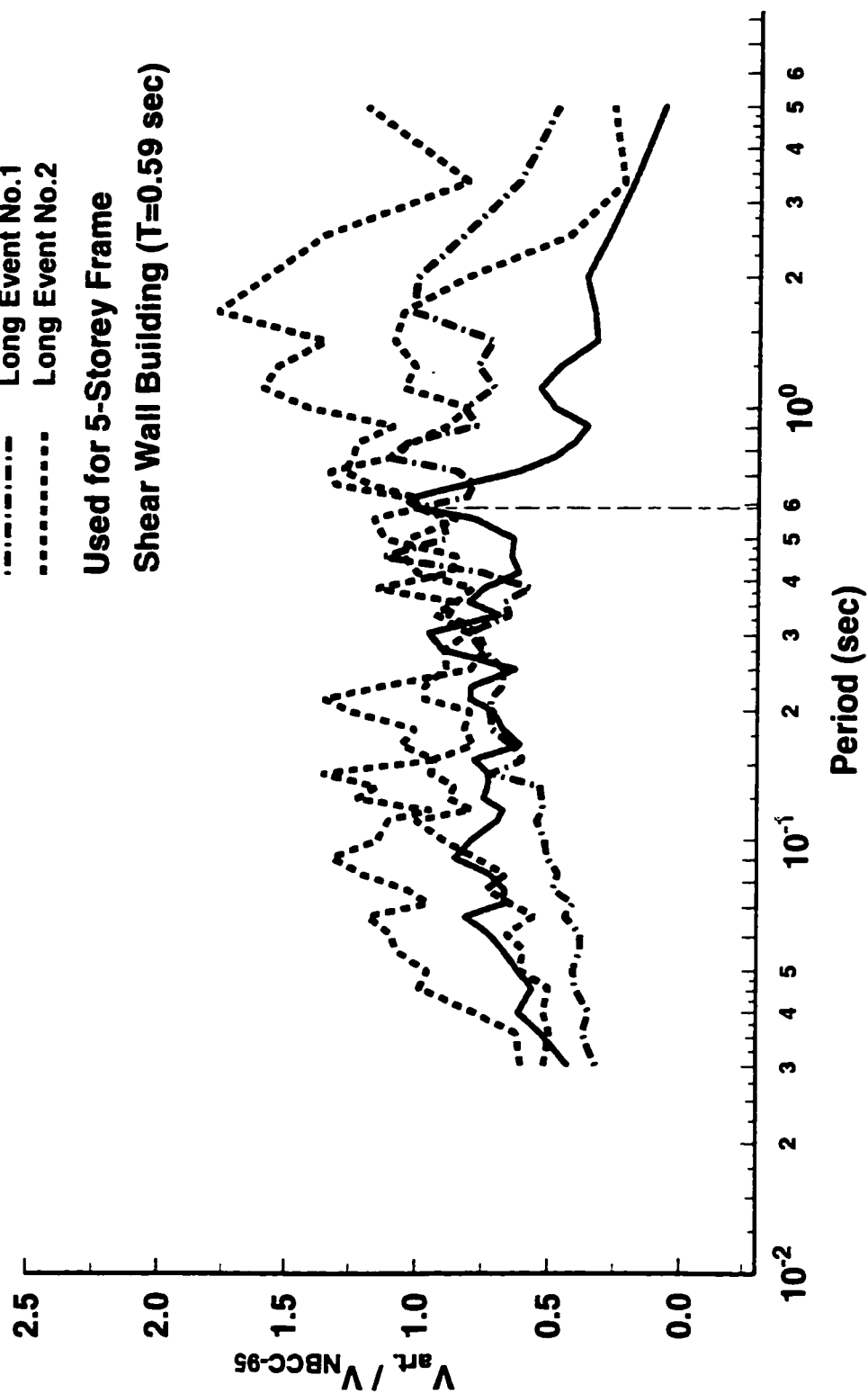
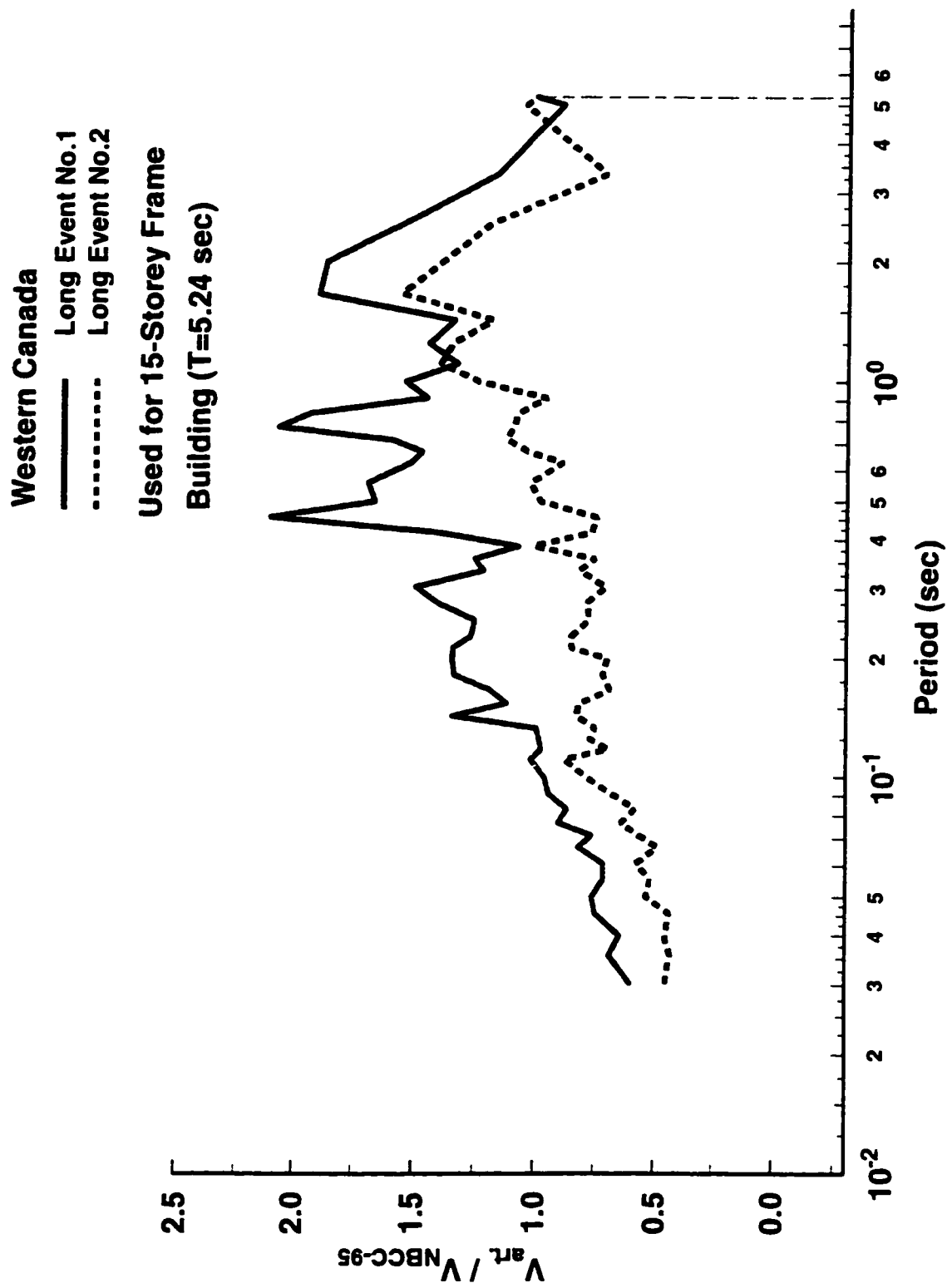
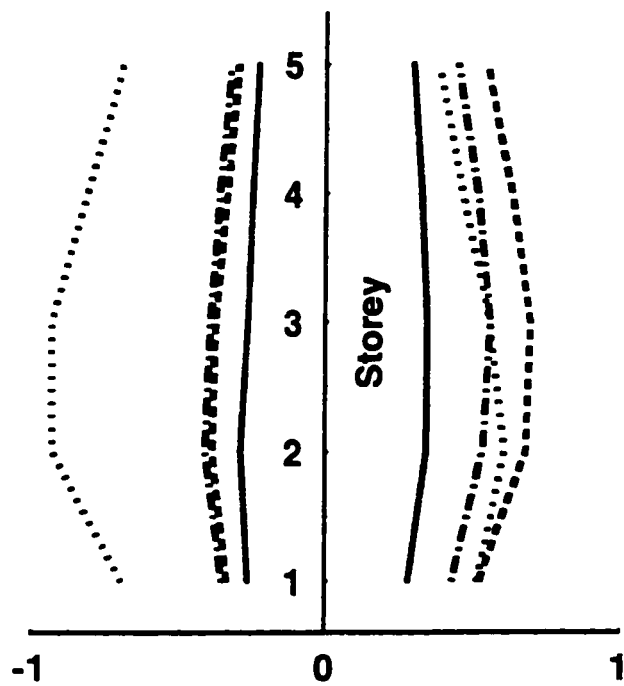


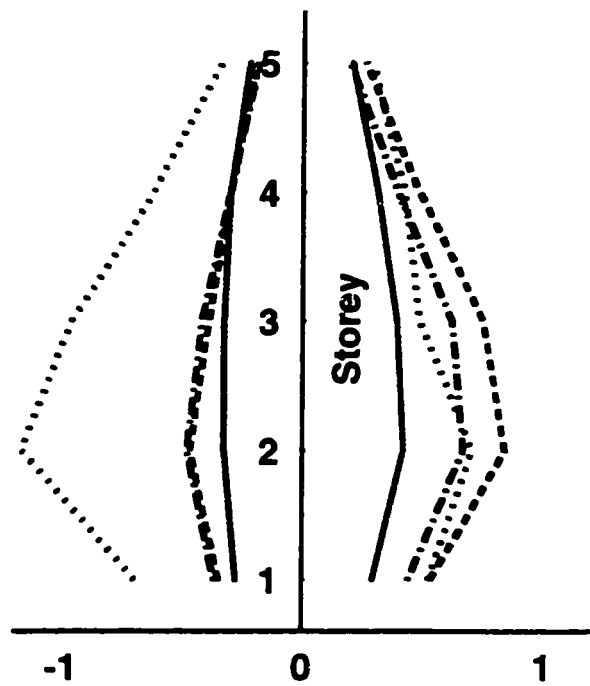
Fig. 5.2 Ratio of Base Shear versus Period for Scaled Artificial Records
Used for 5-Storey Shear Wall Building.



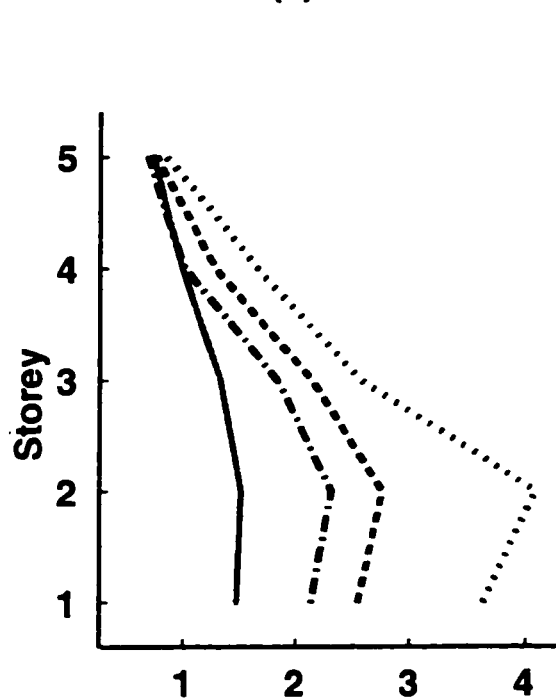
**Fig. 5.3 Ratio of Base Shear versus Period for Scaled Artificial Records
 Used for 15-Storey Frame Building.**



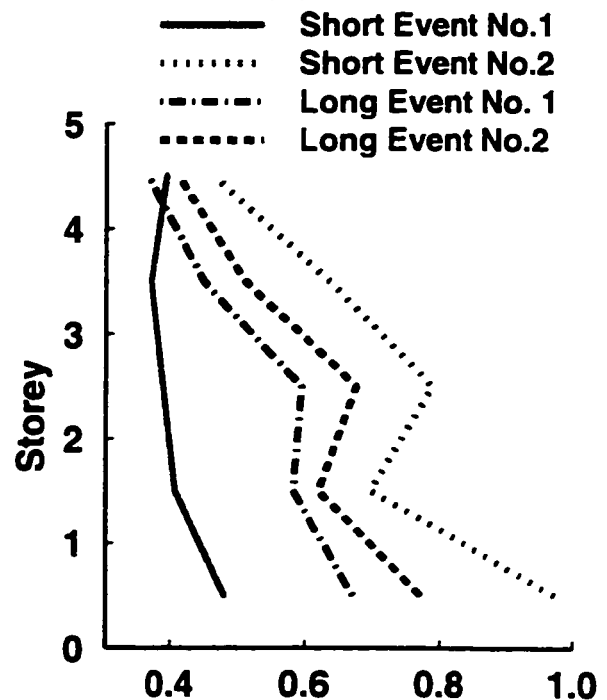
Max. Drift (%)
(a)



Max. Inter-Storey Drift (%)
(b)

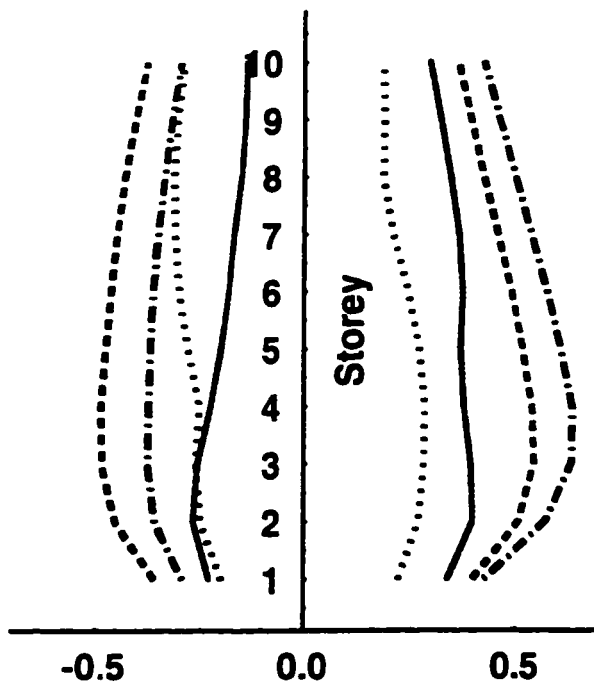


Max. Beam Ductility Ratio
(c)



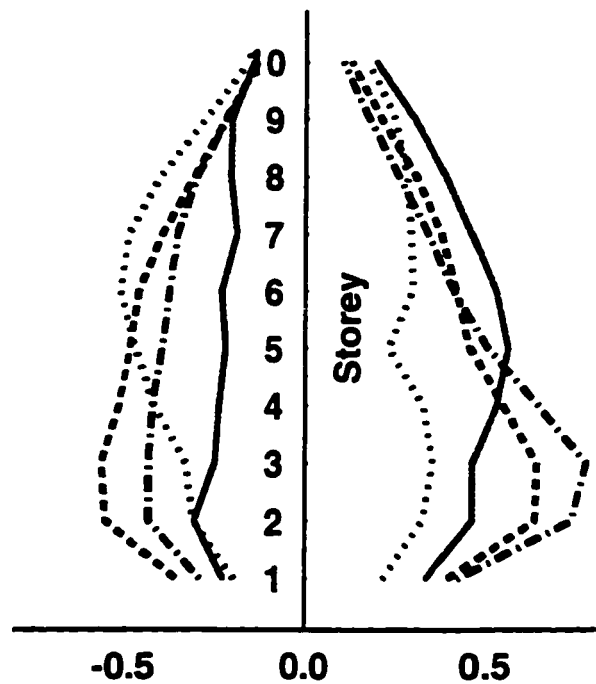
Max. Column Ductility Ratio
(d)

Fig. 5.4 Drift and Ductility Response for 5-Storey MRF Subjected to Artificial Records for Western Canada.



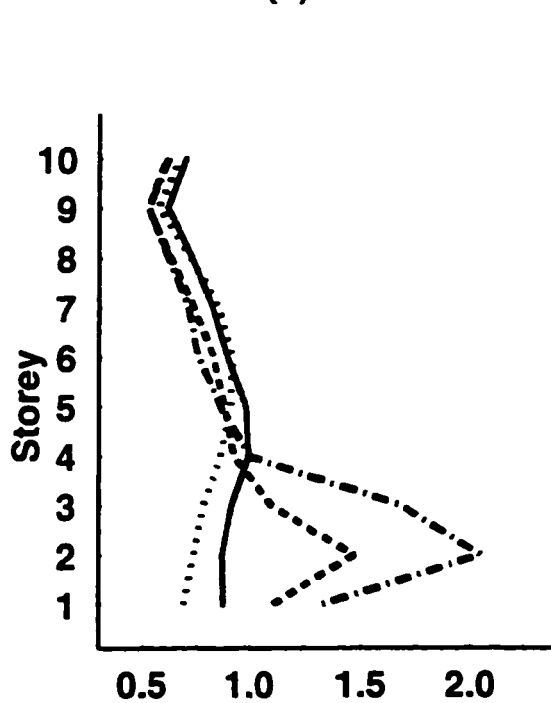
Max. Drift (%)

(a)



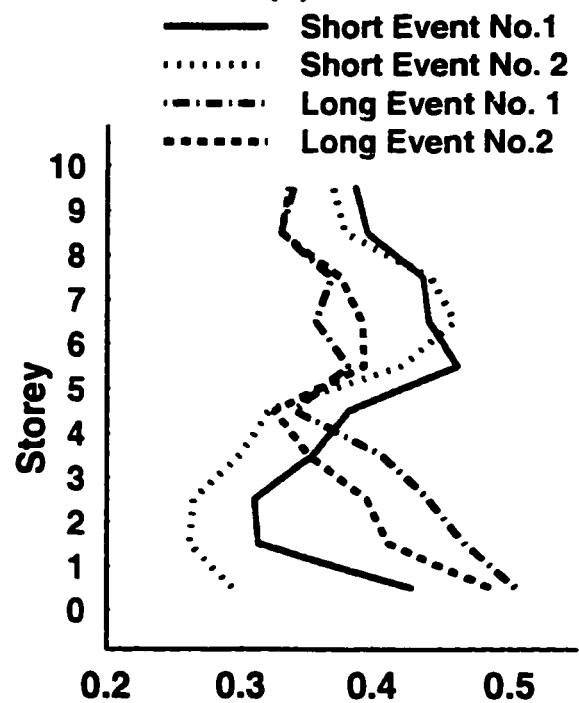
Max. Inter-Storey Drift (%)

(b)



Max. Beam Ductility Ratio

(c)



Max. Column Ductility Ratio

(d)

Fig. 5.5 Drift and Ductility Response for 10-Storey MRF Subjected to Artificial Records for Western Canada.

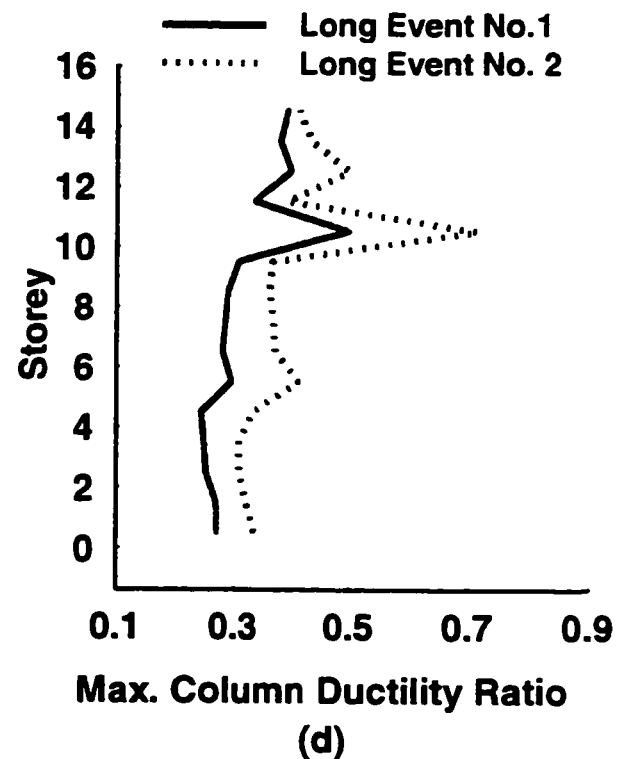
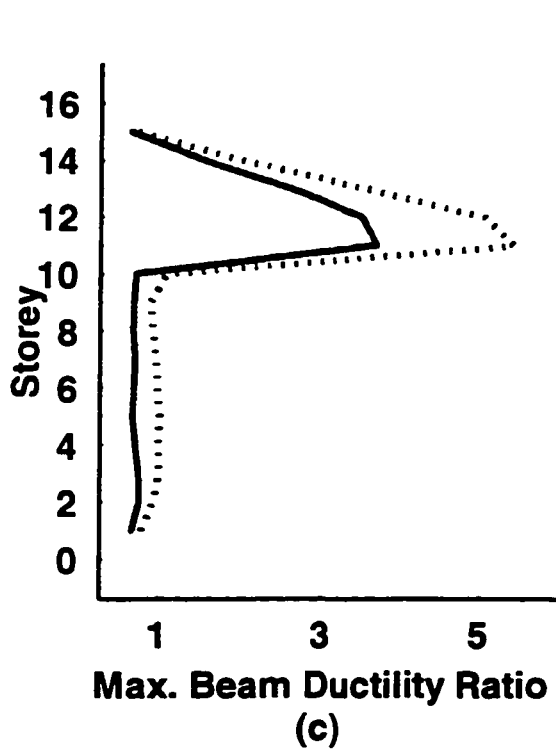
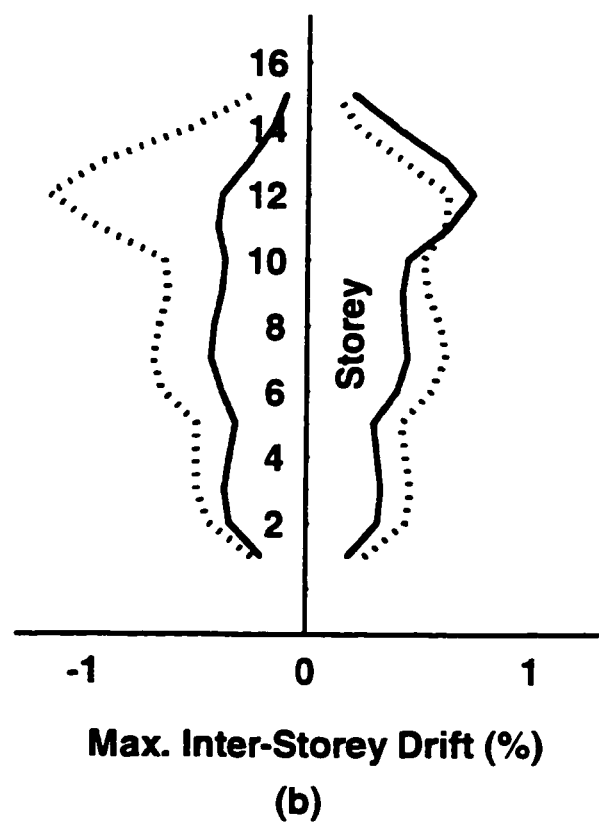
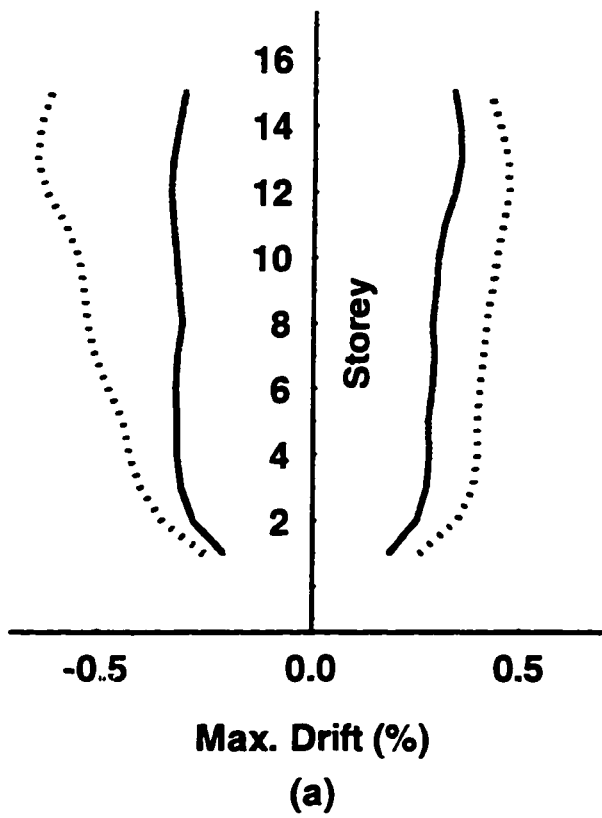


Fig. 5.6 Drift and Ductility Response for 15-Storey MRF Subjected to Artificial Records for Western Canada.

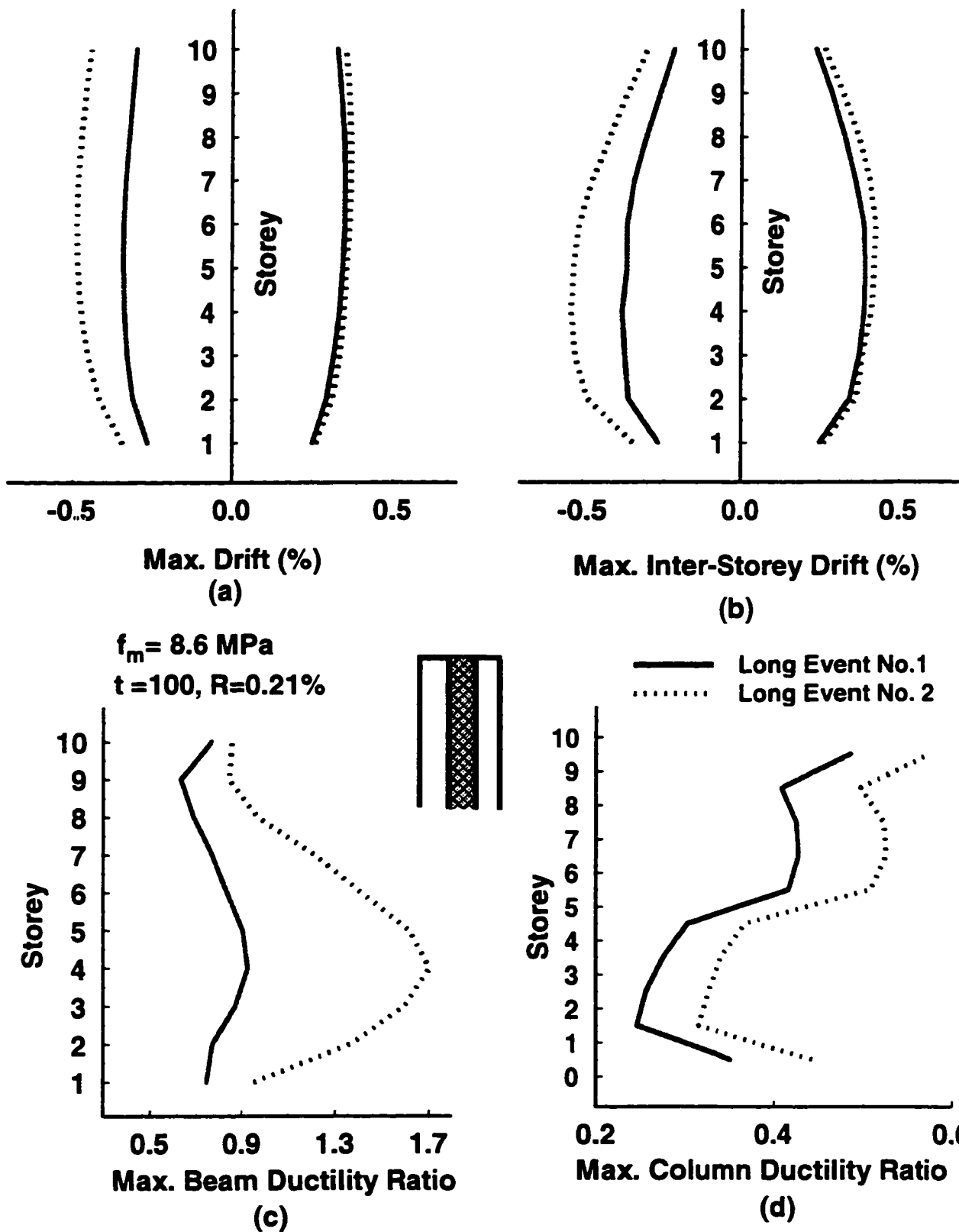


Fig. 5.7 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

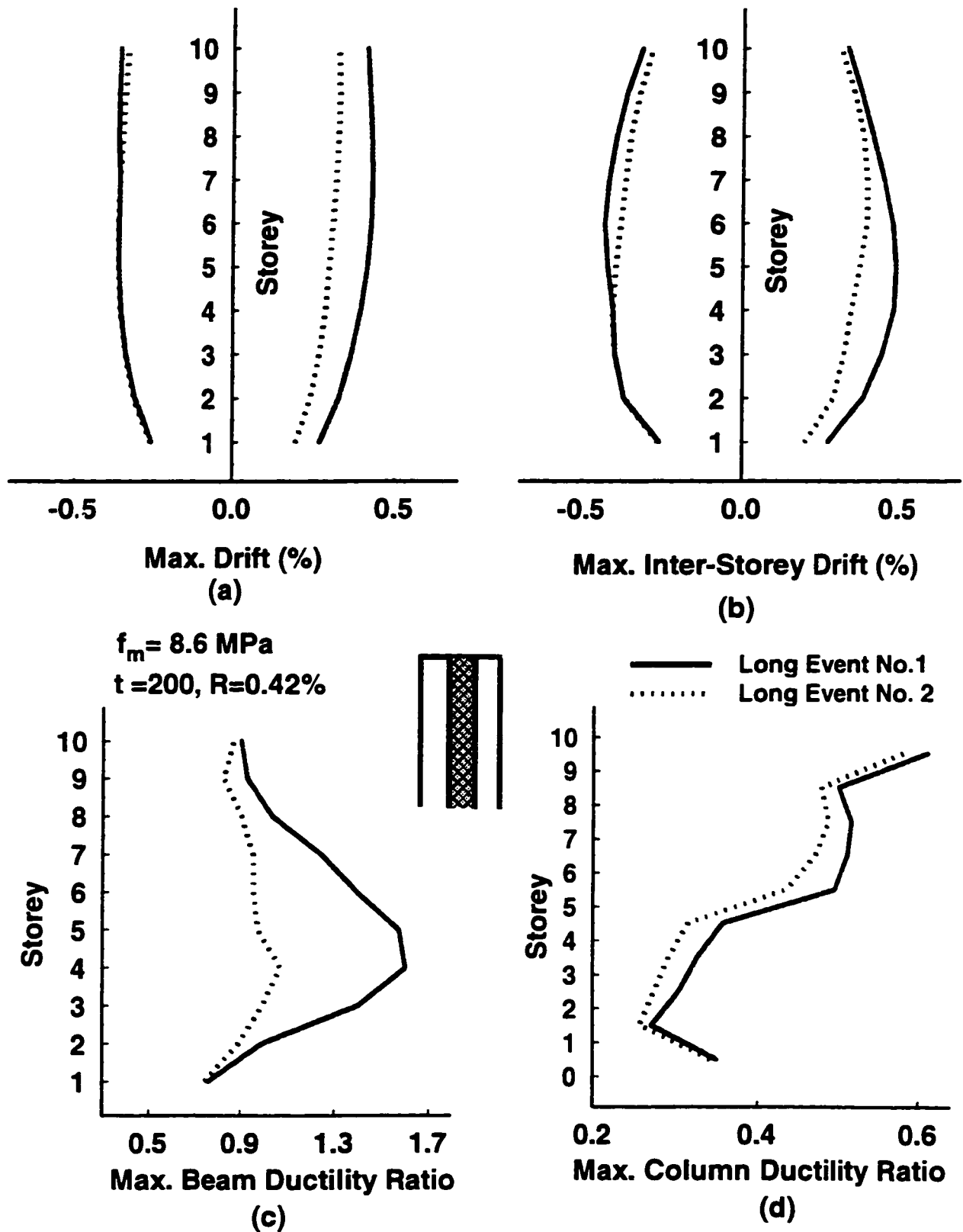


Fig. 5.8 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=200$ mm, $f_m=8.6$ MPa) .

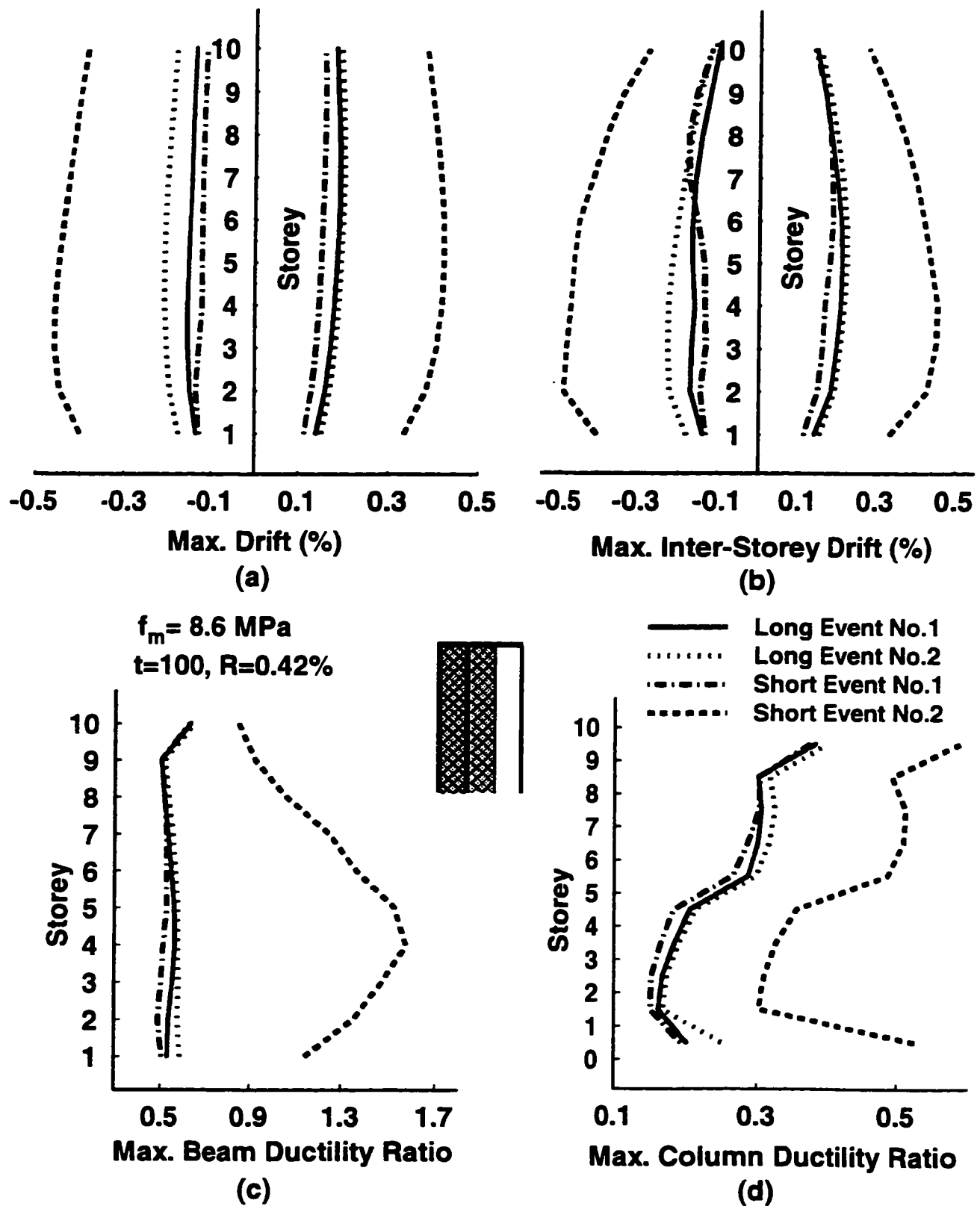


Fig. 5.9 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

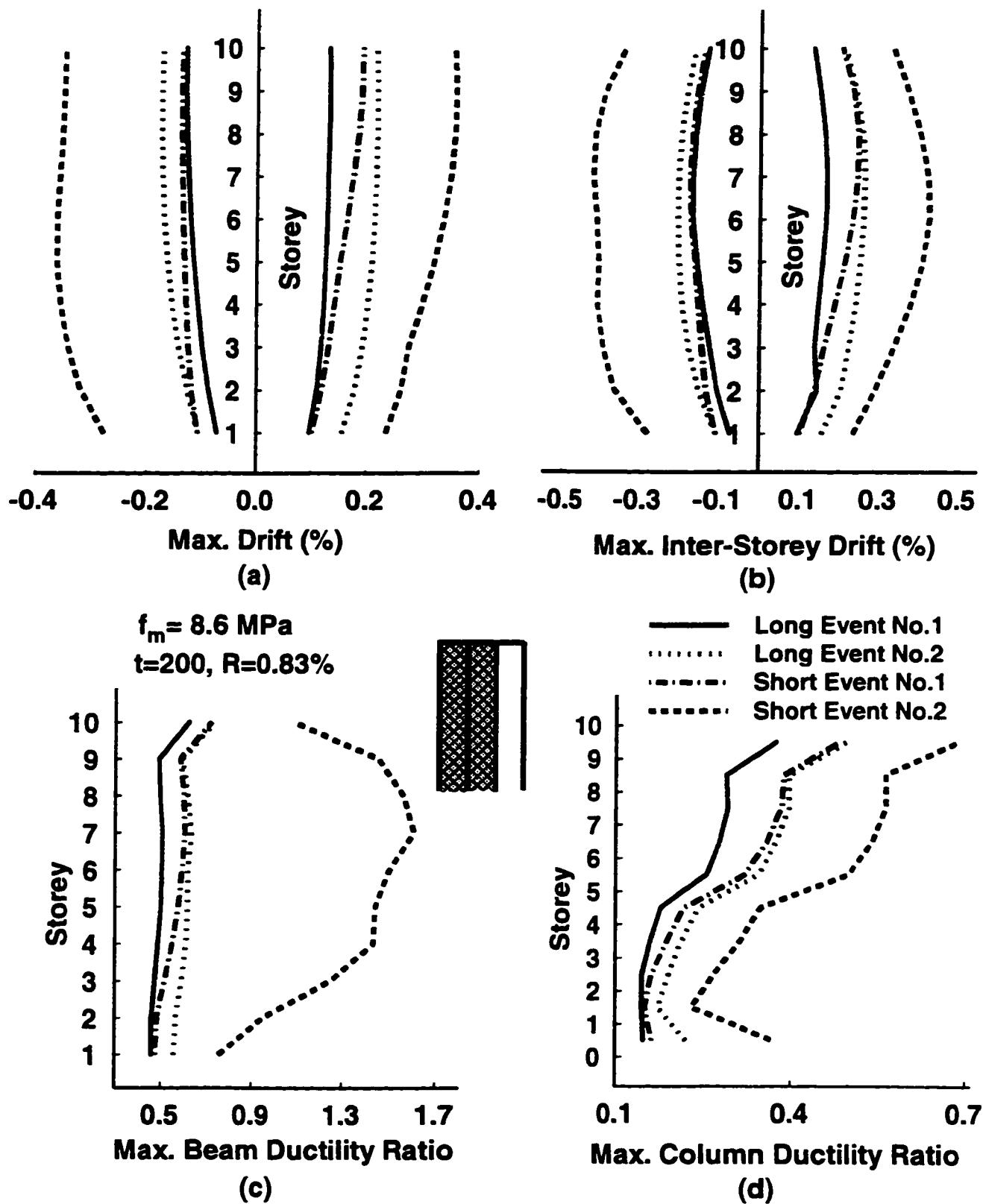


Fig. 5.10 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

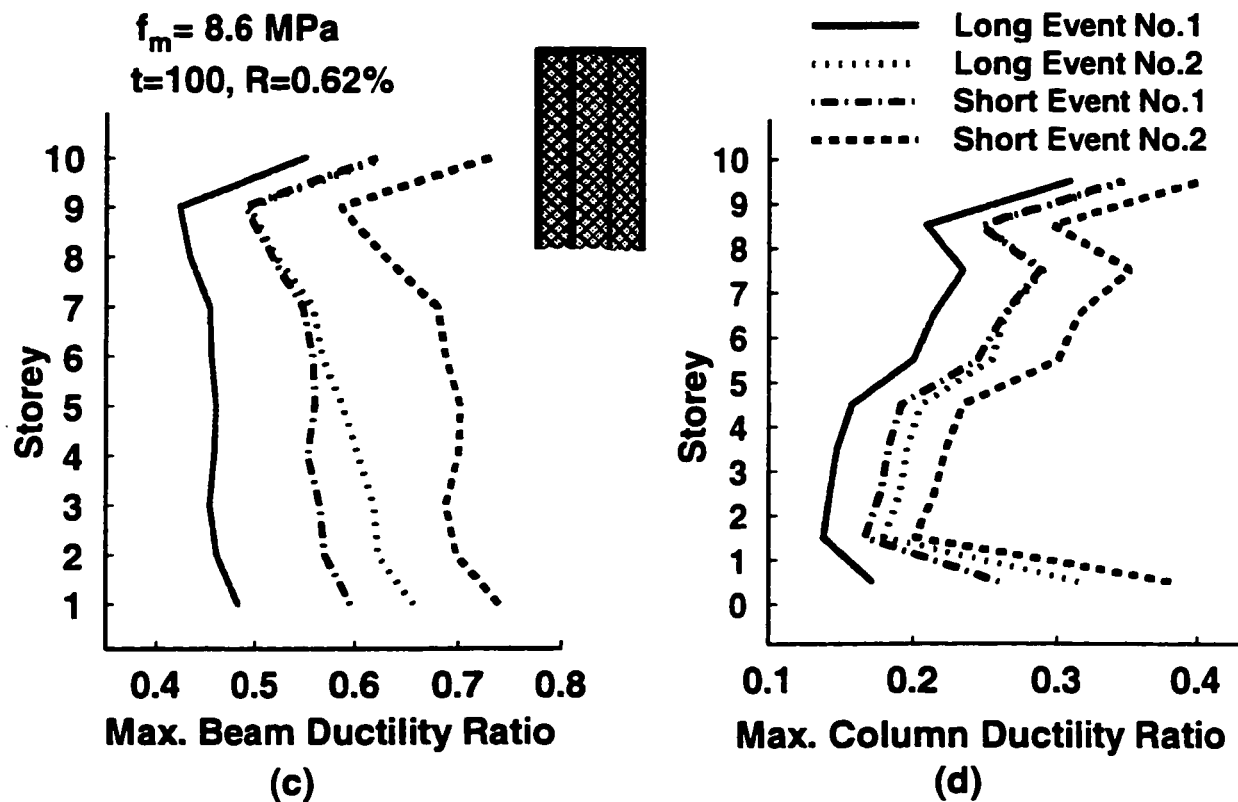
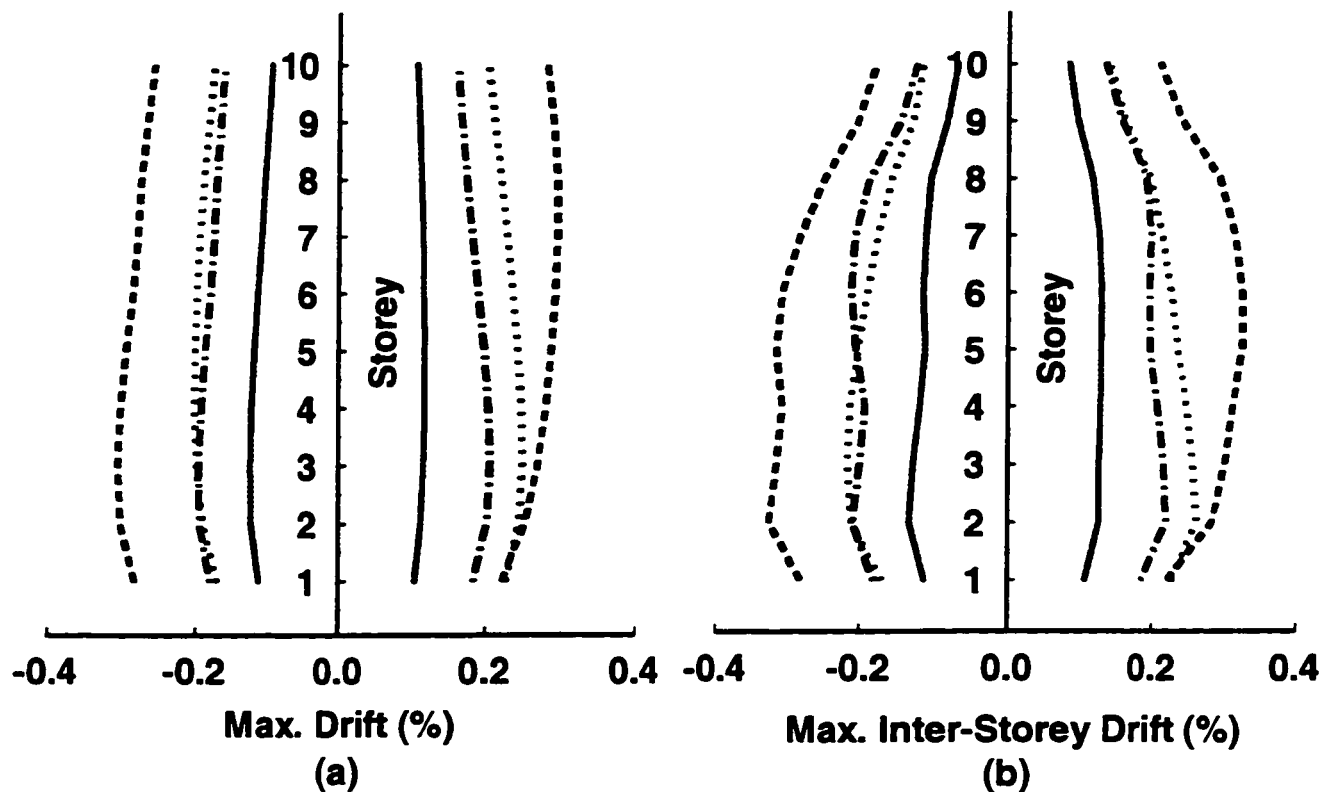


Fig. 5.11 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$) .

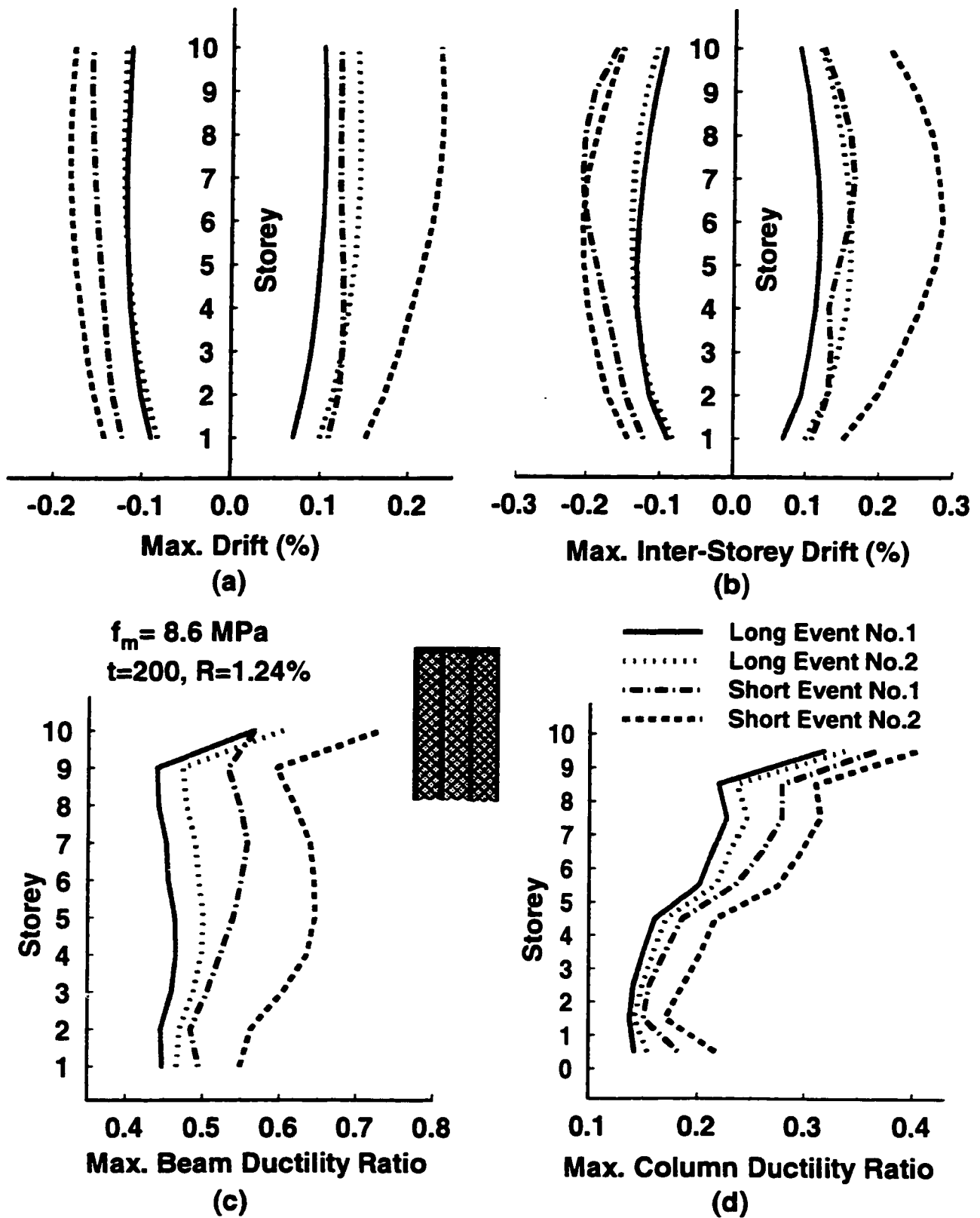


Fig. 5.12 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=200$ mm, $f_m=8.6$ MPa) .

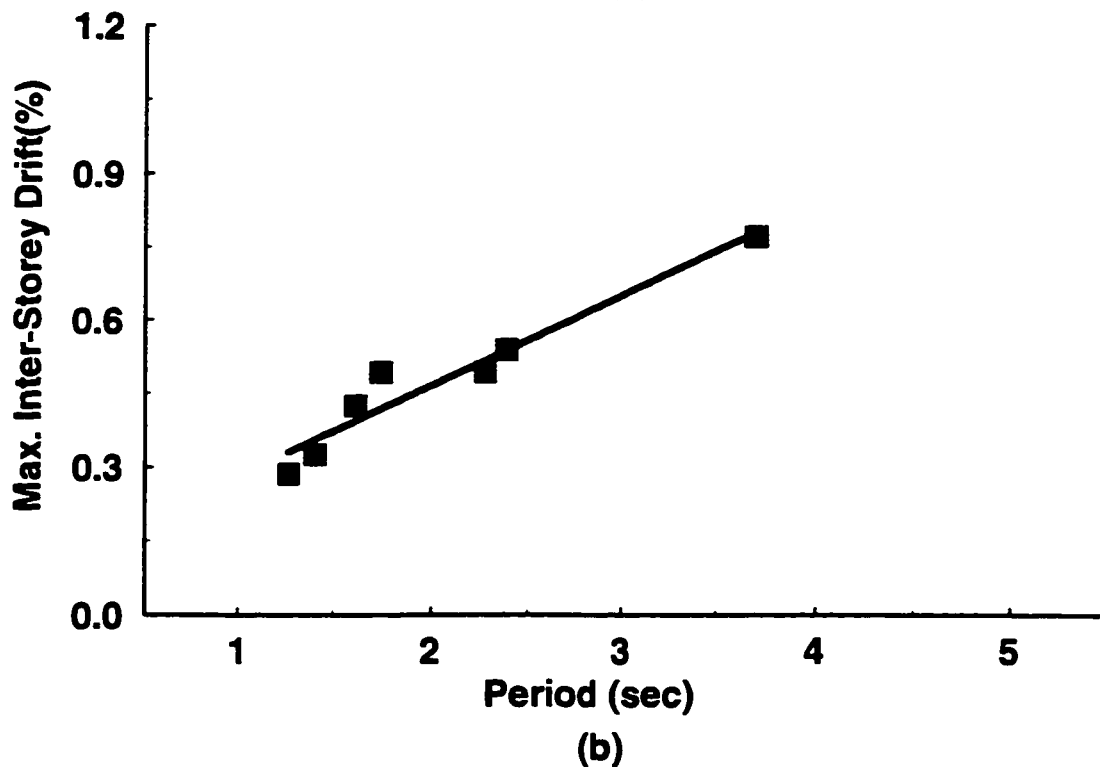
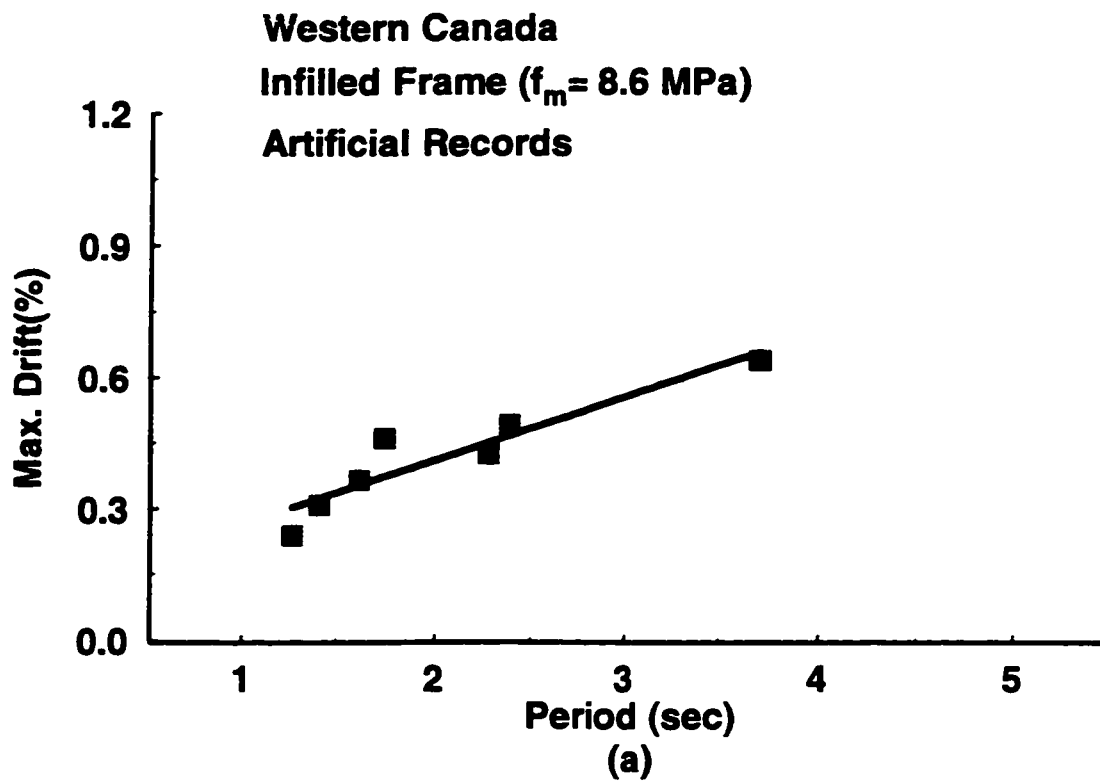


Fig. 5.13 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Artificial Records ($f_m = 8.6$ MPa).

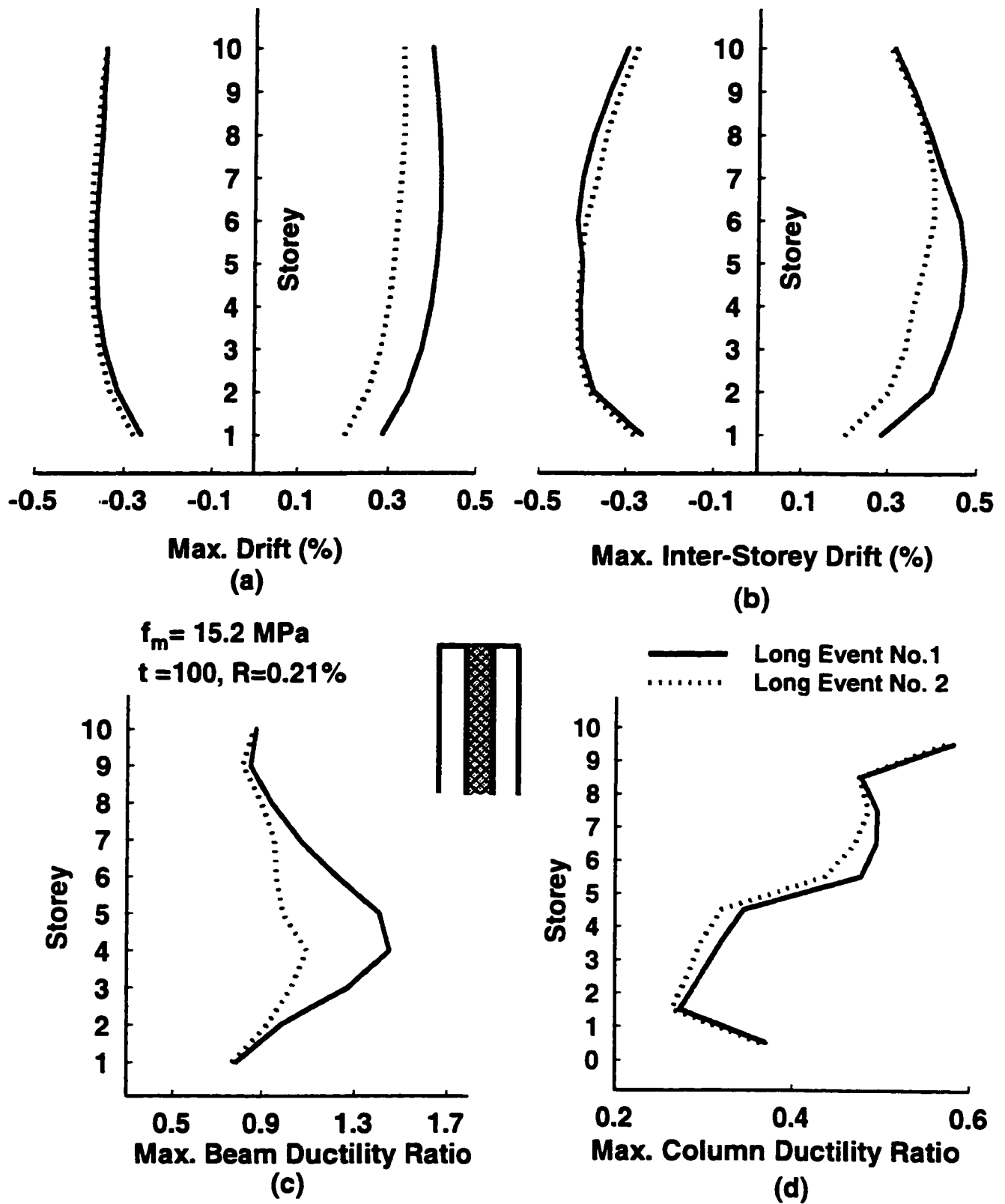


Fig. 5.14 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=100$ mm, $f_m=15.2$ MPa) .

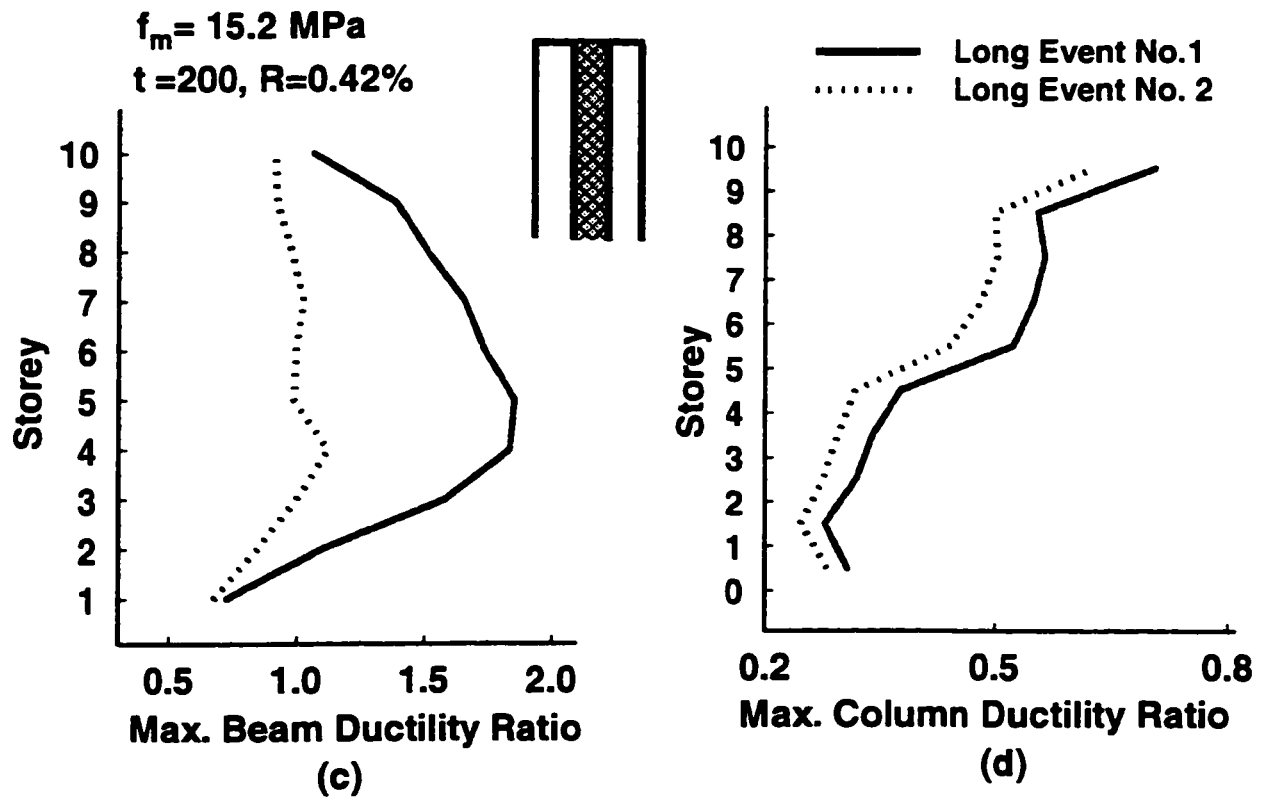
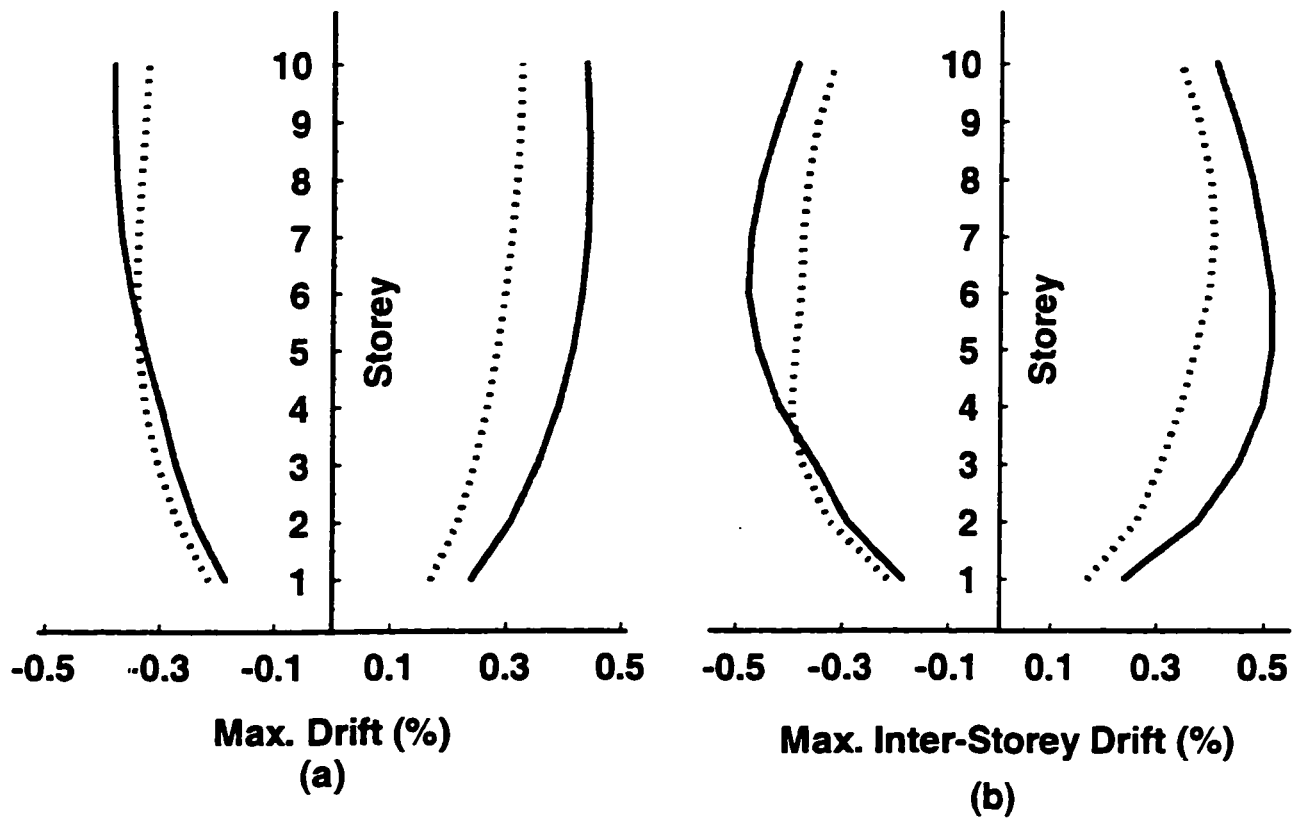


Fig. 5.15 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (One bay, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

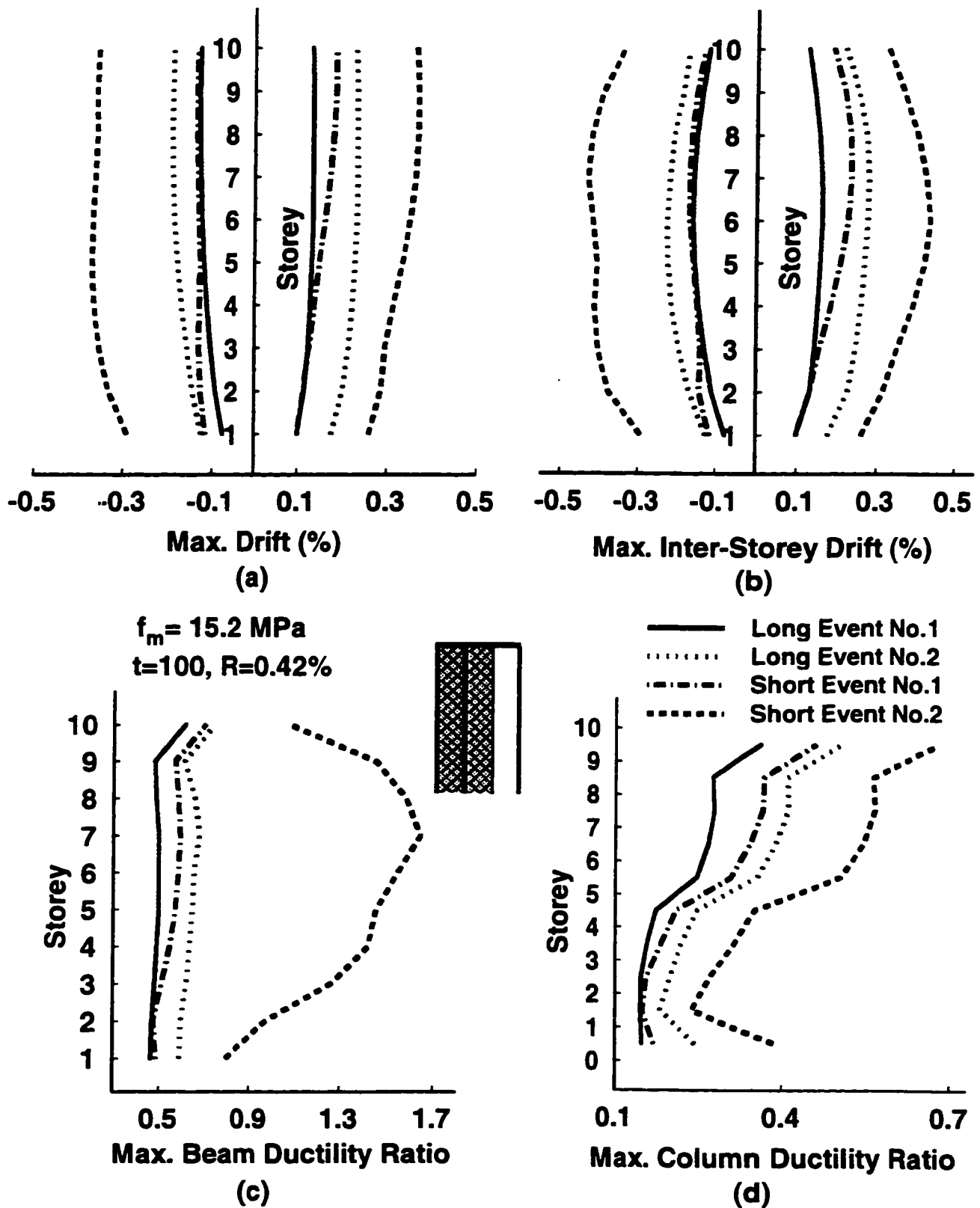


Fig. 5.16 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=100$ mm, $f_m=15.2$ MPa) .

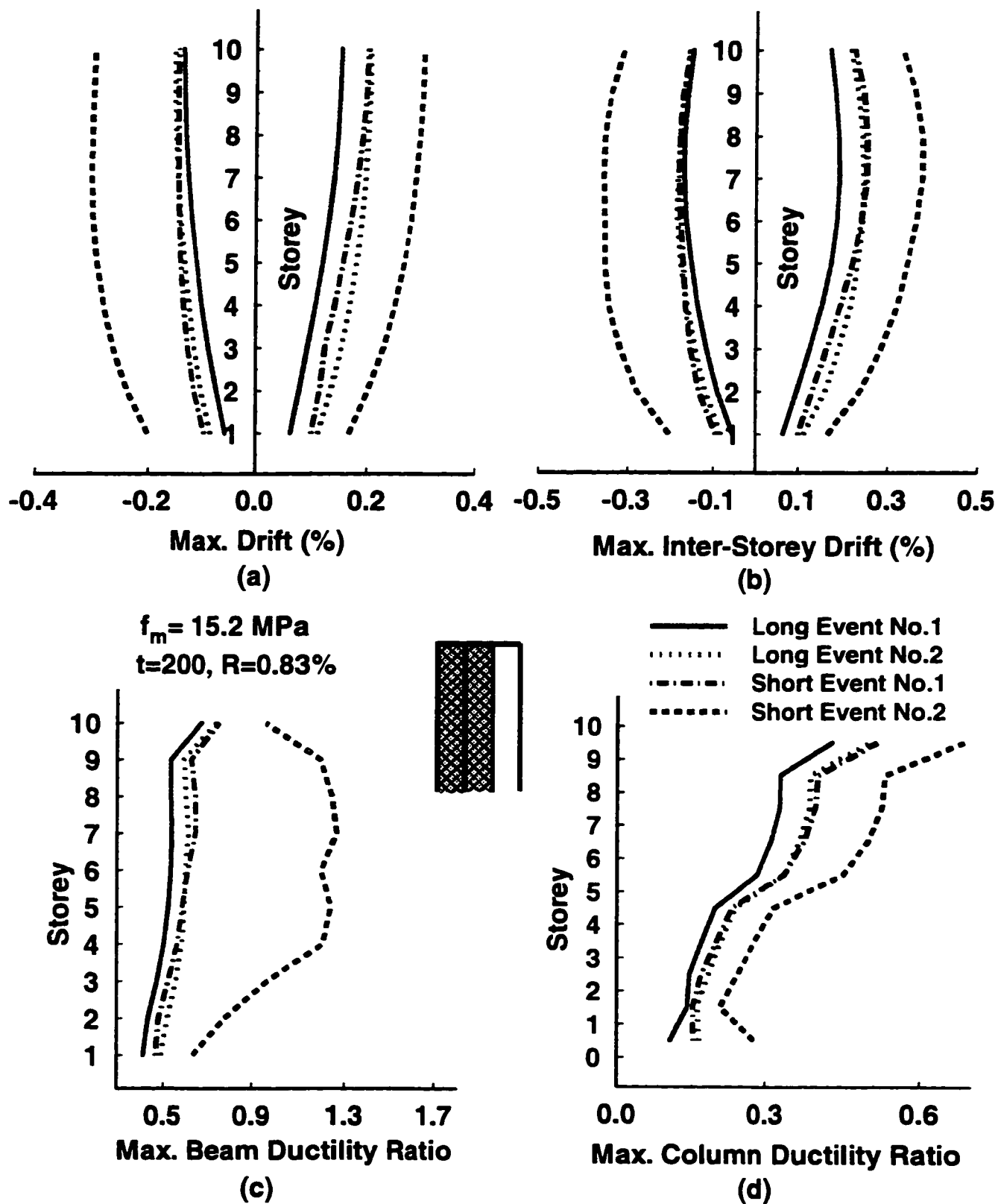


Fig. 5.17 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

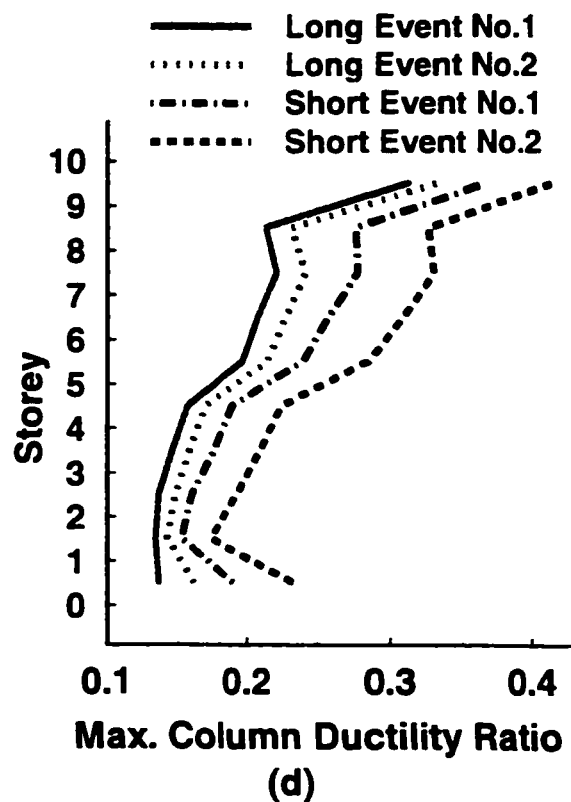
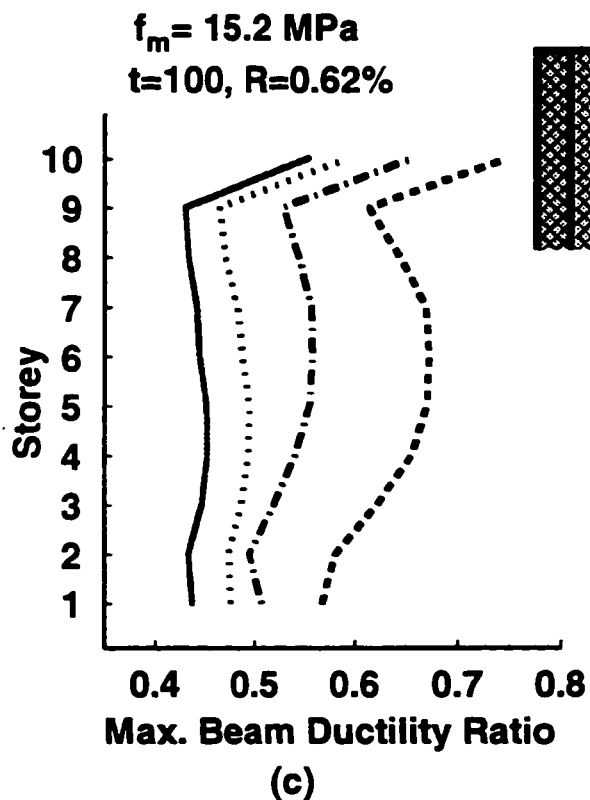
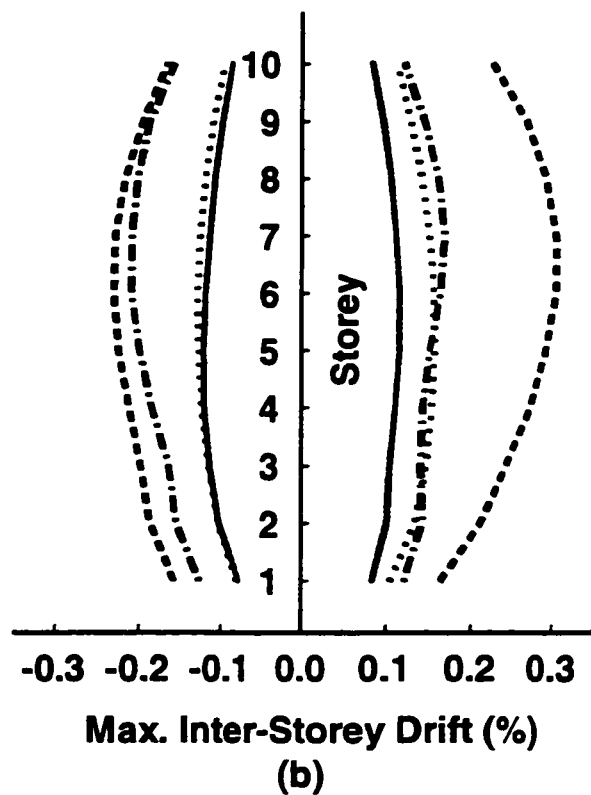
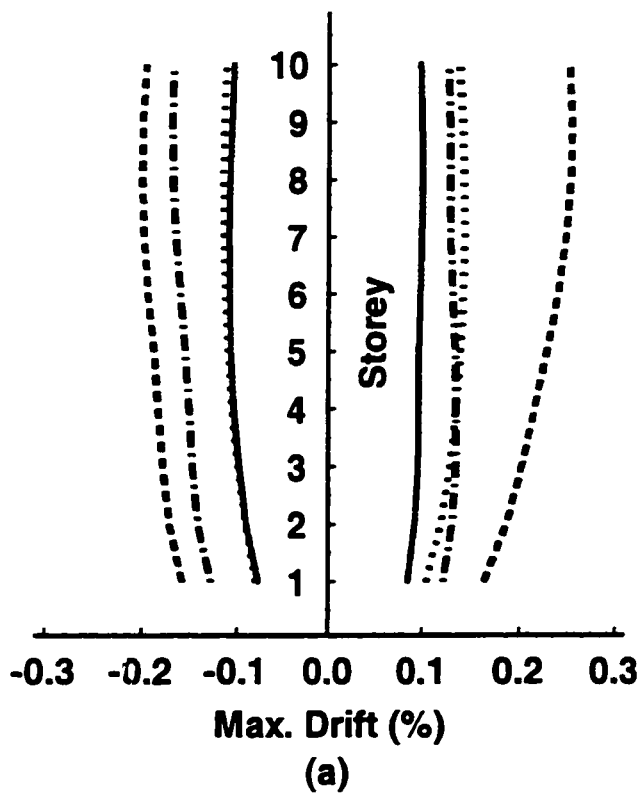


Fig. 5.18 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=100 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

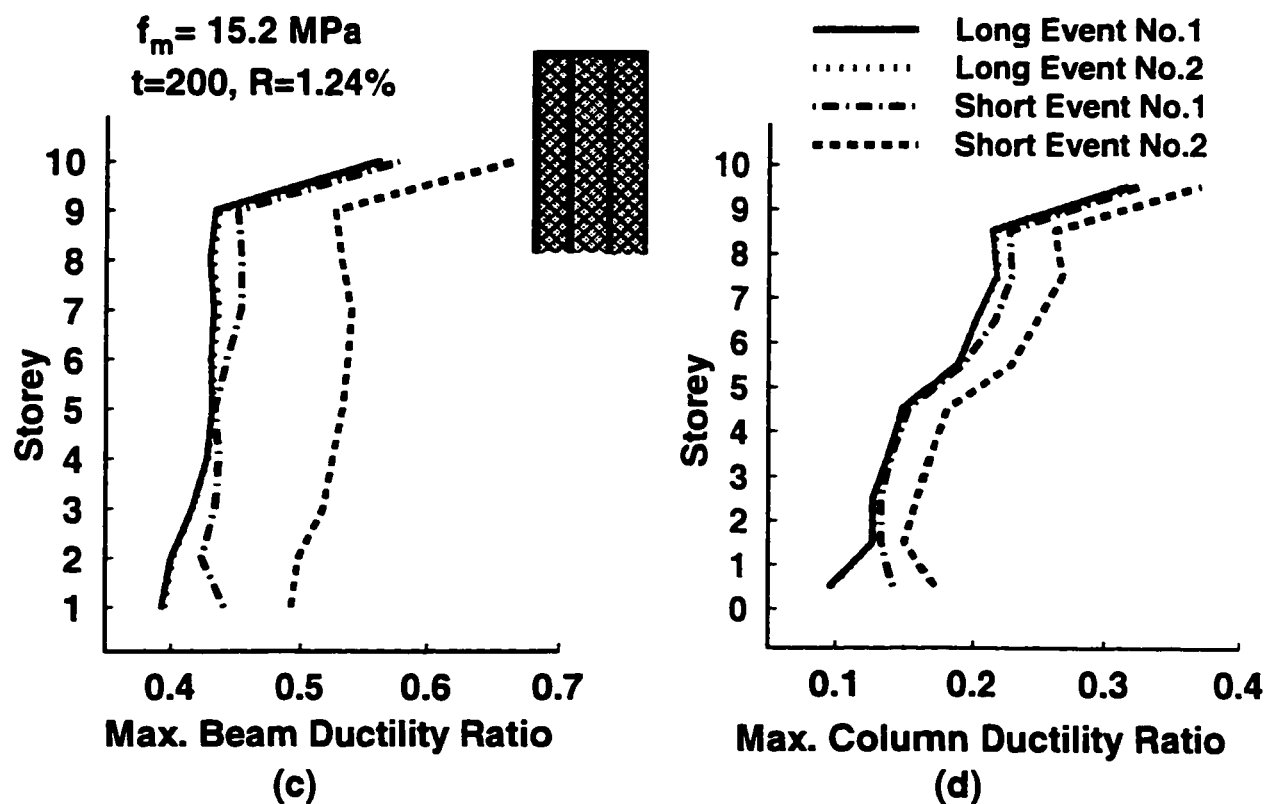
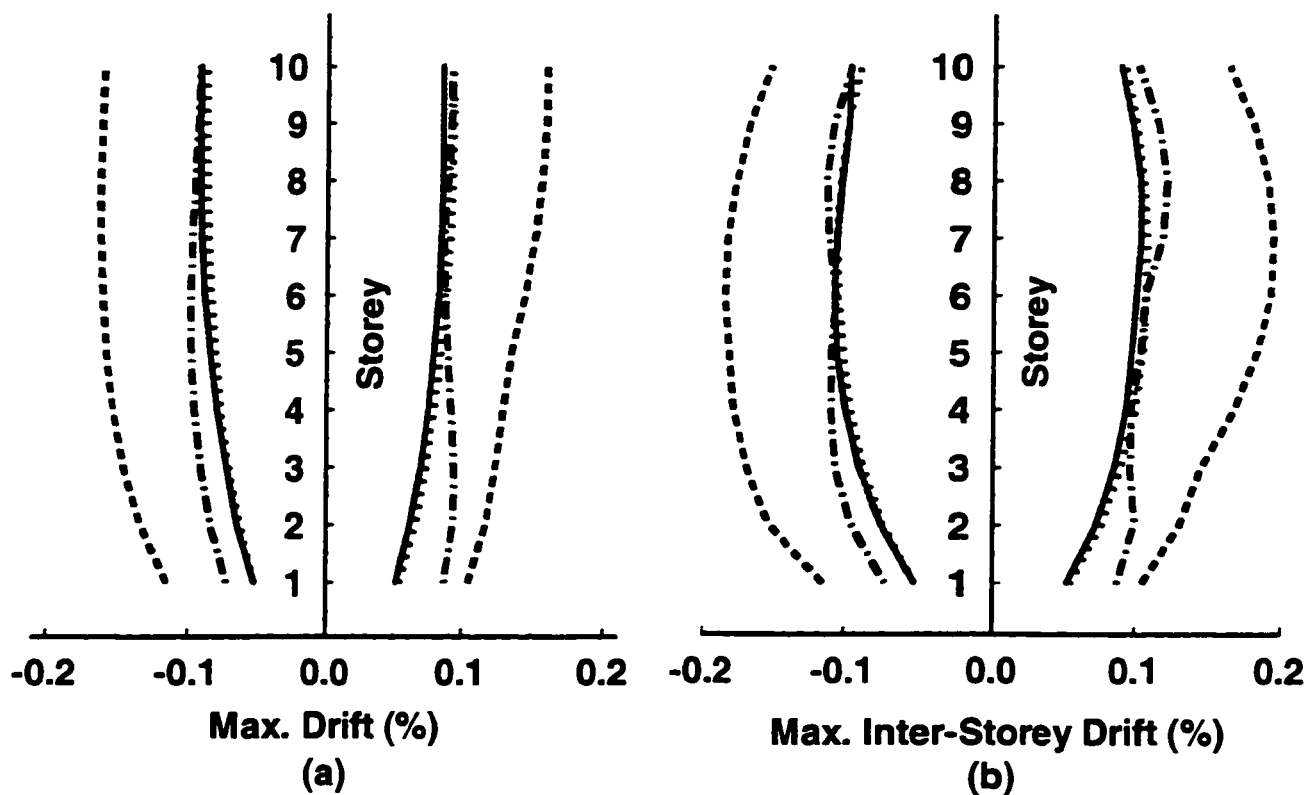


Fig. 5.19 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

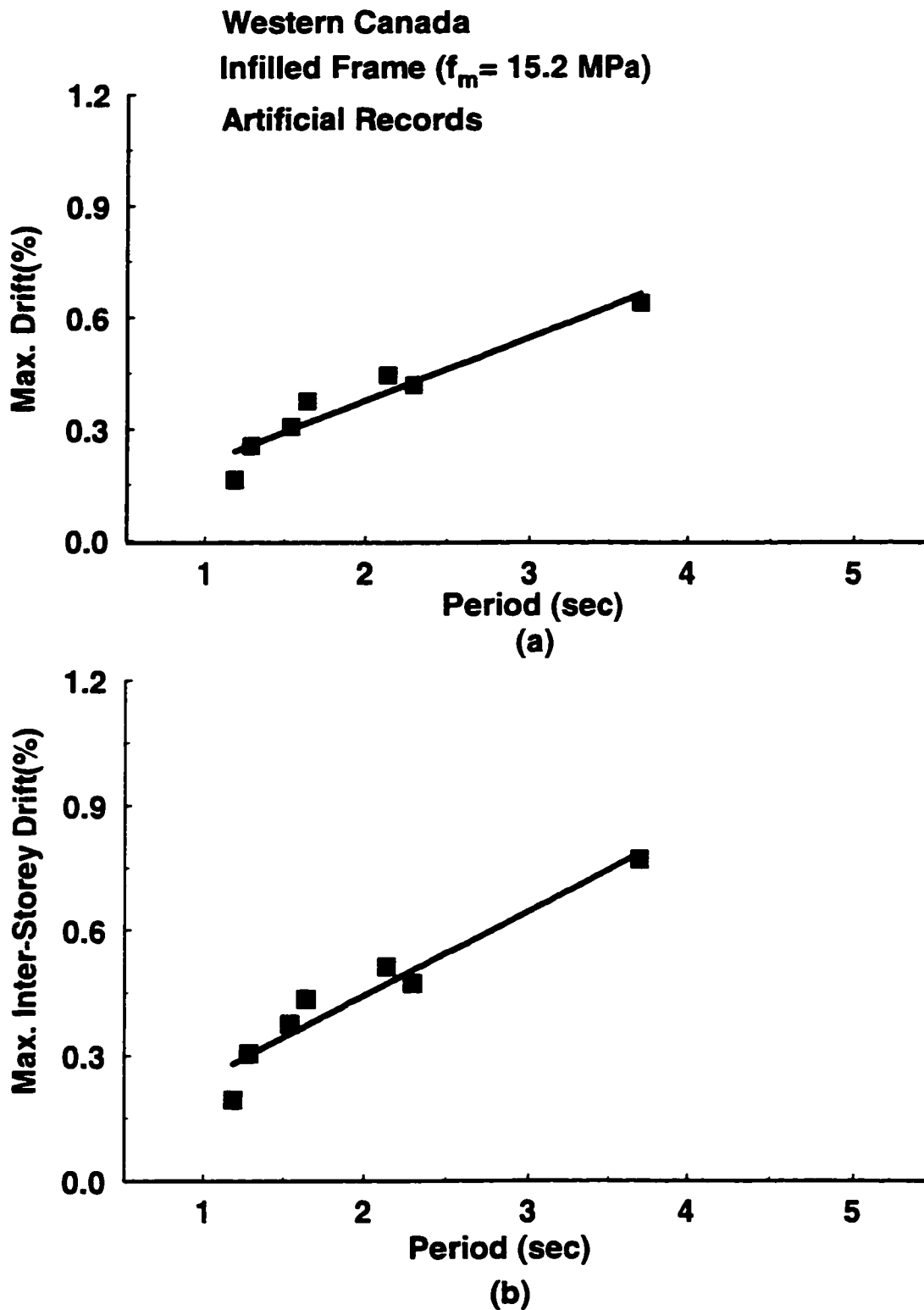
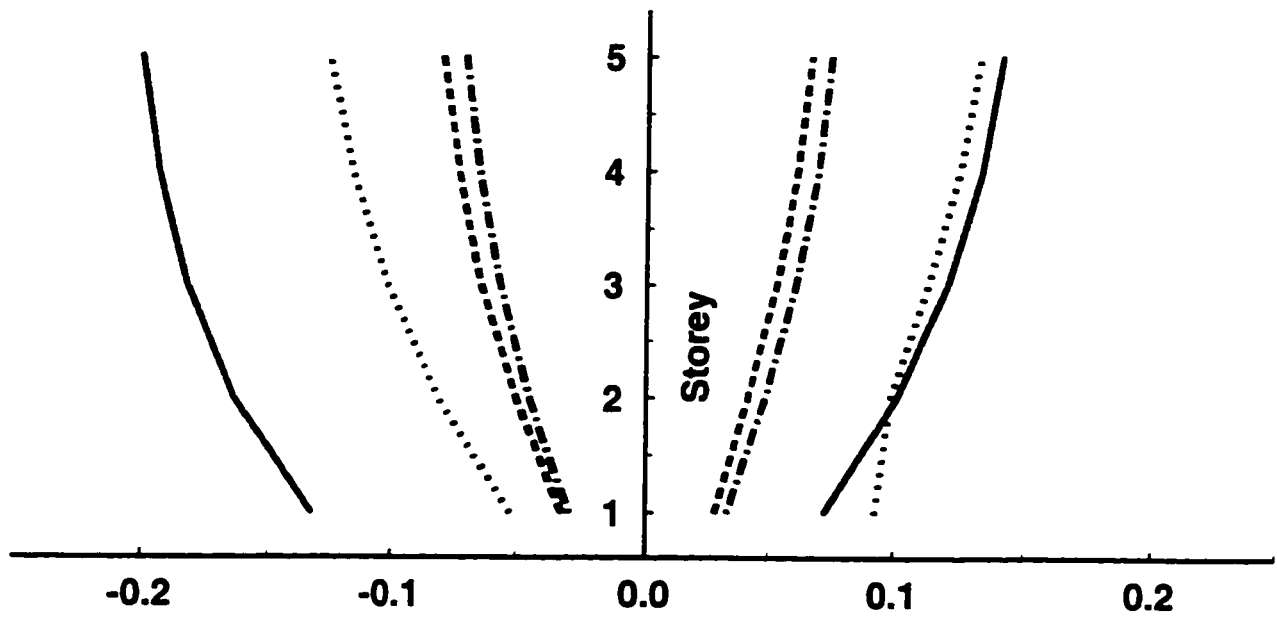


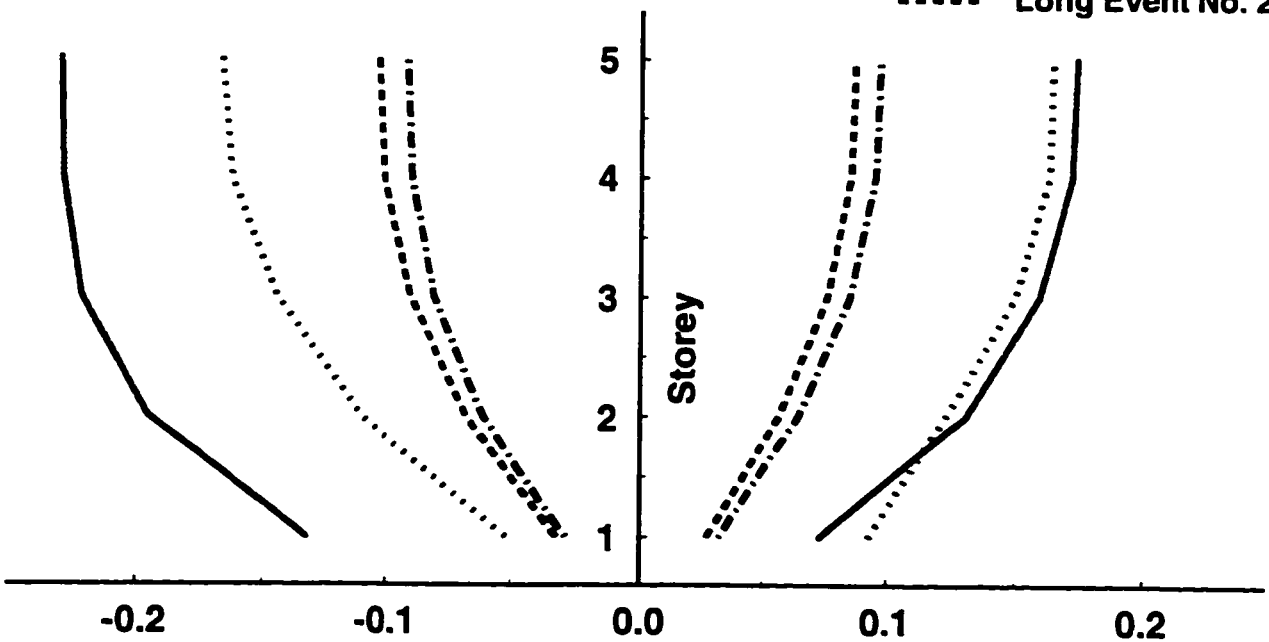
Fig. 5.20 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Artificial Records ($f_m=15.2$ MPa).



Maximum Drift (%)

(a)

- Short Event No. 1
- Short Event No. 2
- . - Long Event No. 1
- - - Long Event No. 2



Maximum Inter-Storey Drift (%)

(b)

Fig. 5.21 Drift Response for 5-Storey SHW Subjected to Artificial Records for Western Canada.

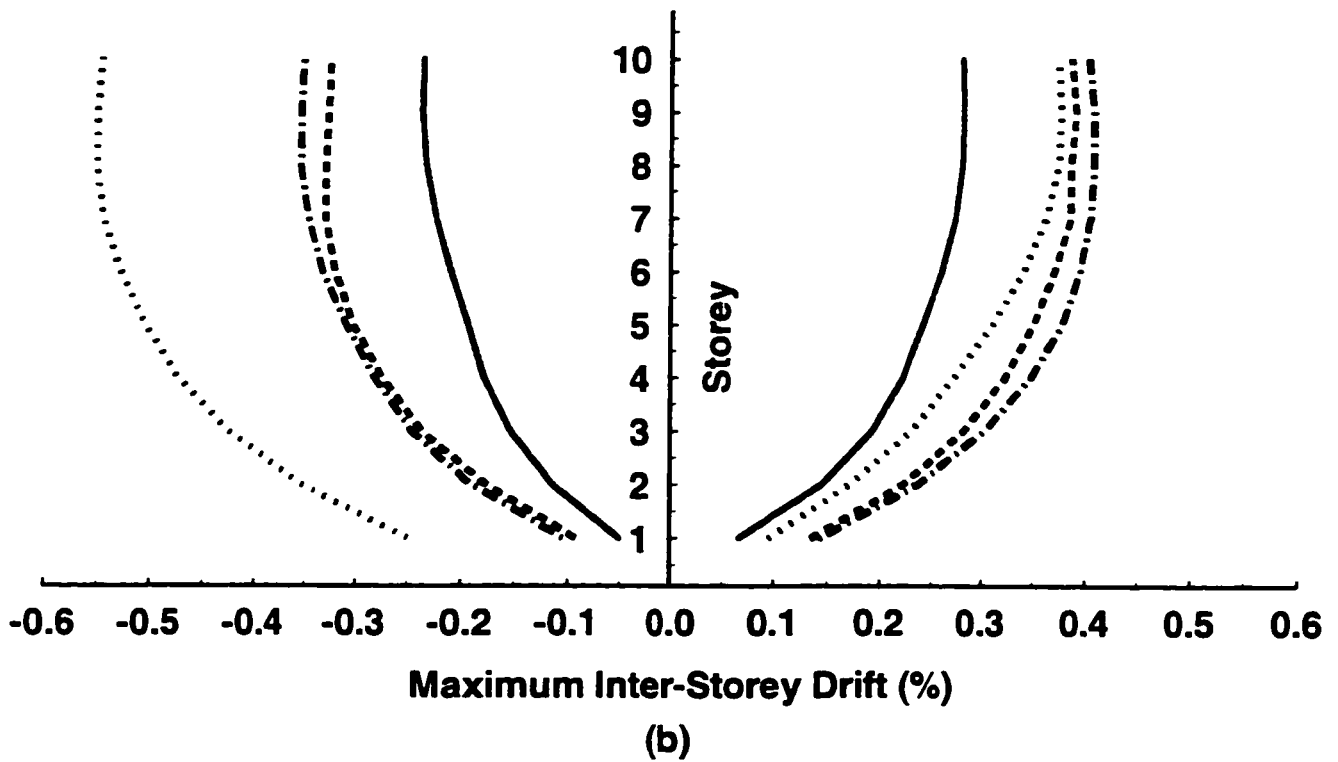
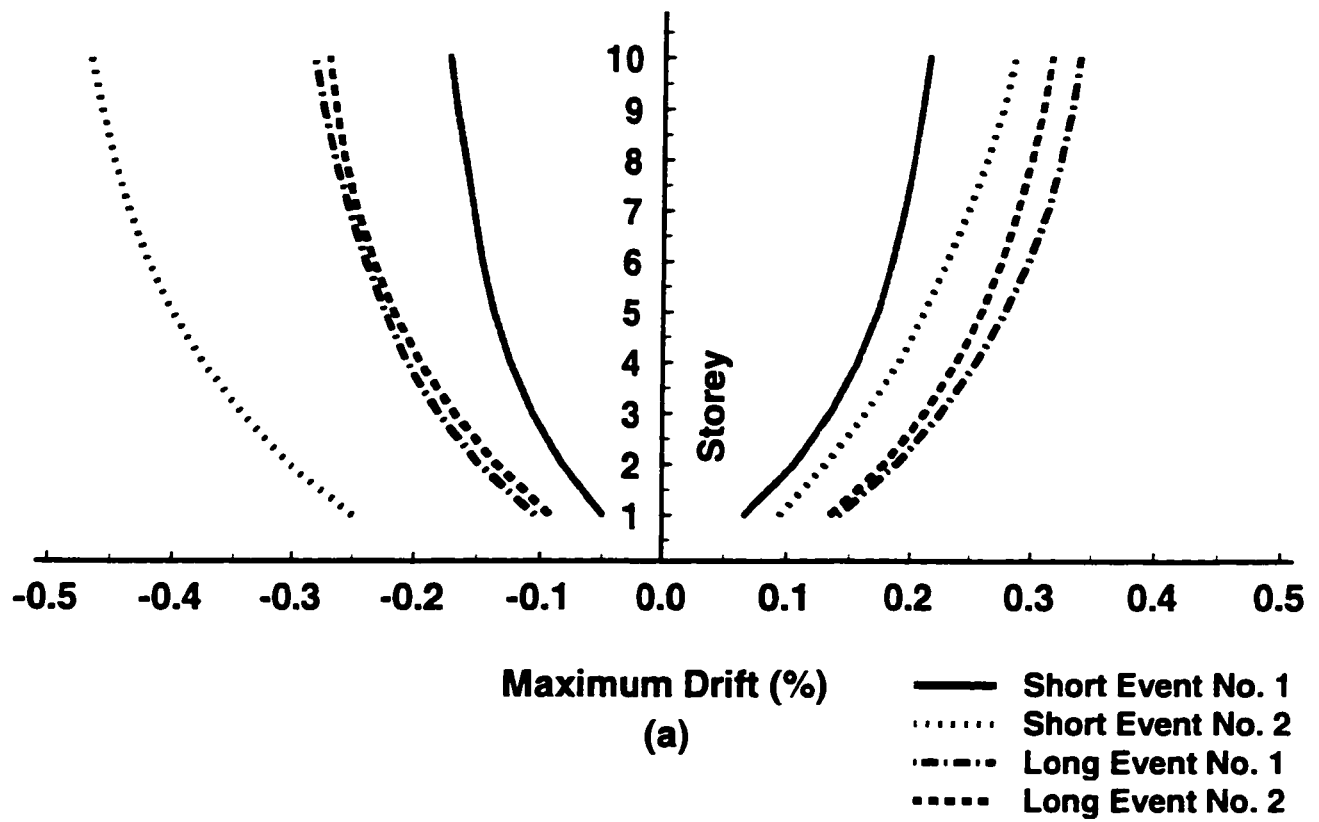


Fig. 5.22 Drift Response for 10-Storey SHW Subjected to Artificial Records for Western Canada.

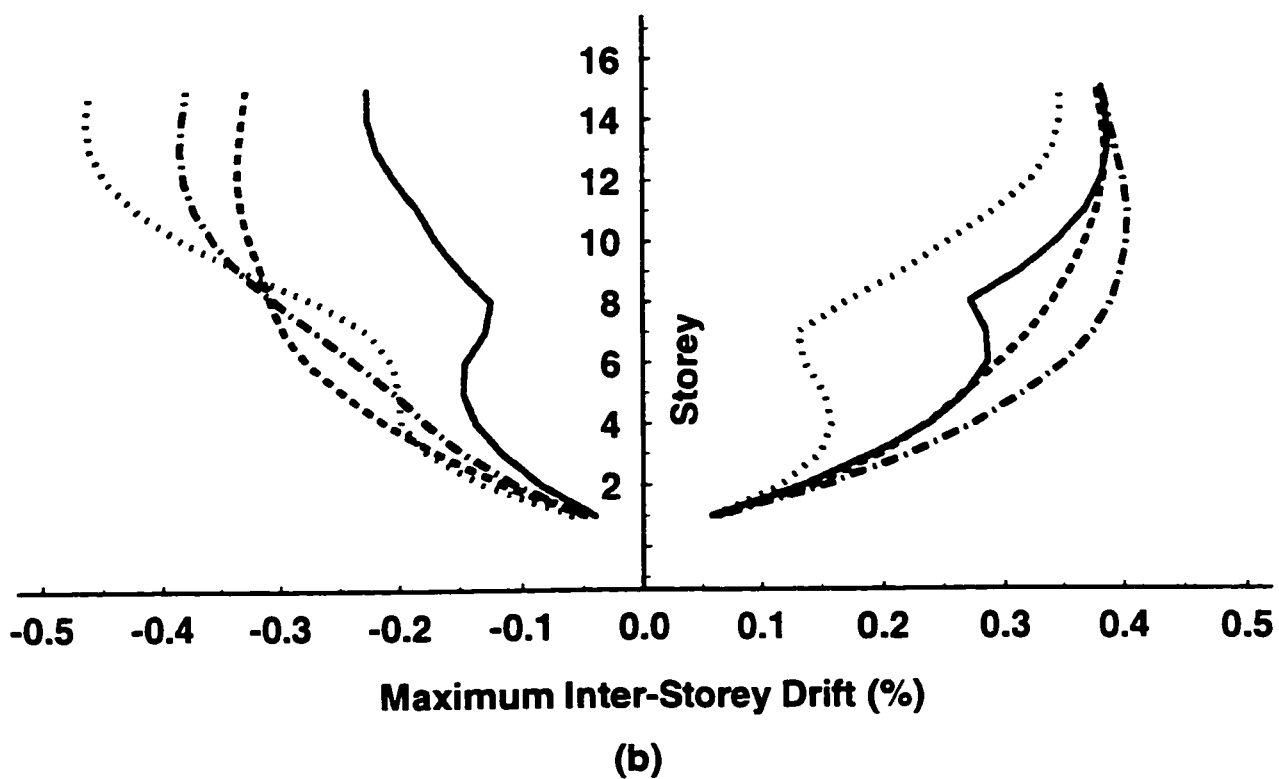
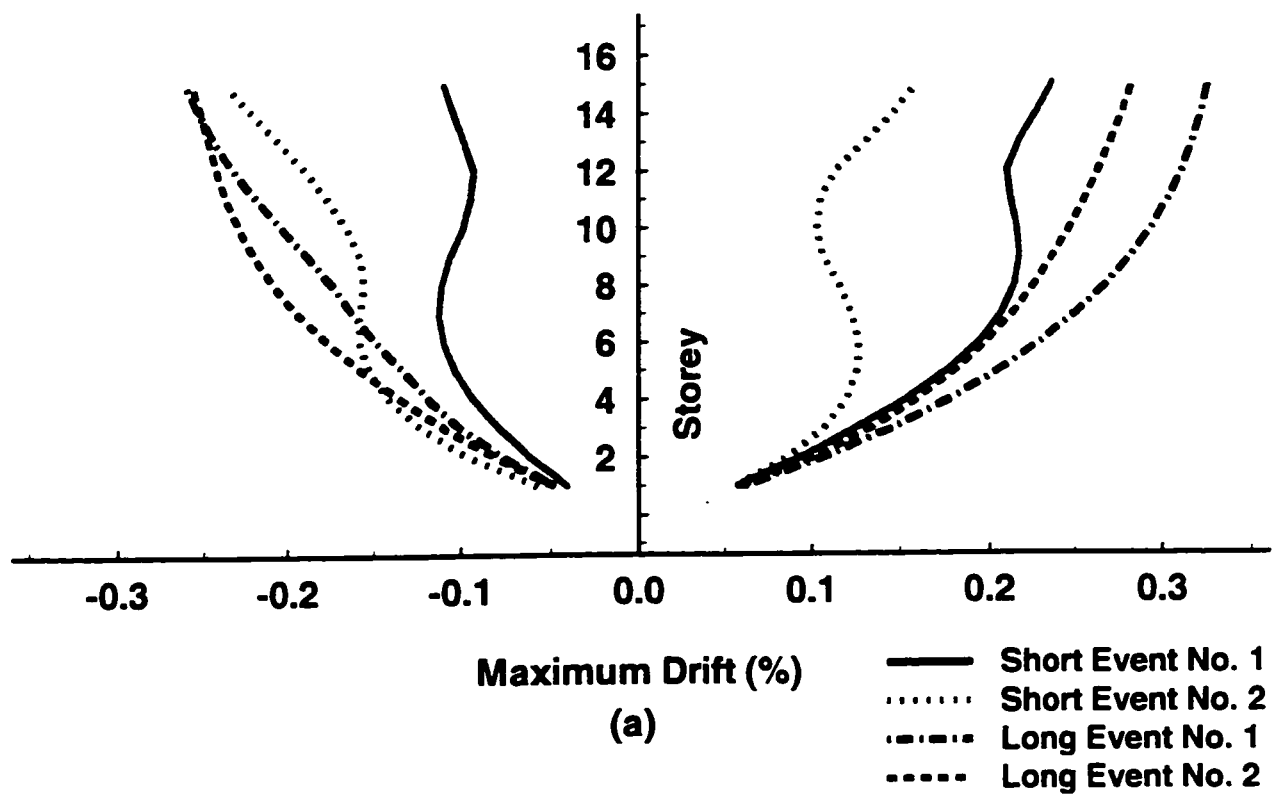
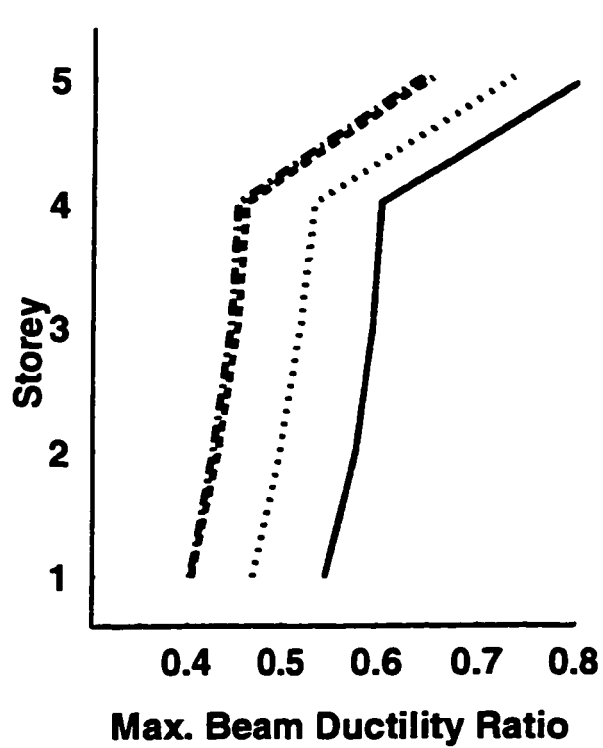
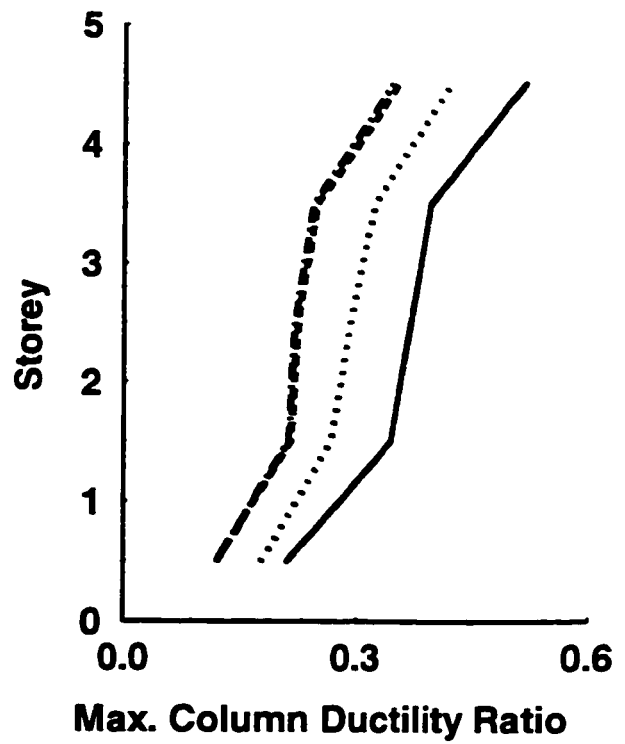


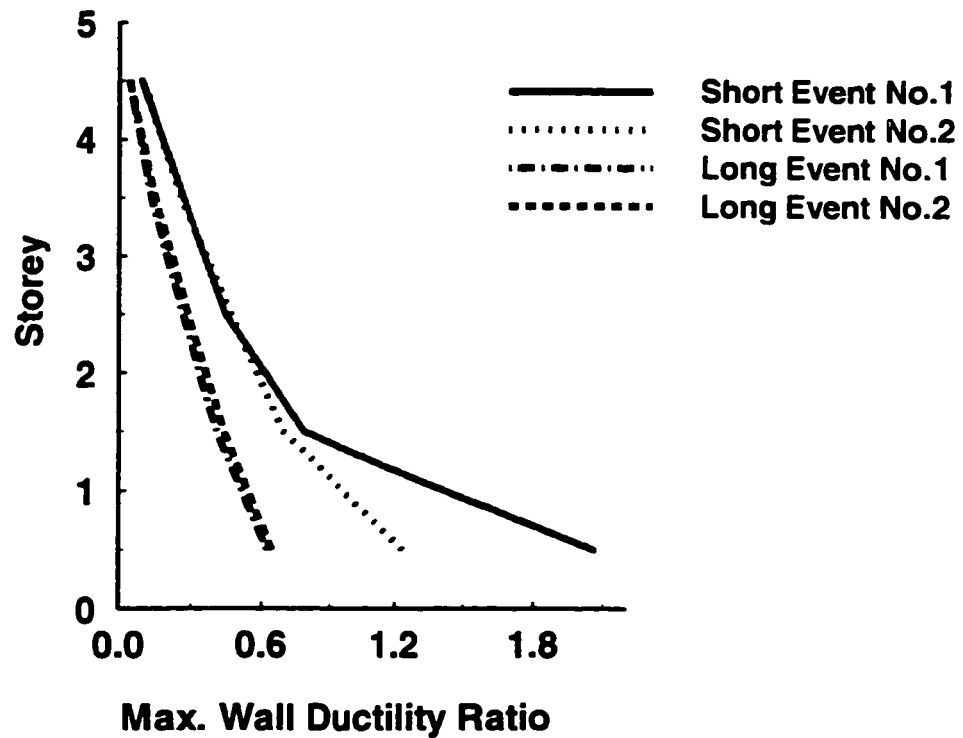
Fig. 5.23 Drift Response for 15-Storey SHW Subjected to Artificial Records for Western Canada.



(a)

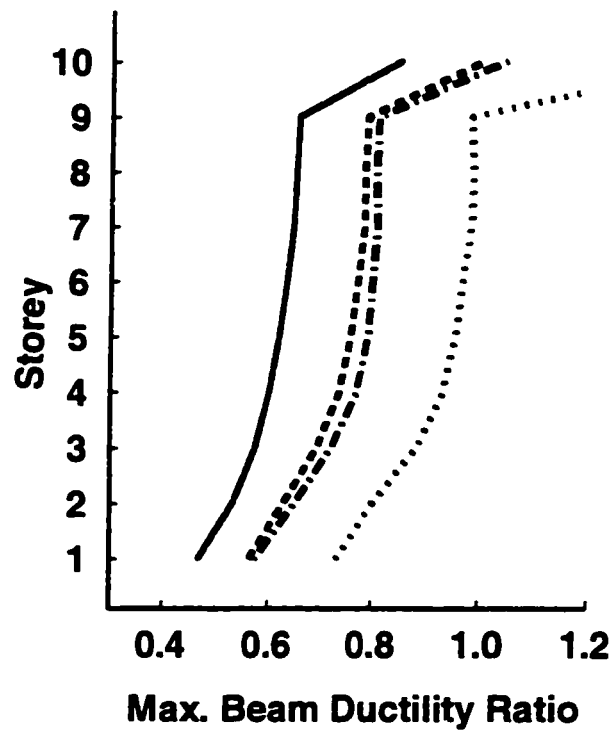


(b)

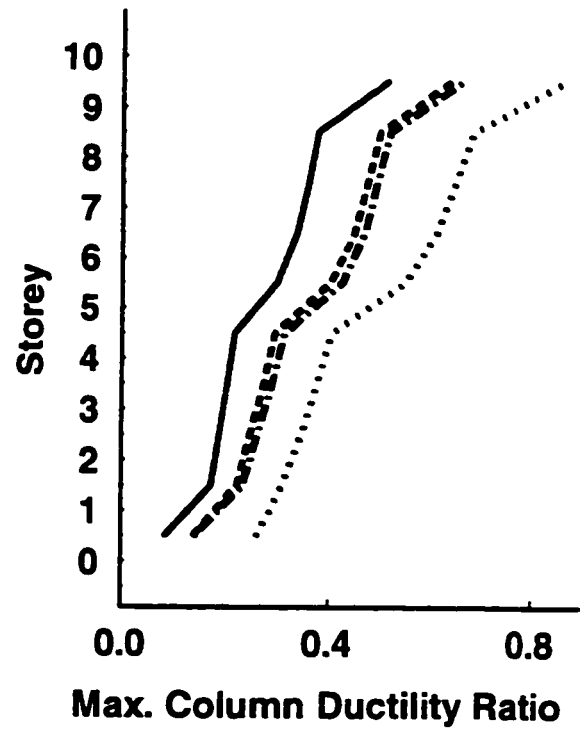


(c)

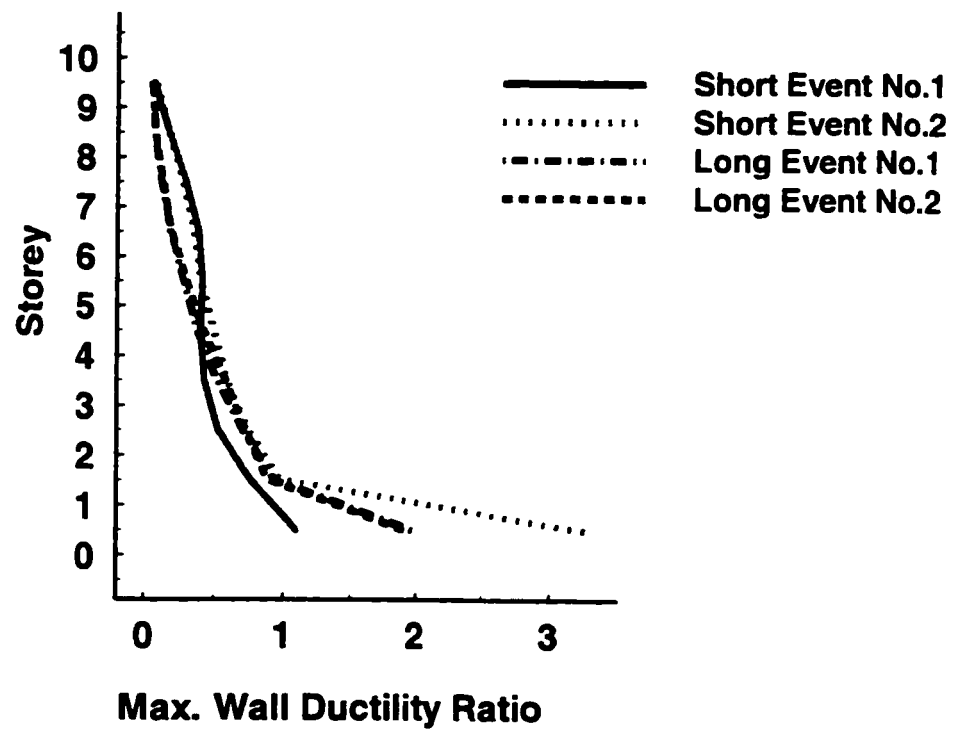
Fig. 5.24 Ductility Response for 5-Storey SHW Subjected to Artificial Records for Western Canada.



(a)



(b)



(c)

Fig. 5.25 Ductility Response for 10-Storey SHW Subjected to Artificial Records for Western Canada.

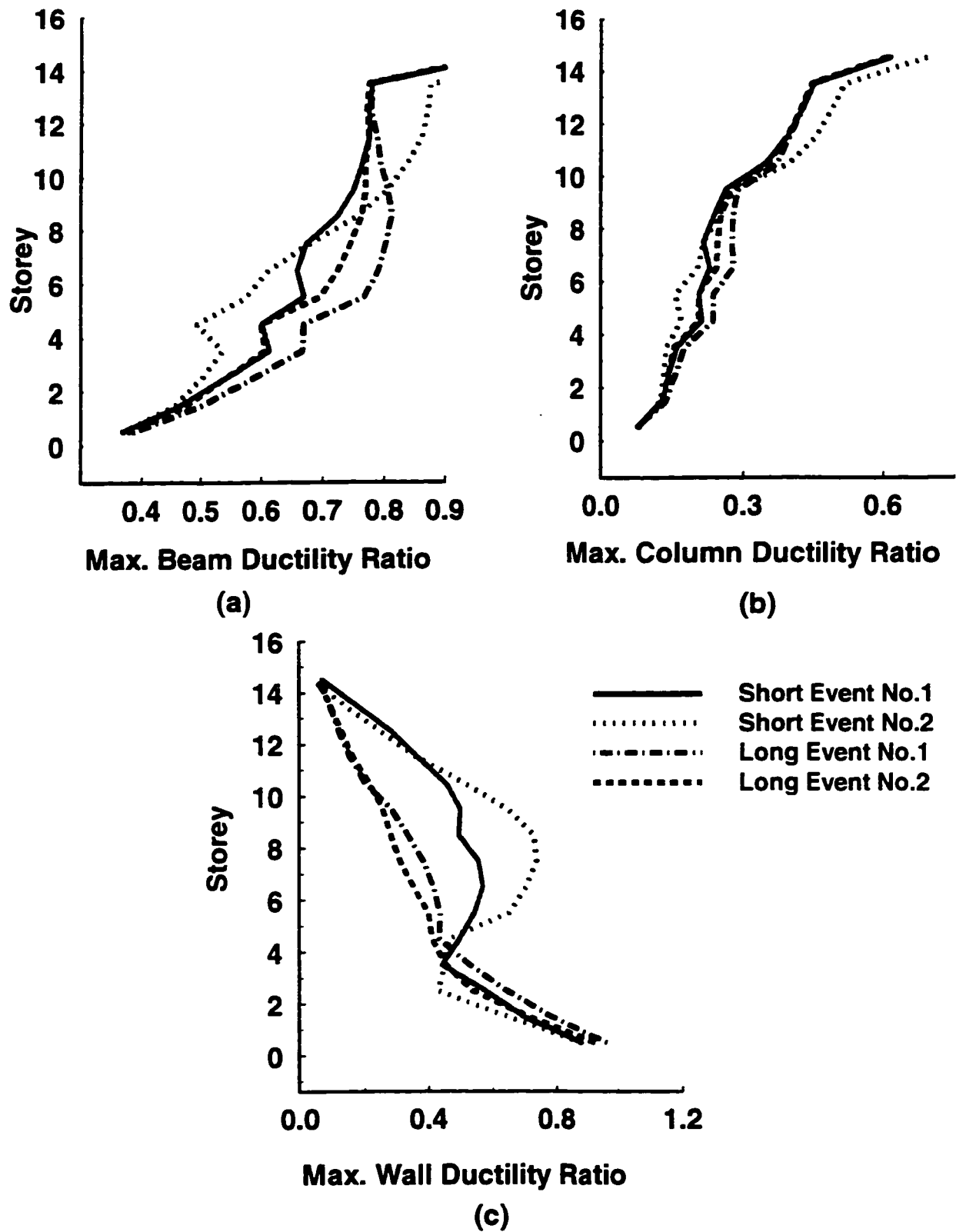


Fig. 5.26 Ductility Response for 15-Storey SHW Subjected to Artificial Records for Western Canada.

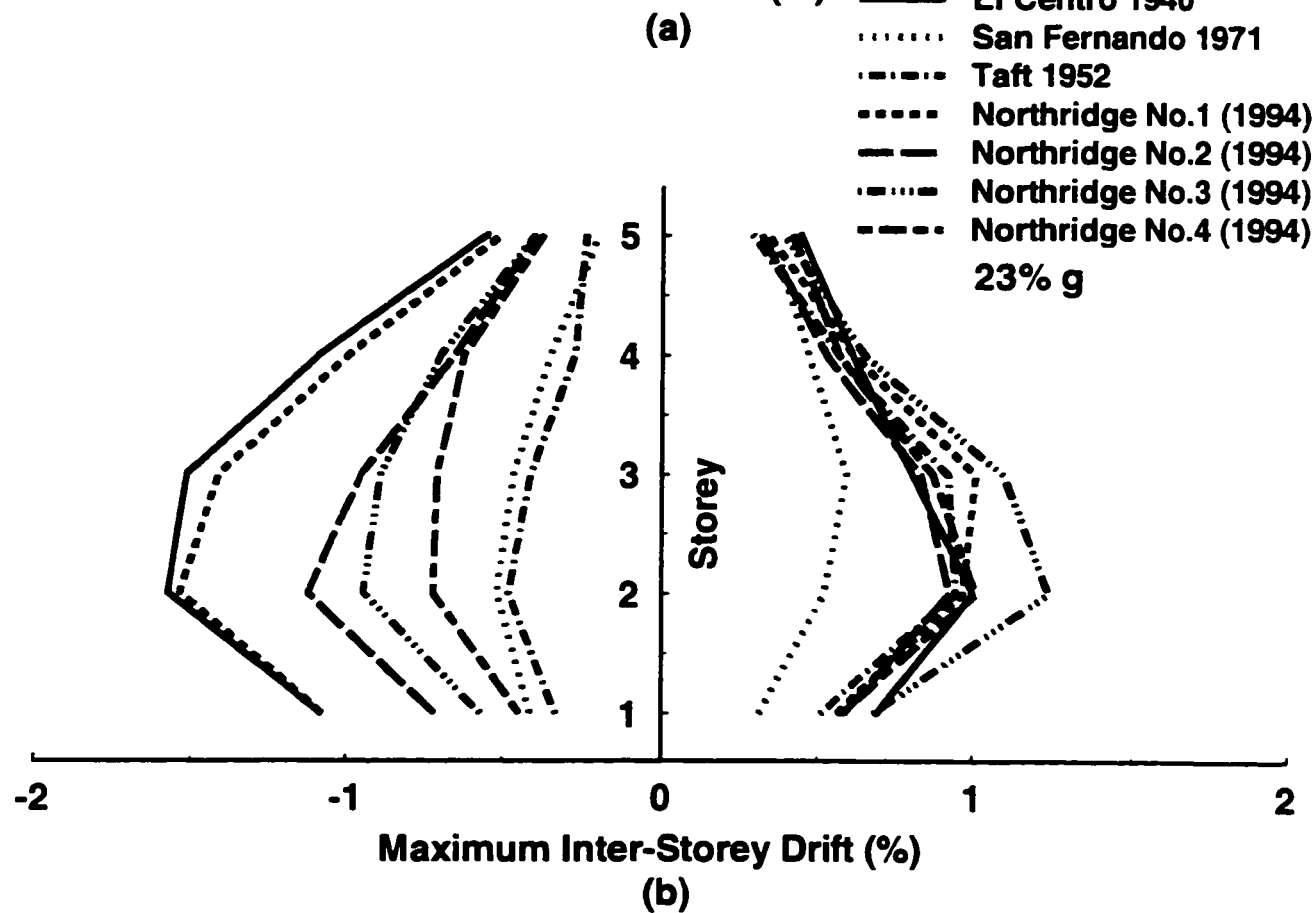
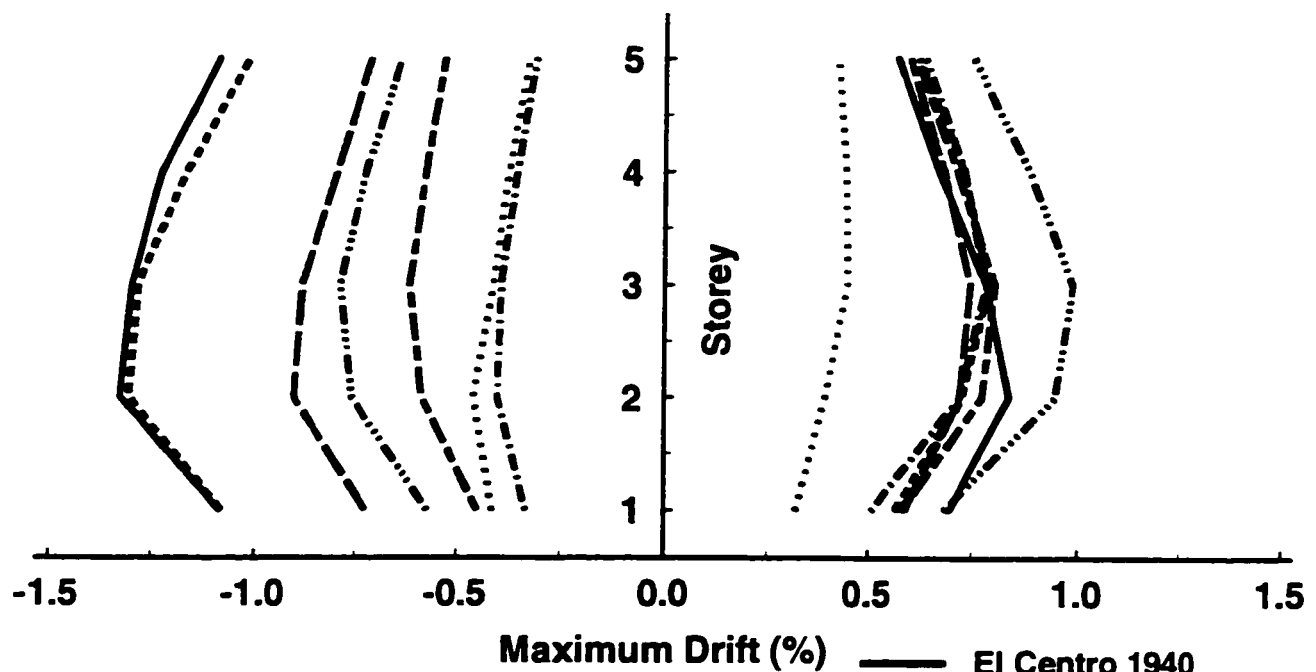


Fig. 5.27 Drift Response for 5-Storey MRF Subjected to
All Selected Records for Western Canada.

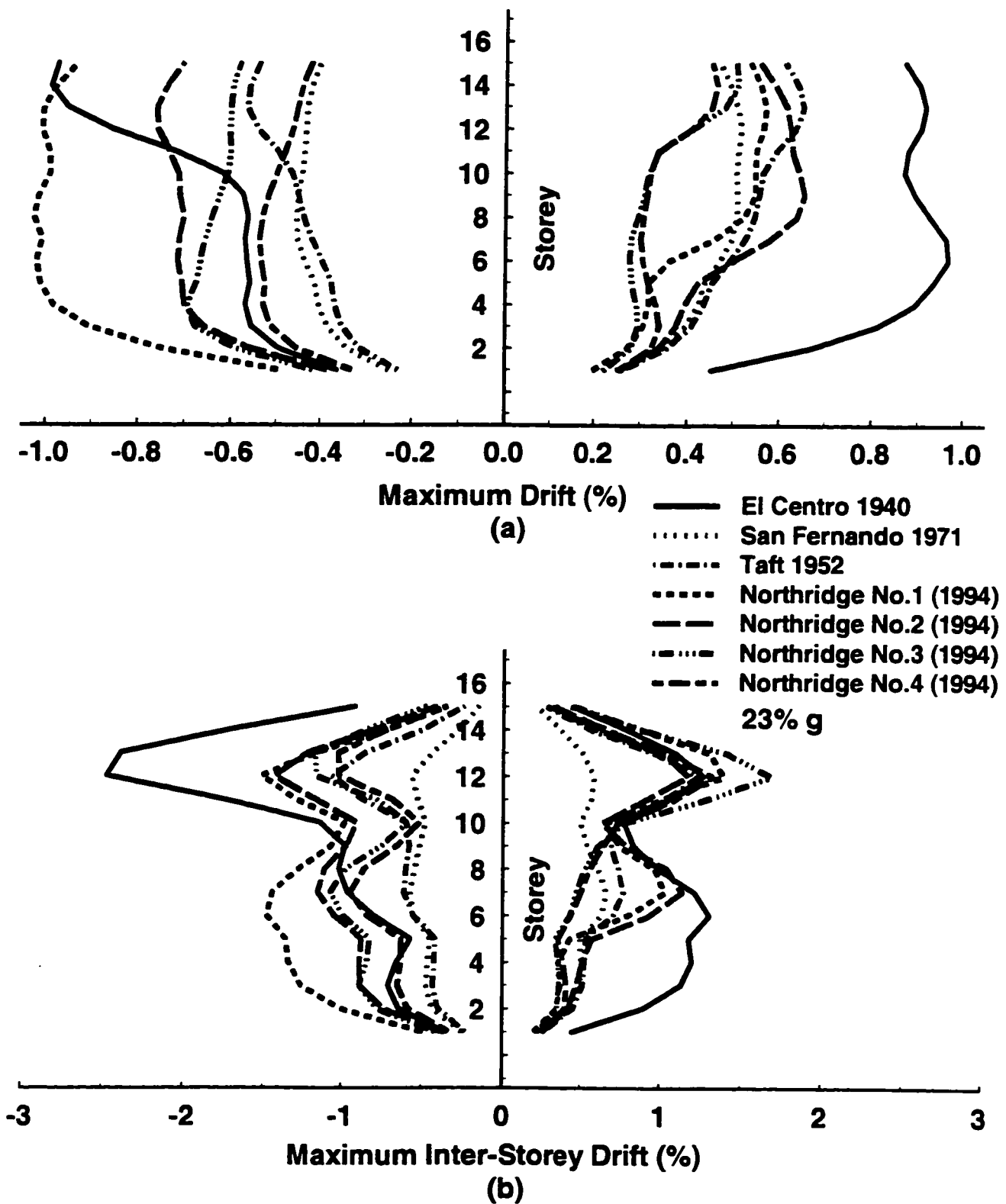
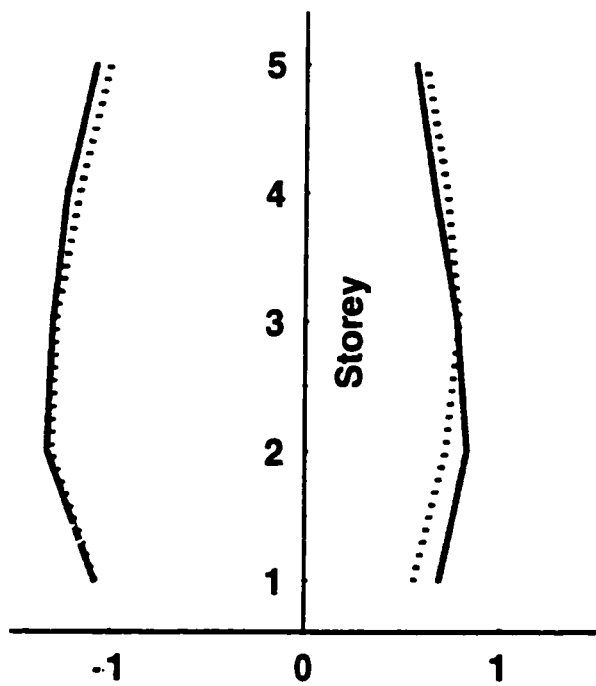
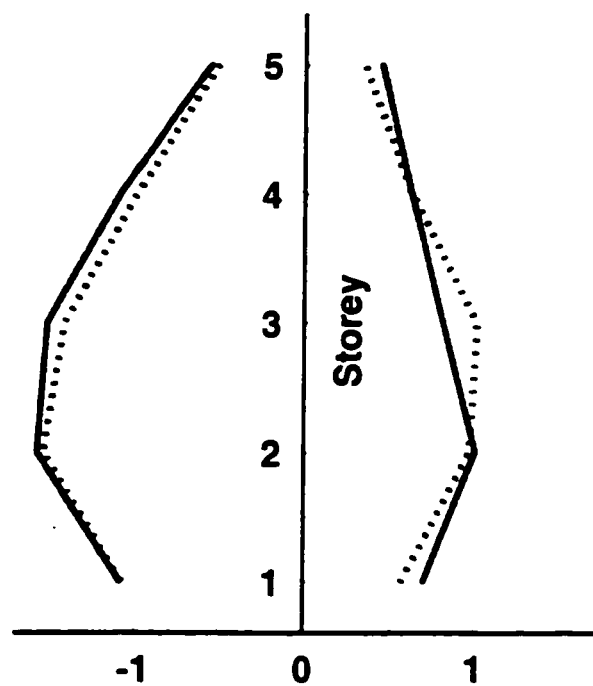


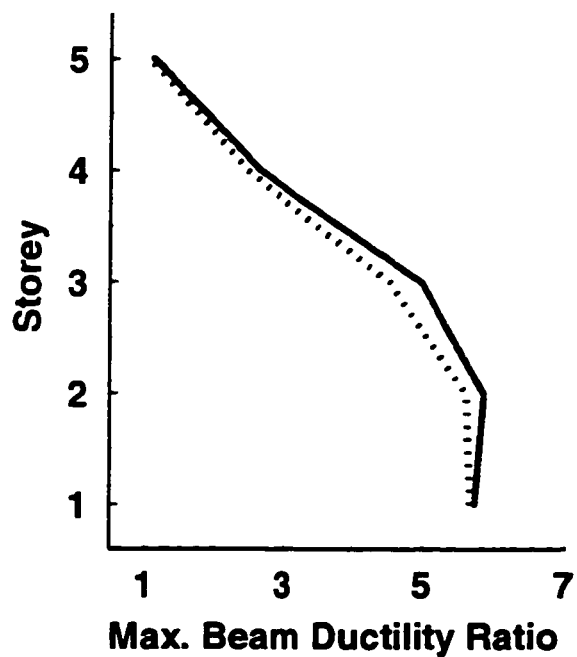
Fig. 5.28 Drift Response for 15-Storey MRF Subjected to All Selected Records for Western Canada.



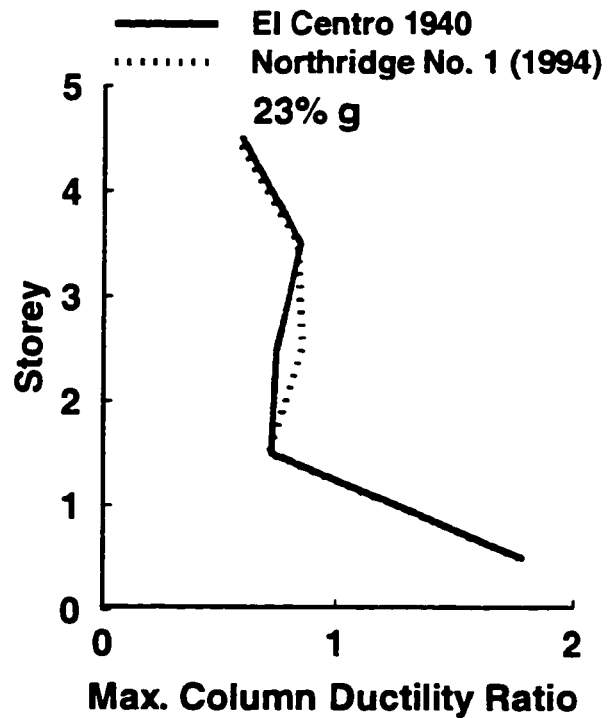
Max. Drift (%)
(a)



Max. Inter-Storey Drift (%)
(b)

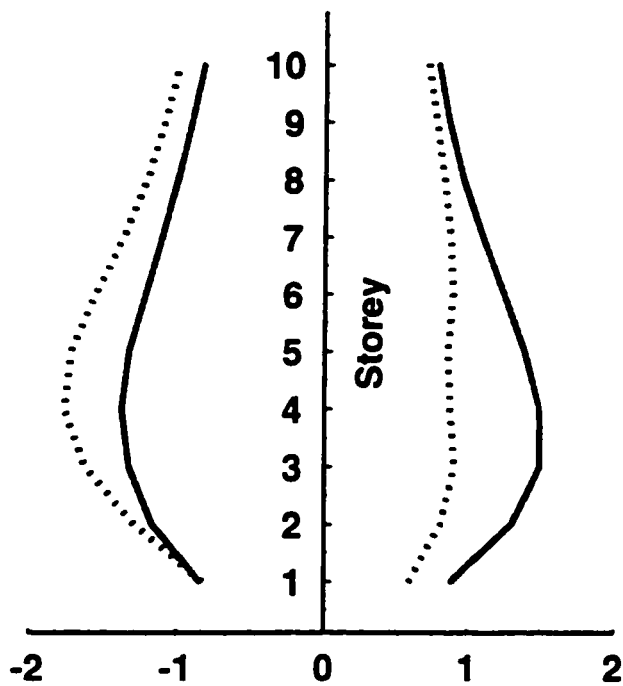


(c)



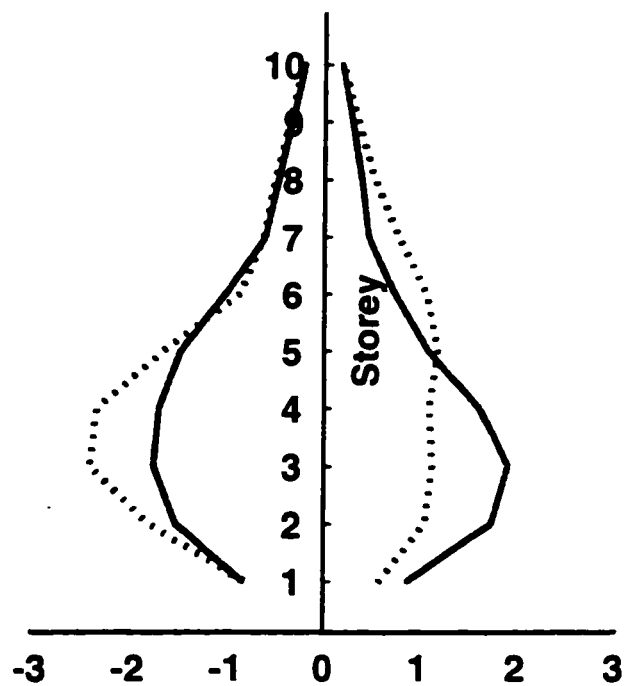
(d)

Fig. 5.29 Drift and Ductility Response for 5-Storey MRF Subjected to Actual Records for Western Canada.



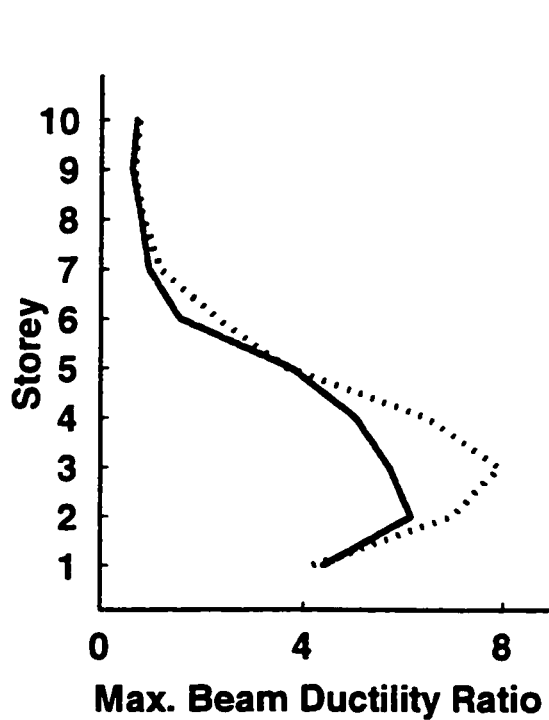
Max. Drift (%)

(a)



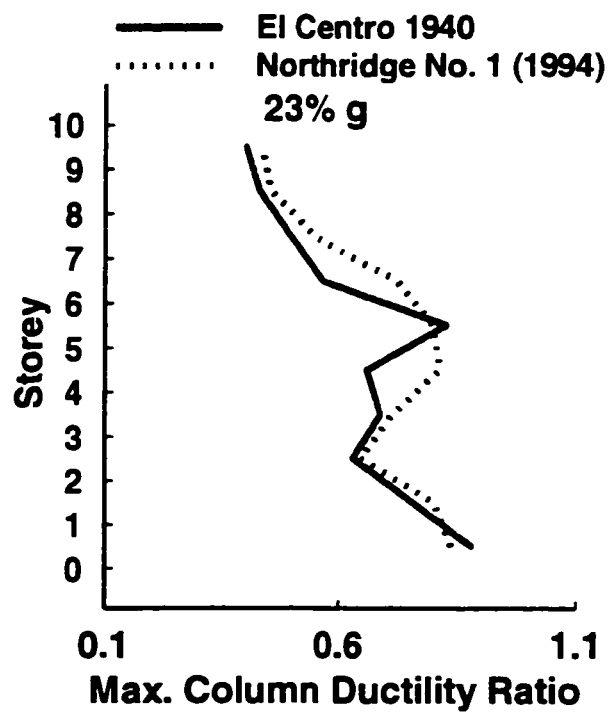
Max. Inter-Storey Drift (%)

(b)



Max. Beam Ductility Ratio

(c)



Max. Column Ductility Ratio

(d)

Fig. 5.30 Drift and Ductility Response for 10-Storey MRF Subjected to Actual Records for Western Canada.

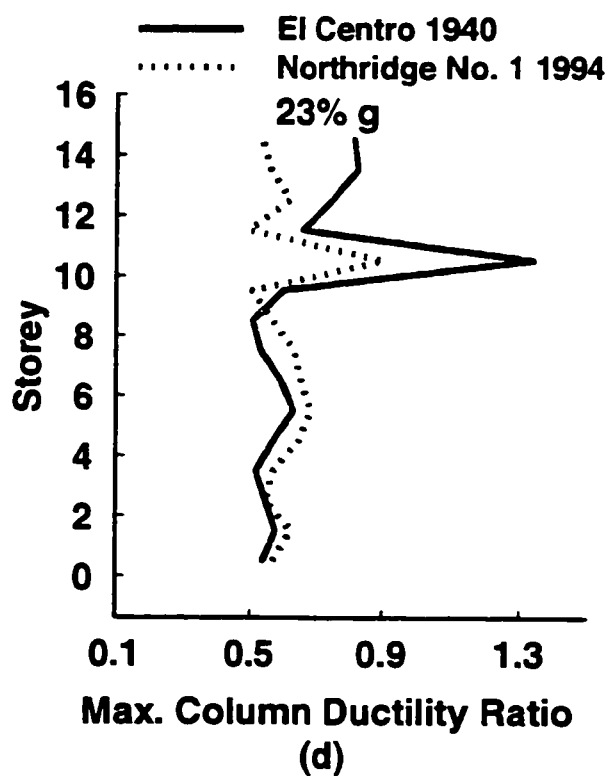
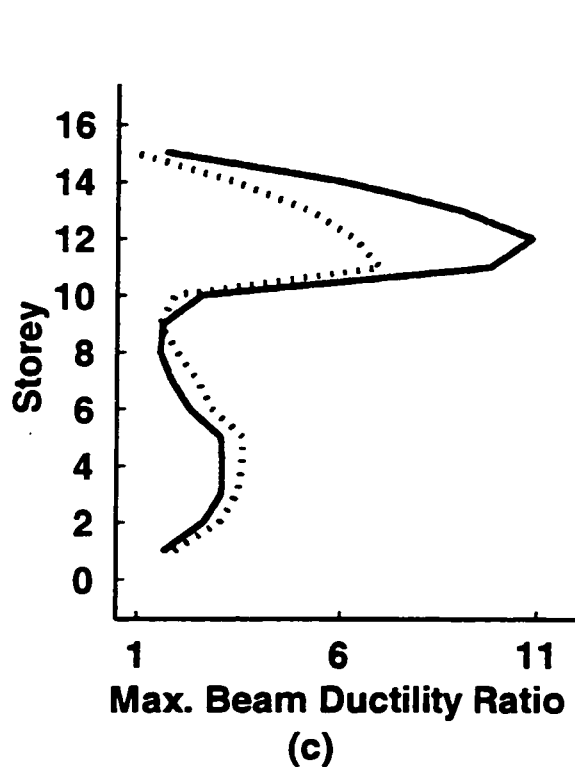
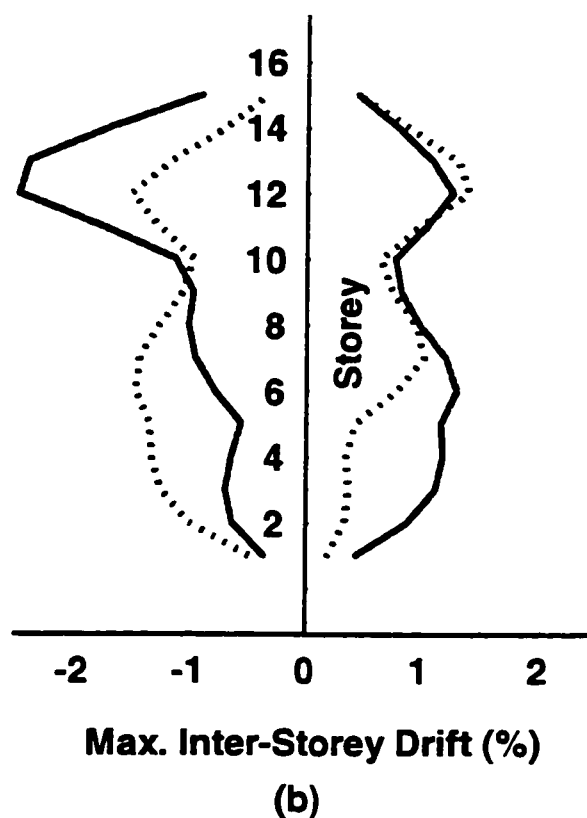
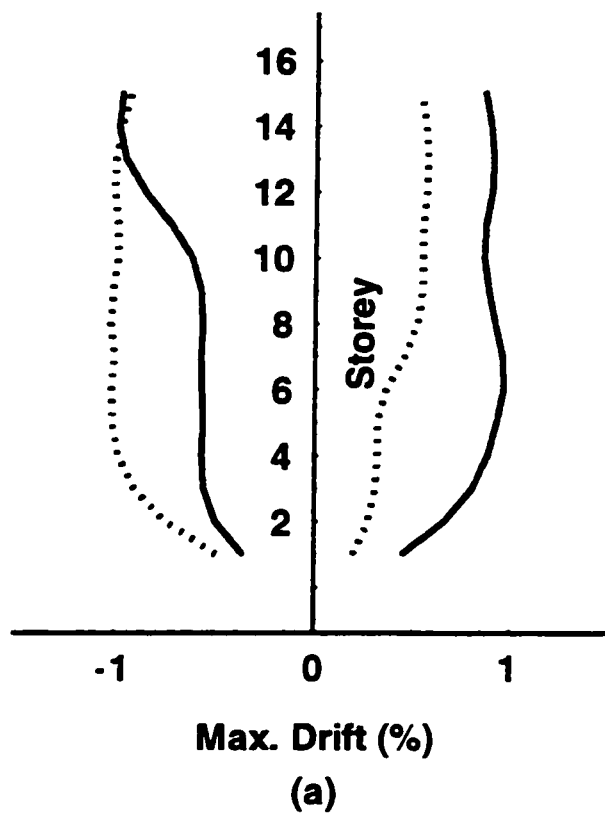


Fig. 5.31 Drift and Ductility Response for 15-Storey MRF Subjected to Actual Records for Western Canada.

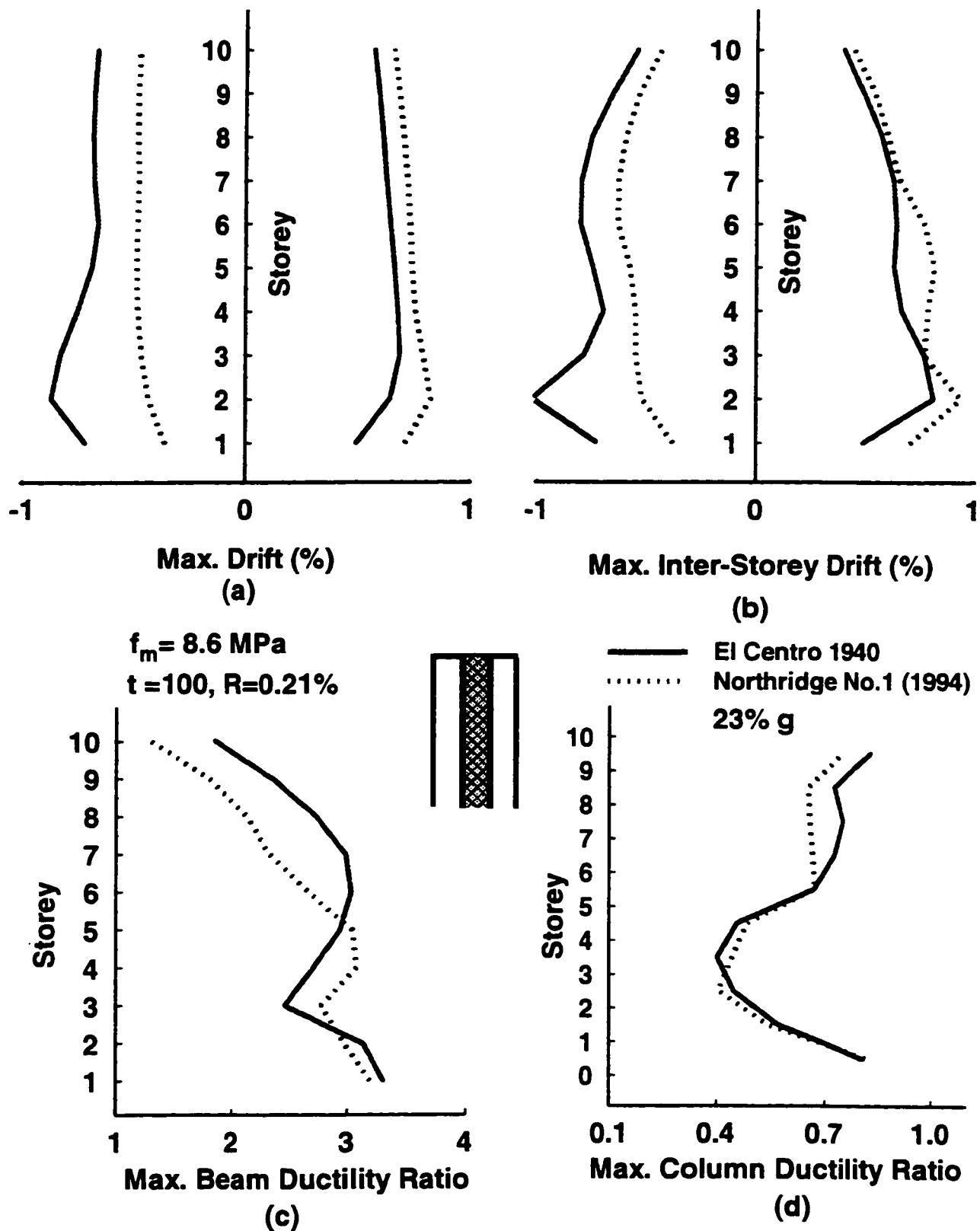


Fig. 5.32 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

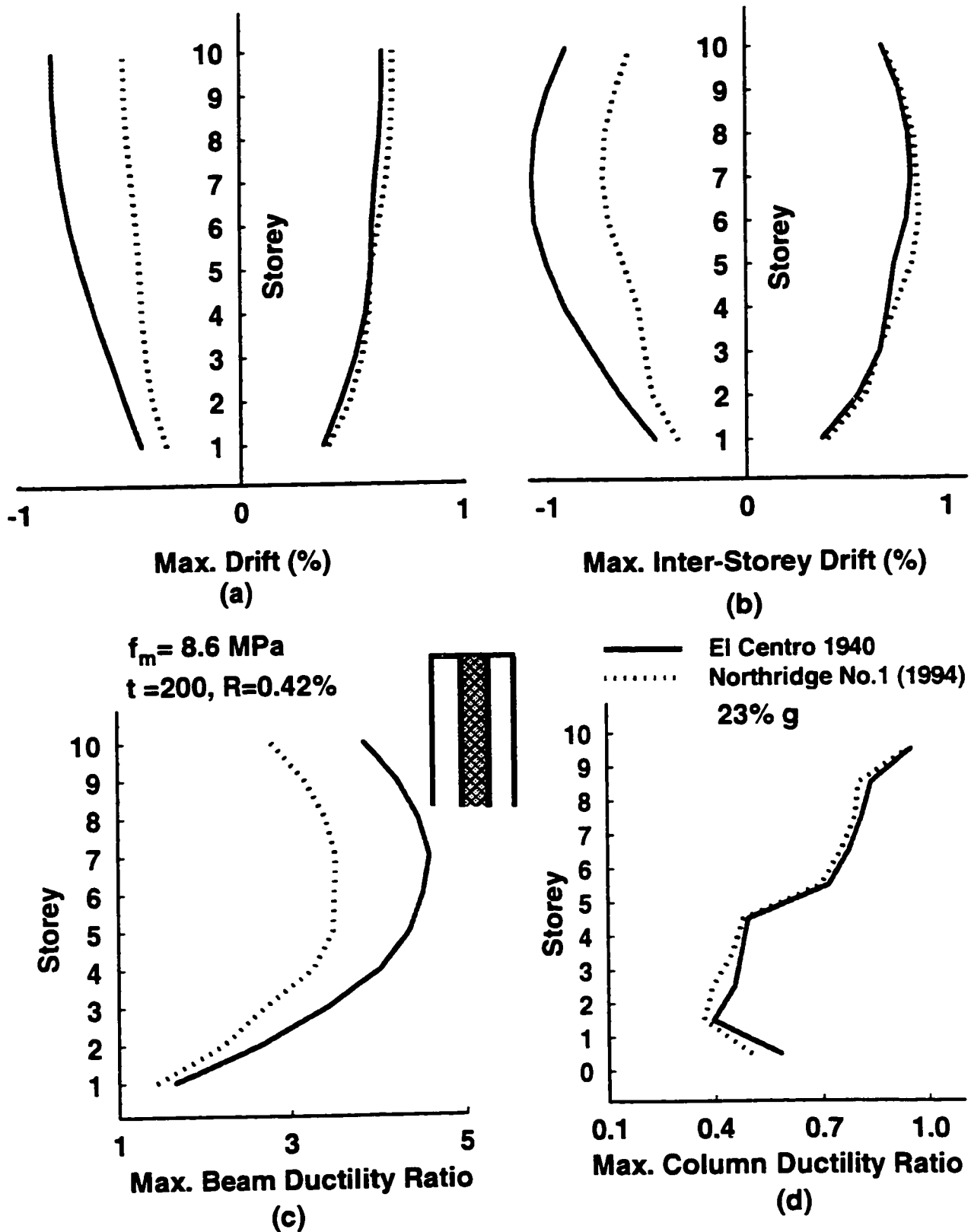


Fig. 5.33 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

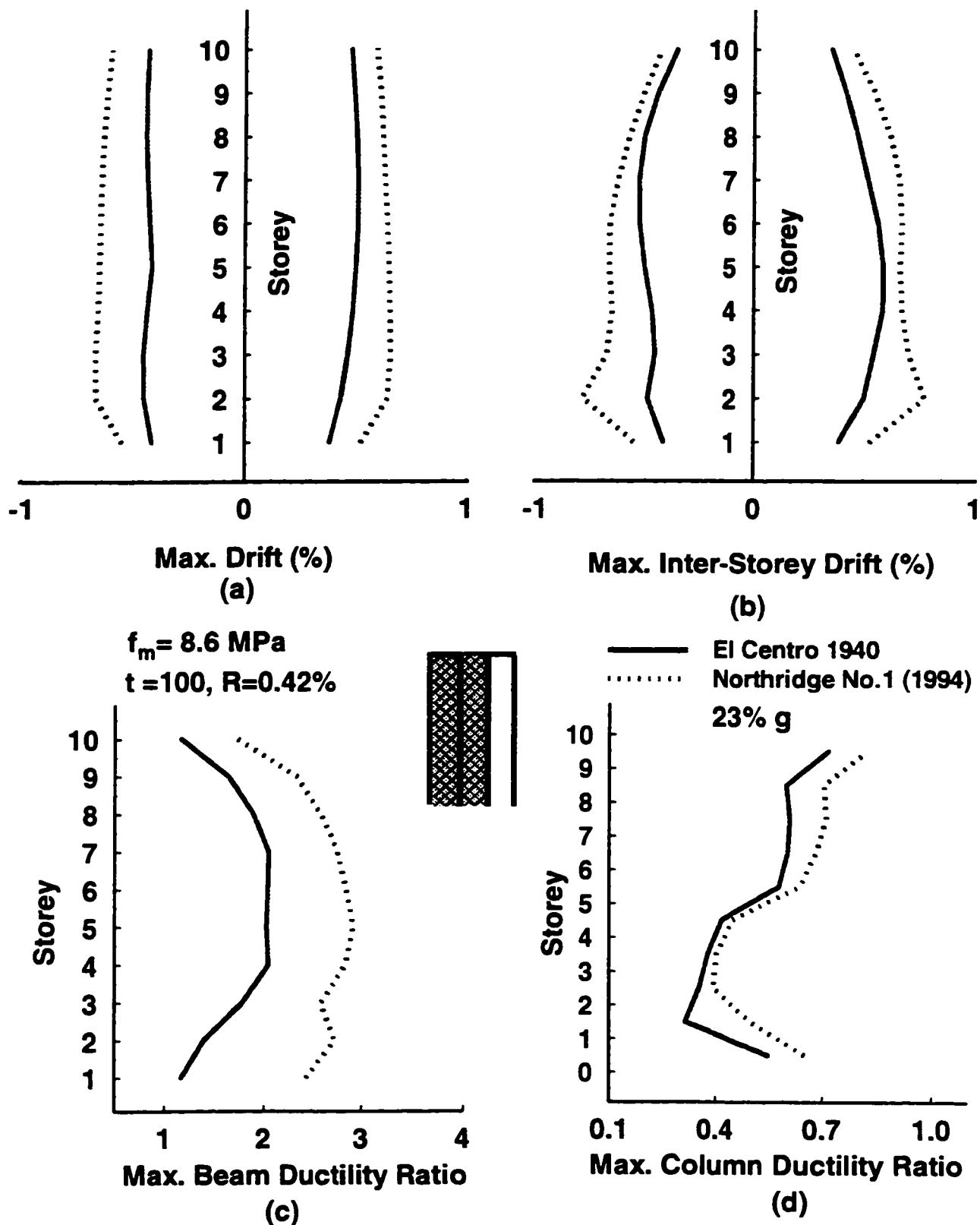


Fig. 5.34 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$) .

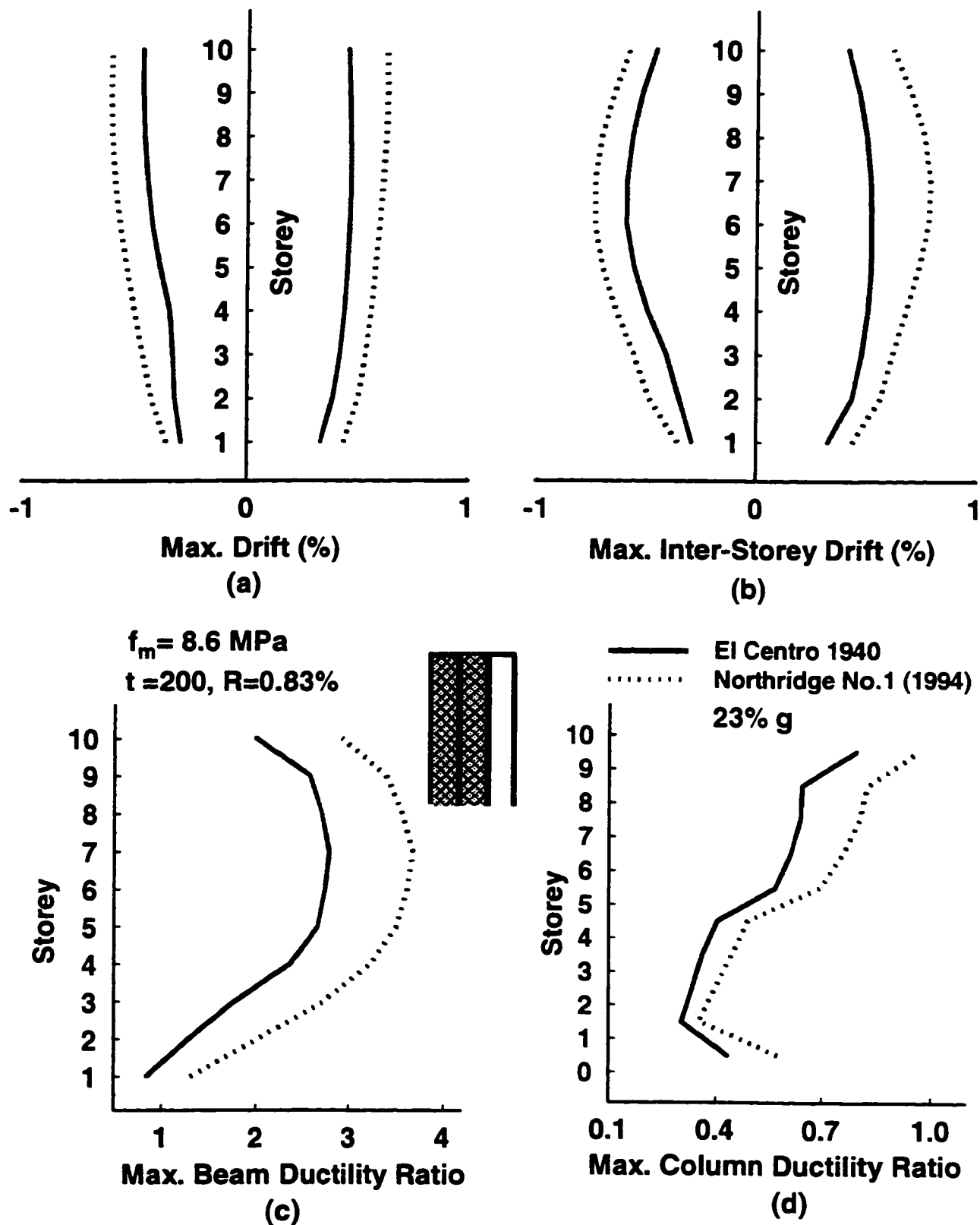


Fig. 5.35 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

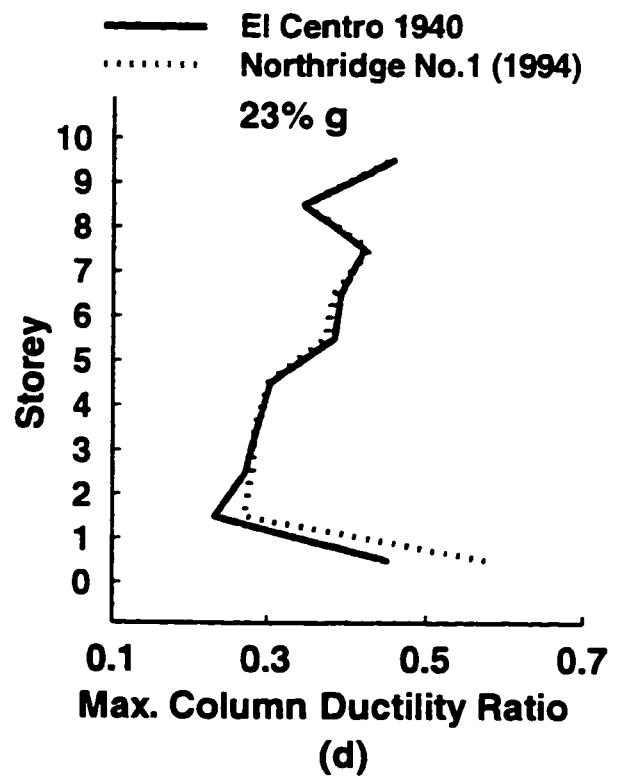
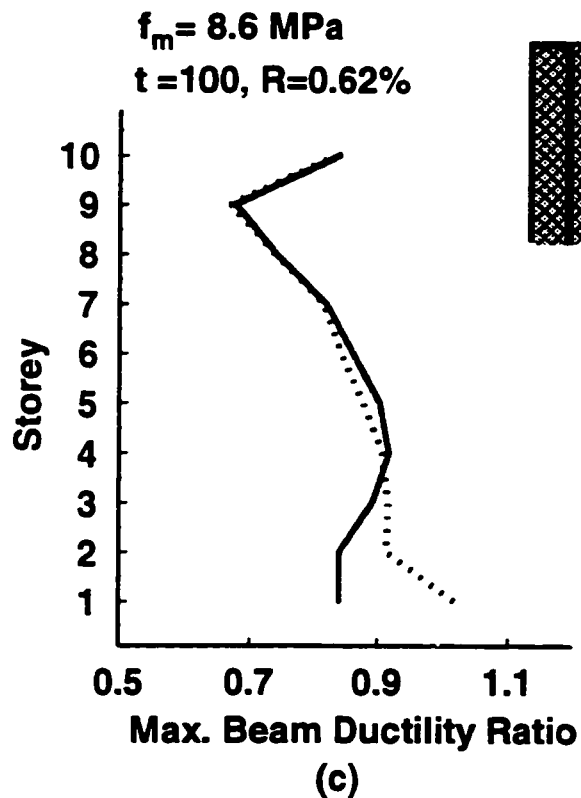
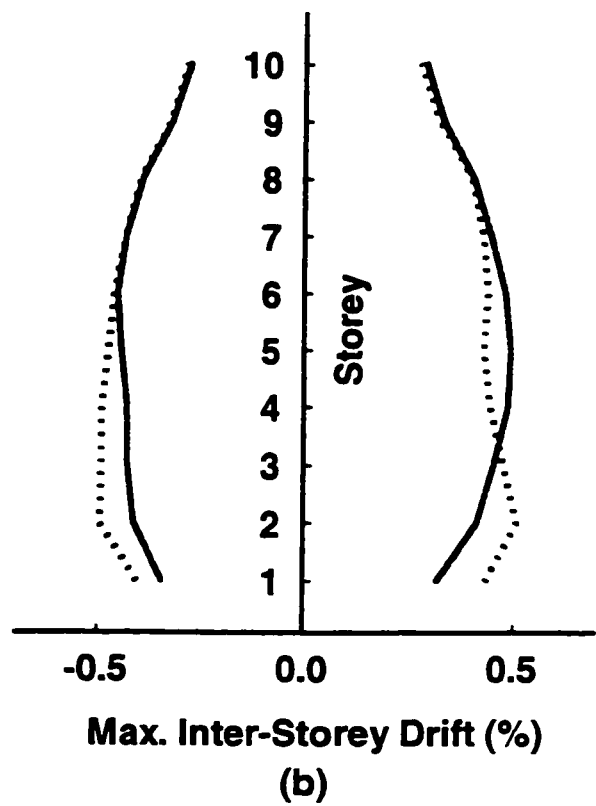
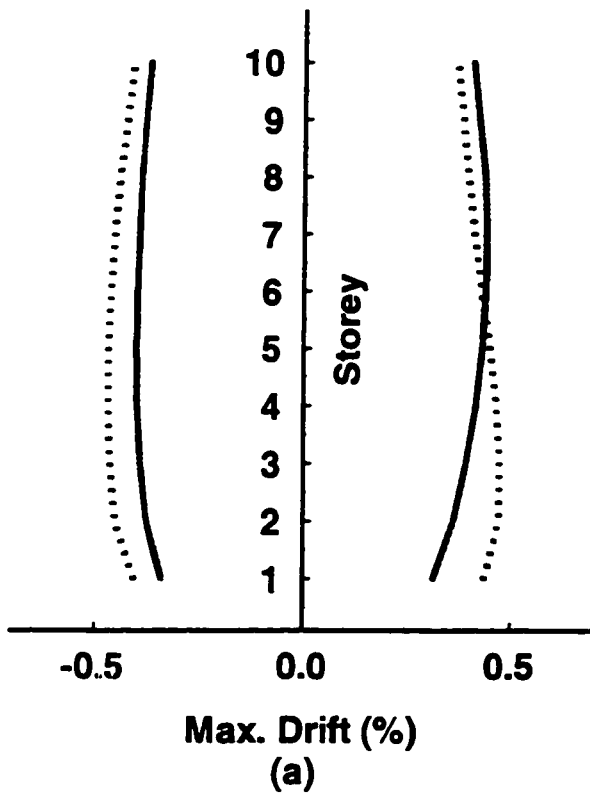


Fig. 5.36 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

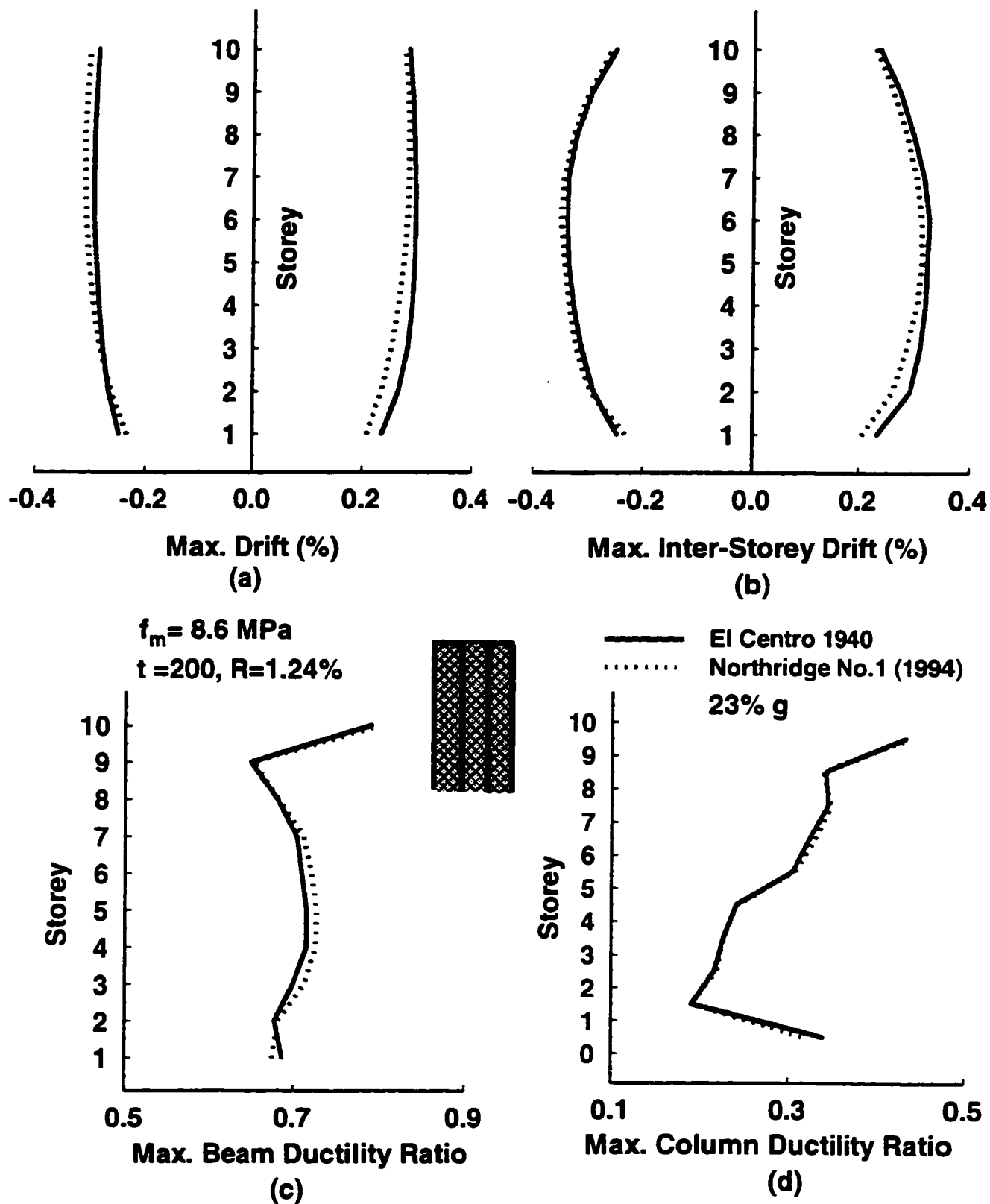


Fig. 5.37 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

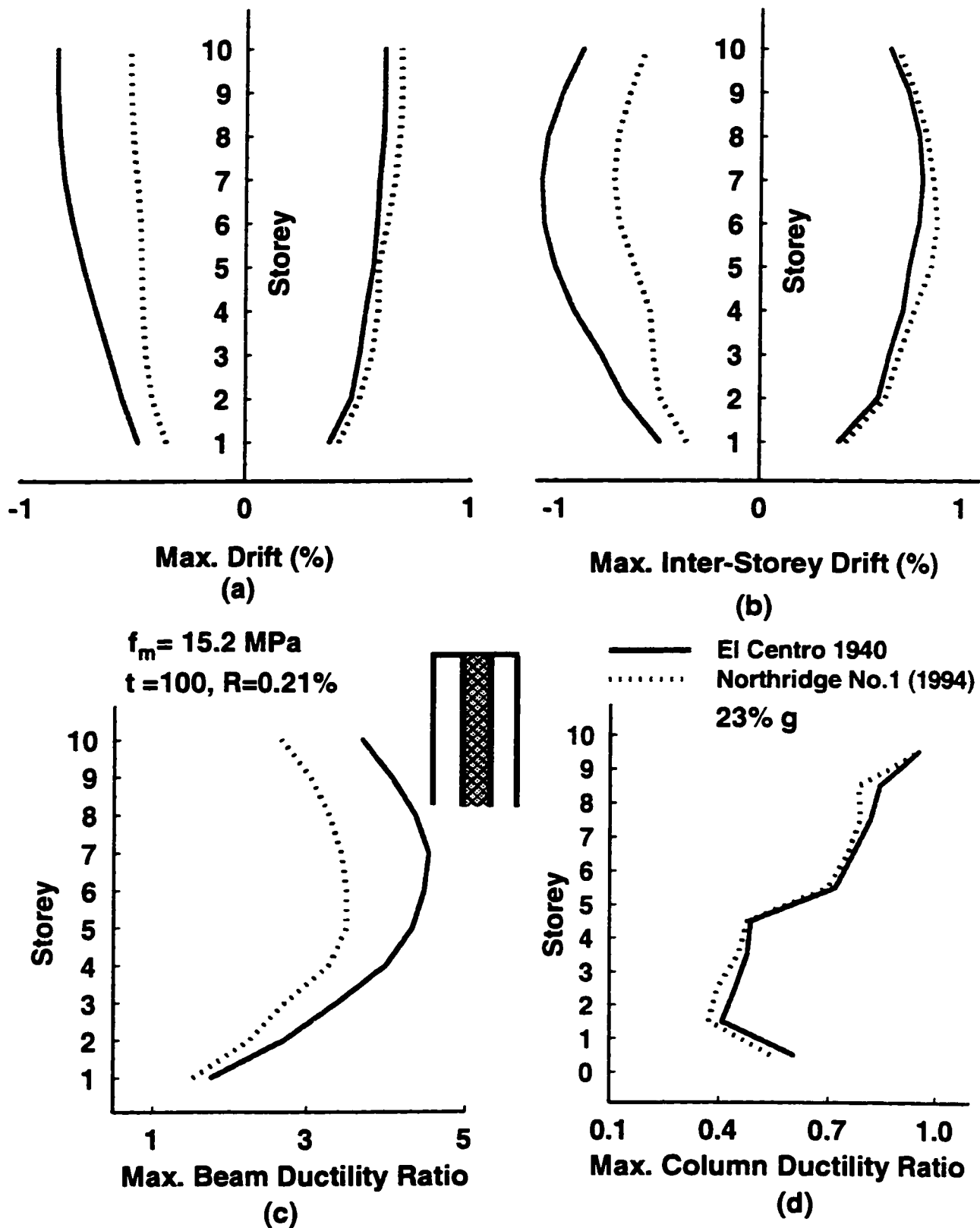


Fig. 5.38 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=100 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

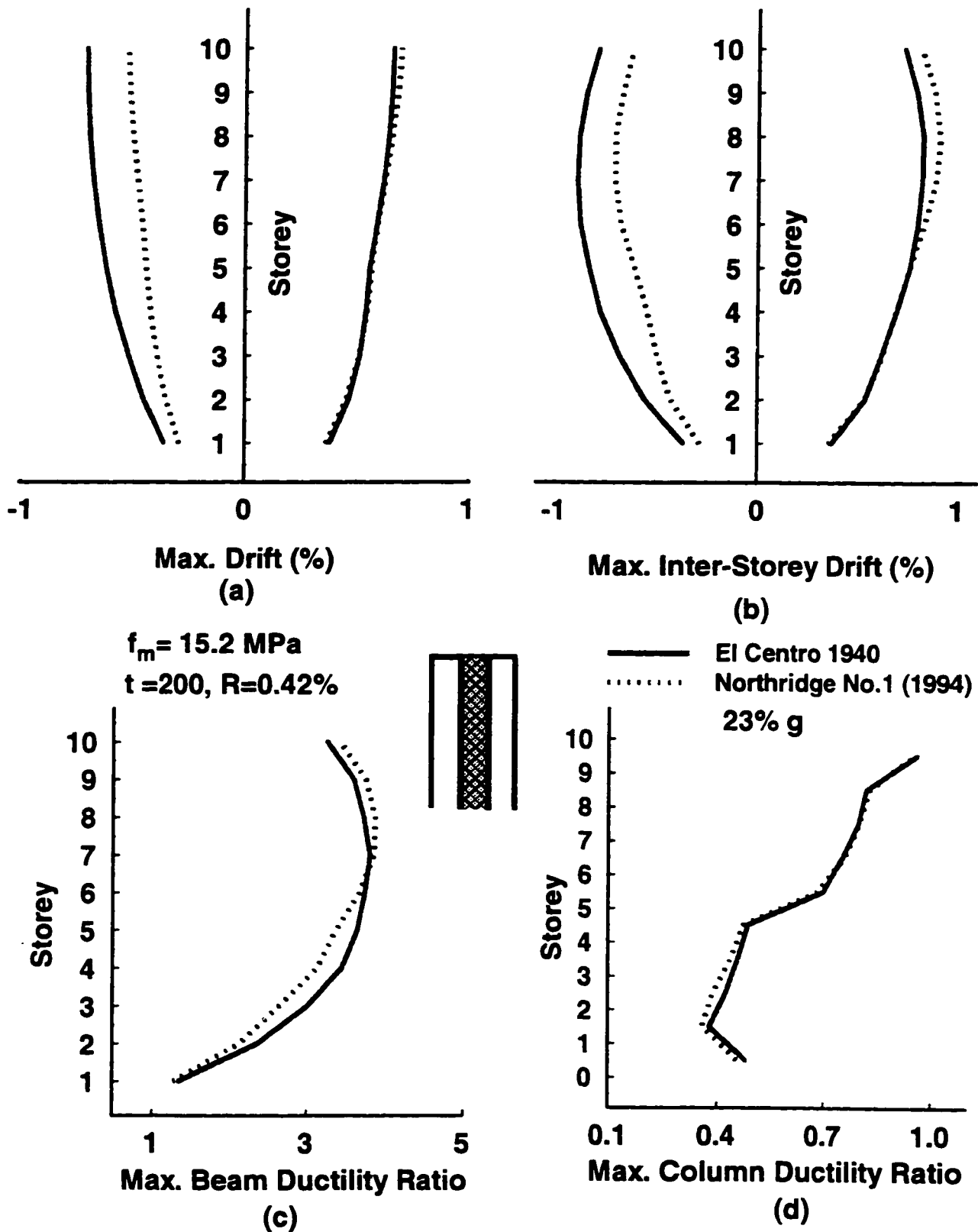


Fig. 5.39 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (One bay, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

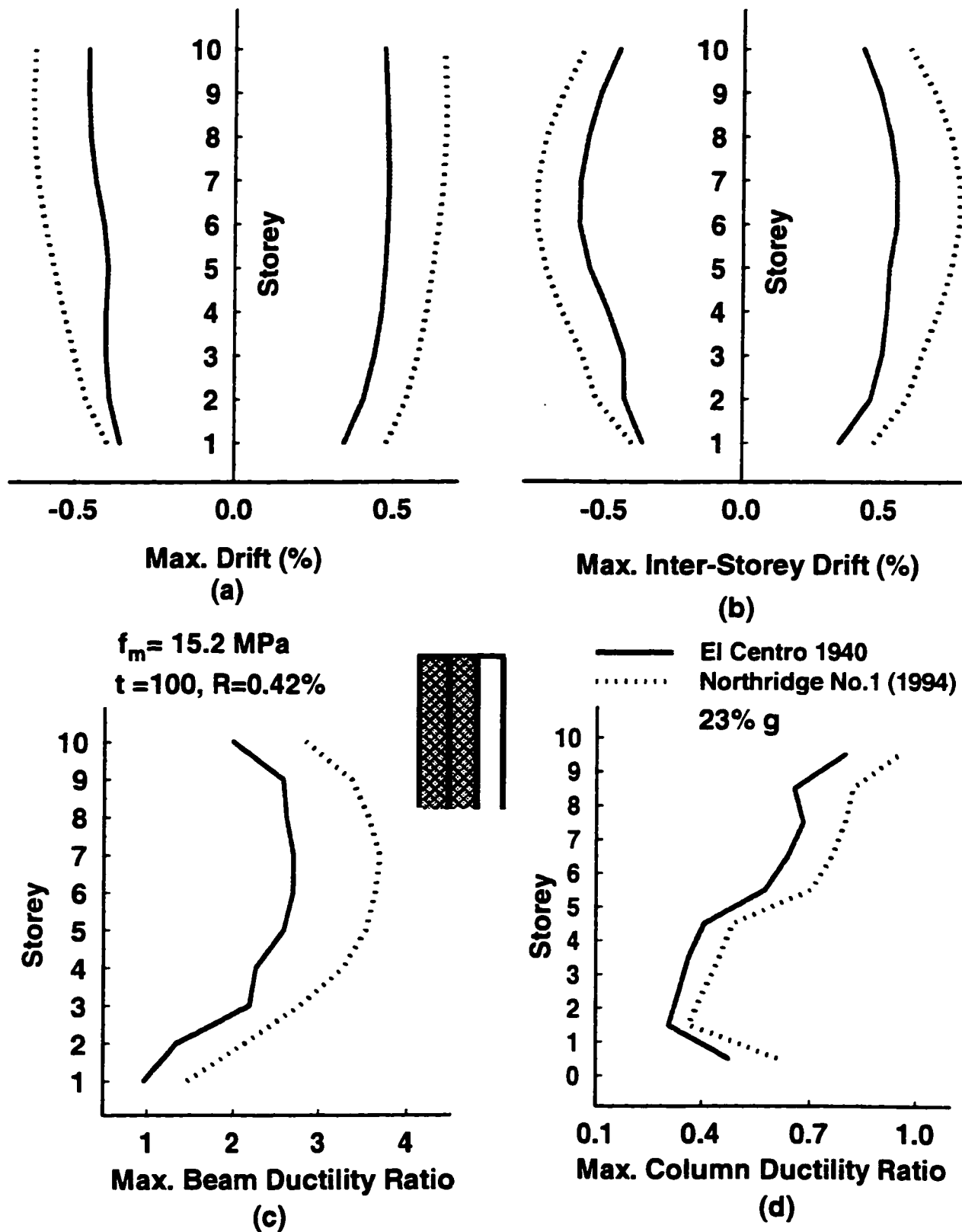


Fig. 5.40 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=100$ mm, $f_m=15.2$ MPa).

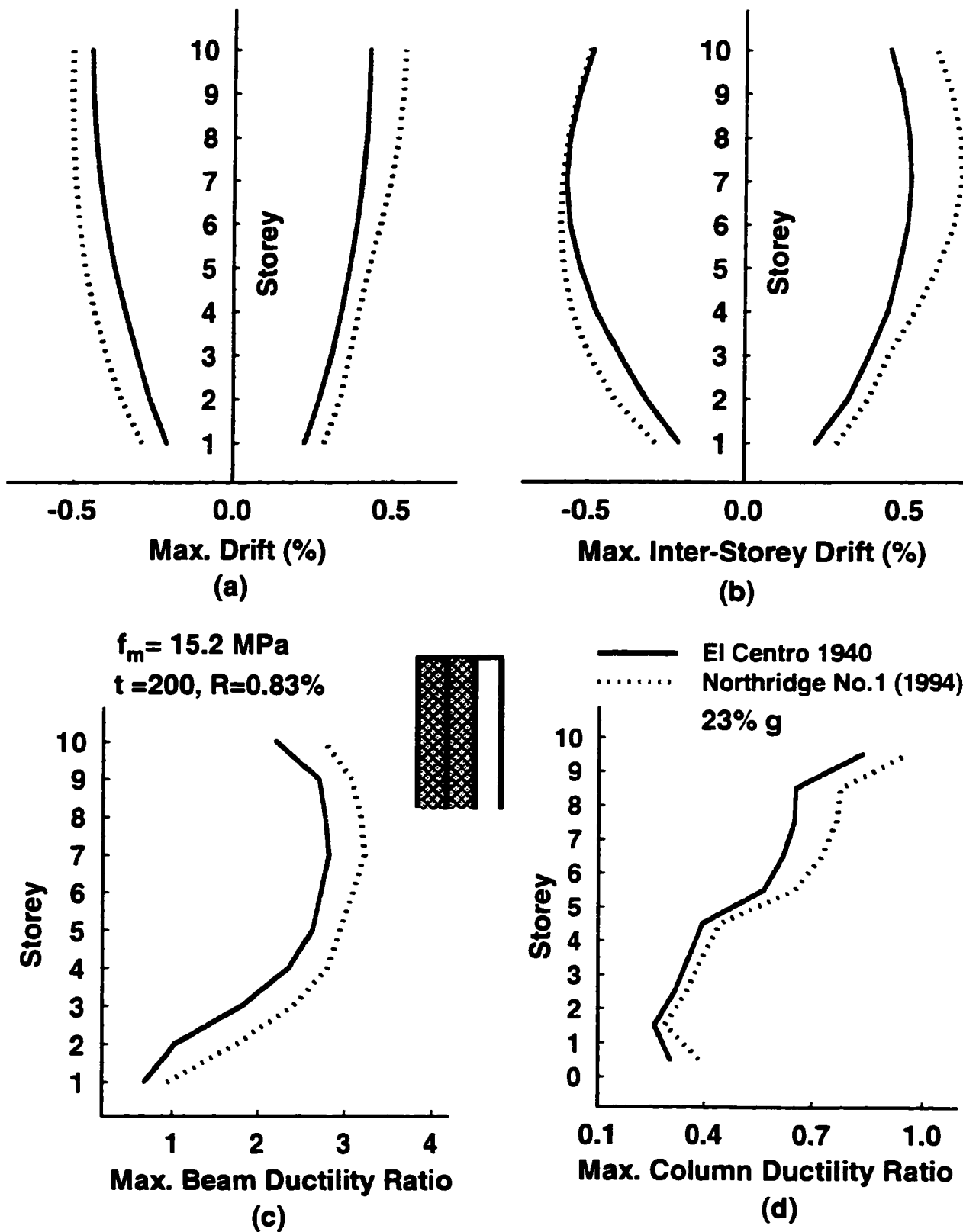


Fig. 5.41 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Two bays, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

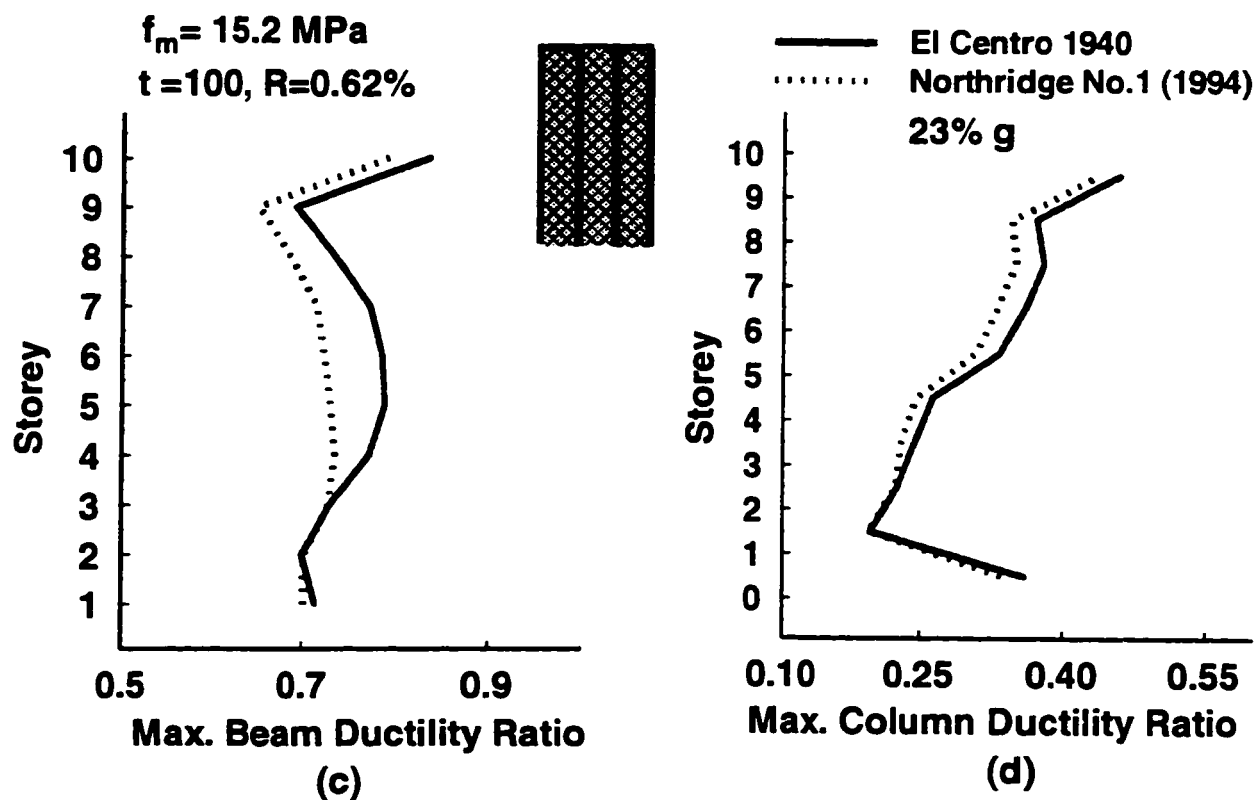
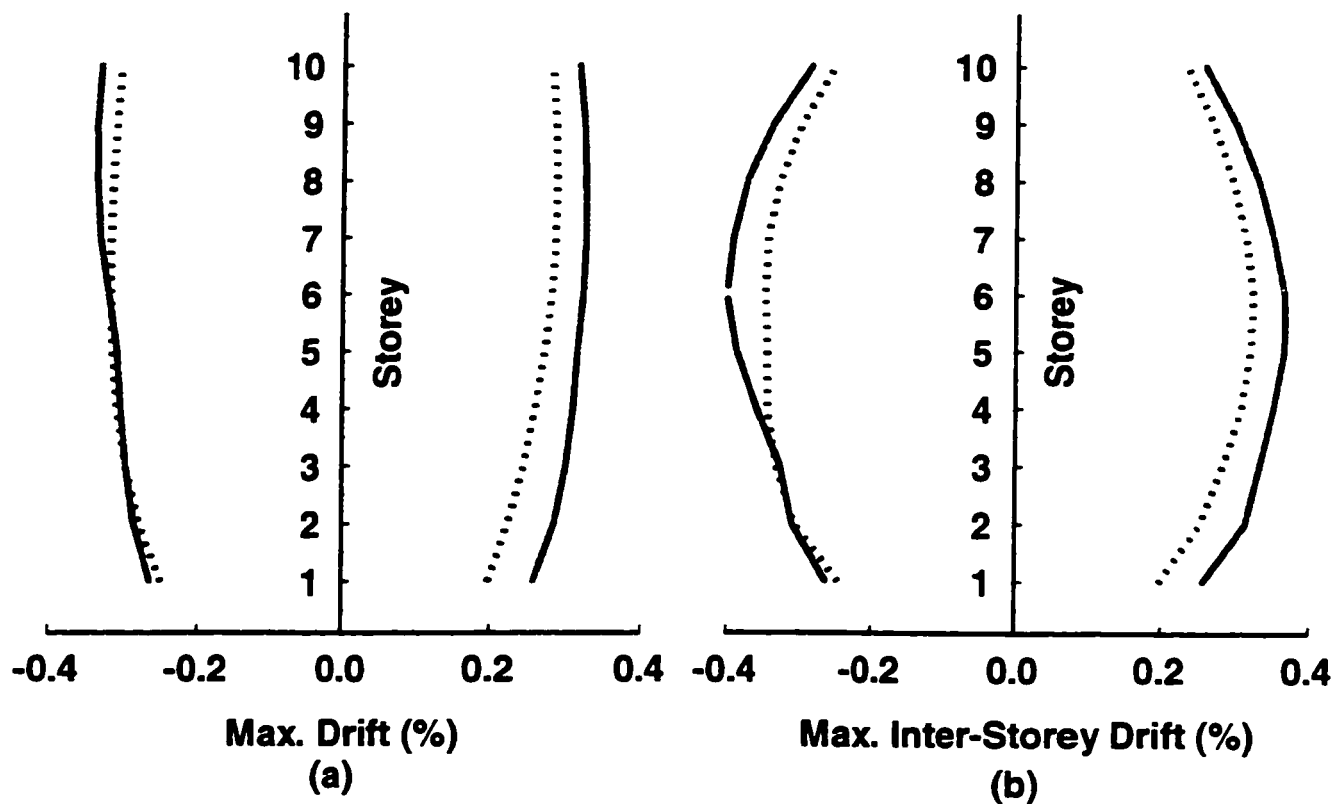


Fig. 5.42 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=100$ mm, $f_m=15.2$ MPa).

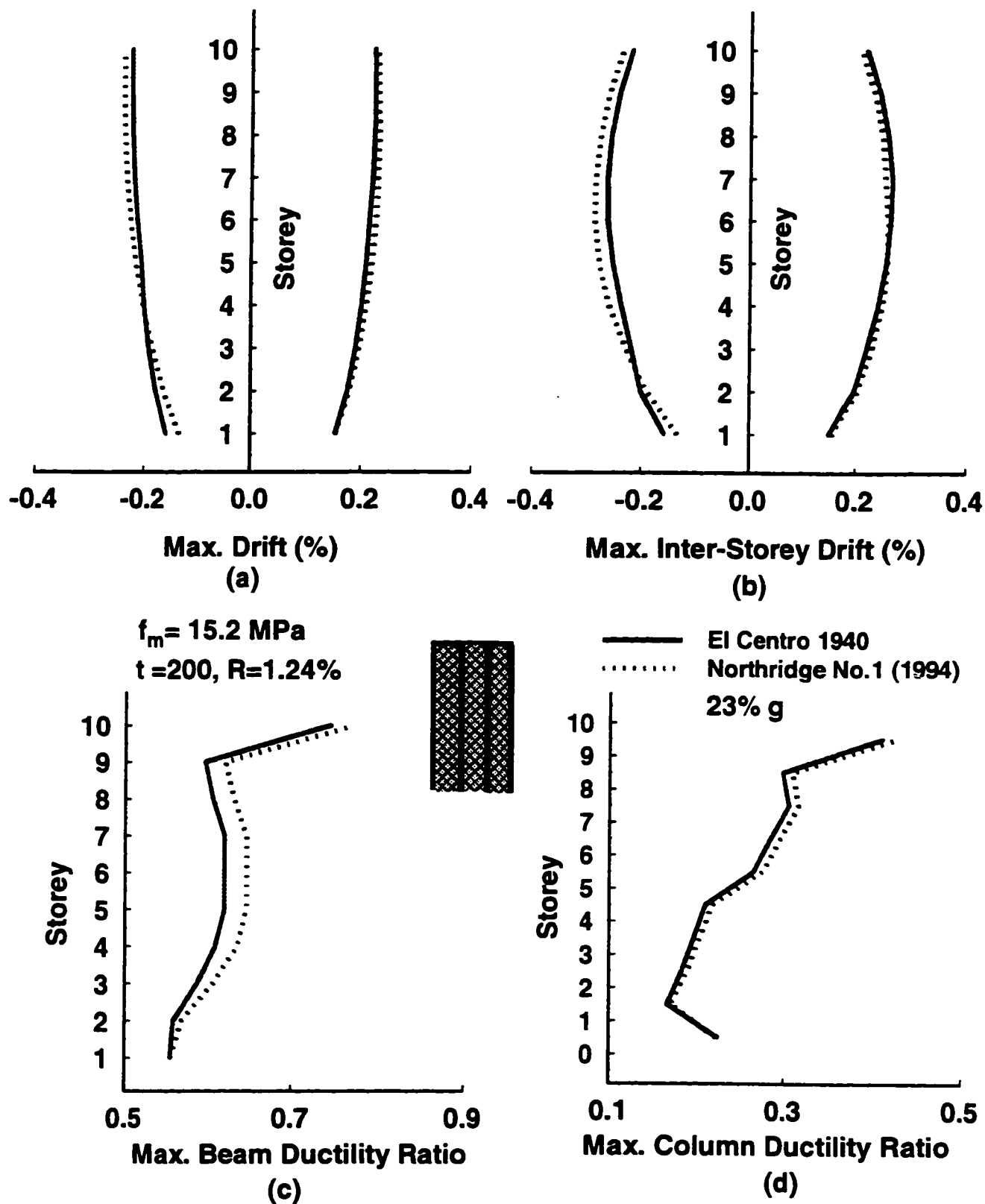


Fig. 5.43 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Actual Records (Three bays, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

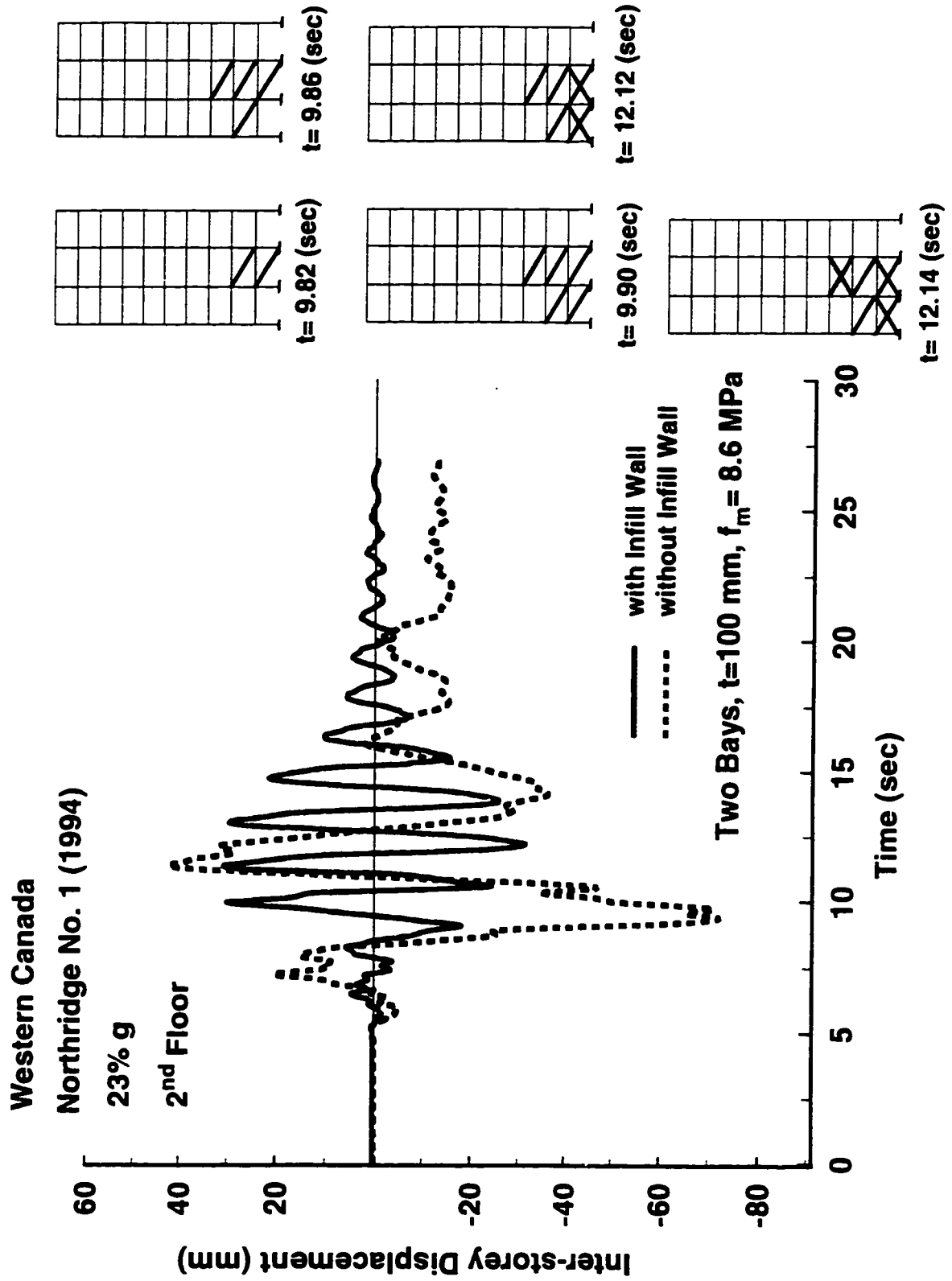


Fig. 5.44 Time History of 2nd Floor Inter-Storey Displacement
with Progression of Inelasticity in Masonry Walls

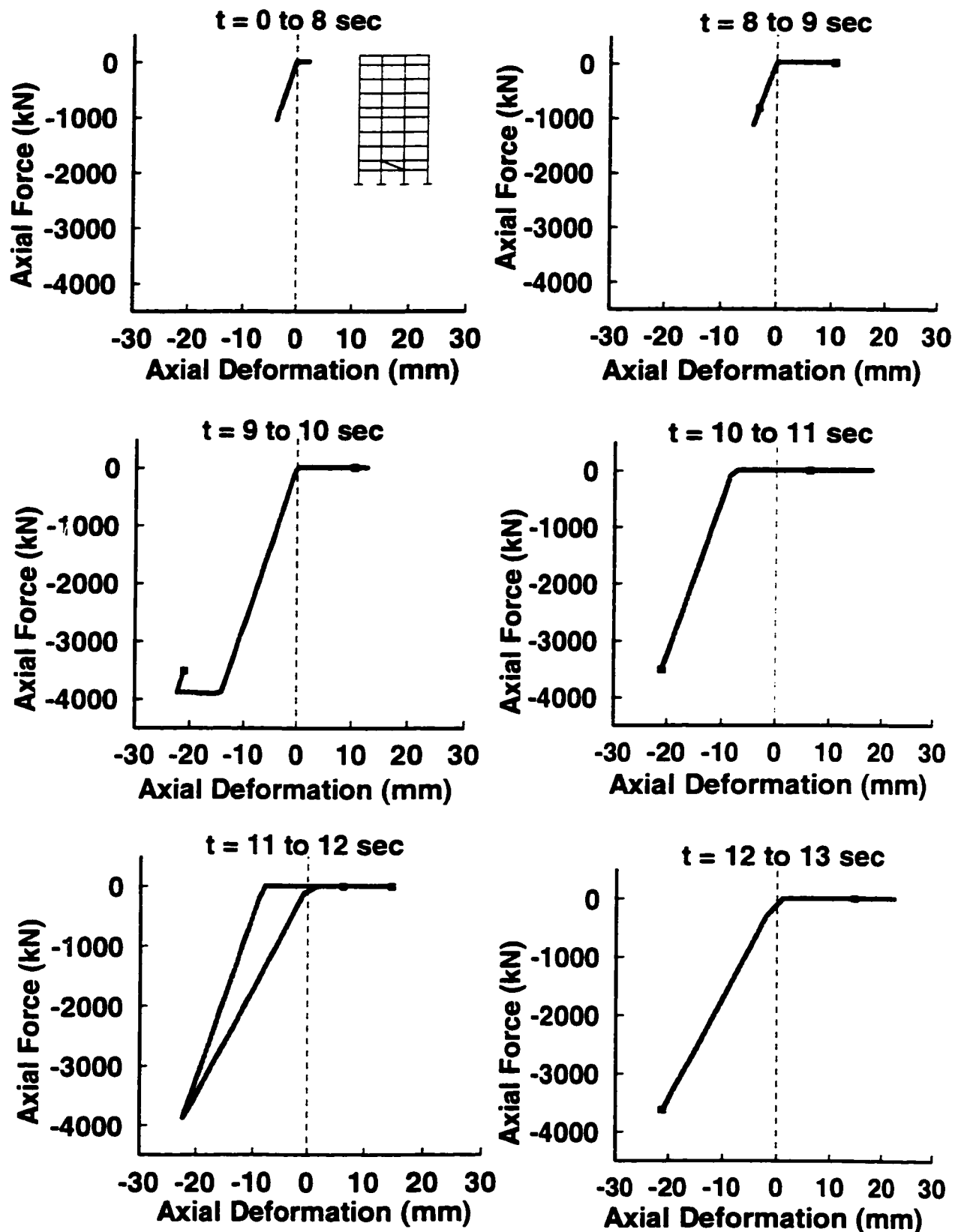


Fig. 5.45 Hysteretic Behaviour of Diagonal Strut in 2nd Floor
Masonry Wall (Northridge No.1, 23 % g)

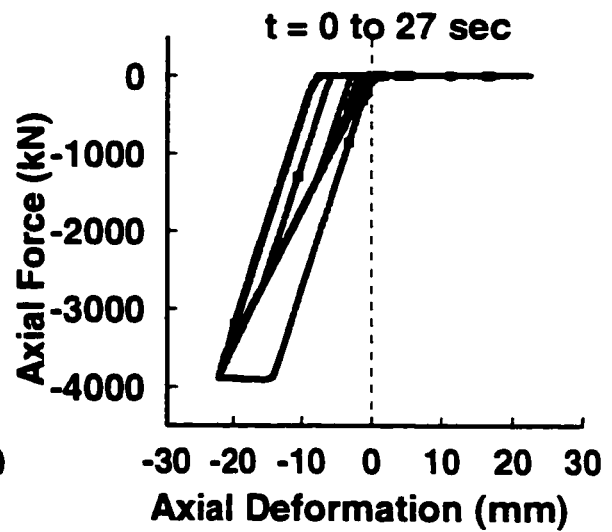
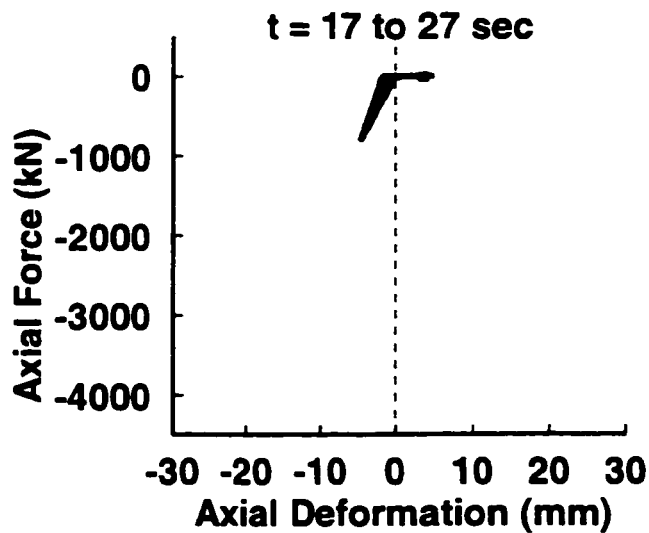
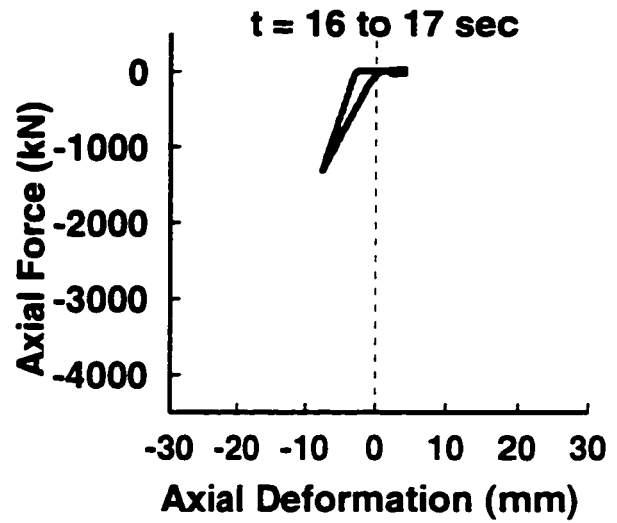
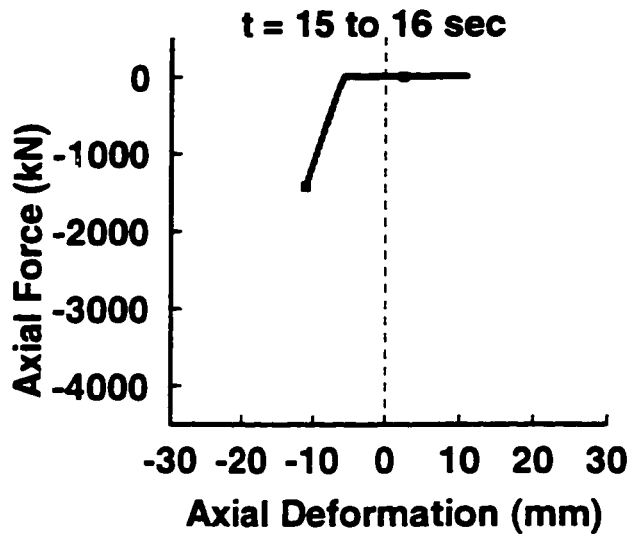
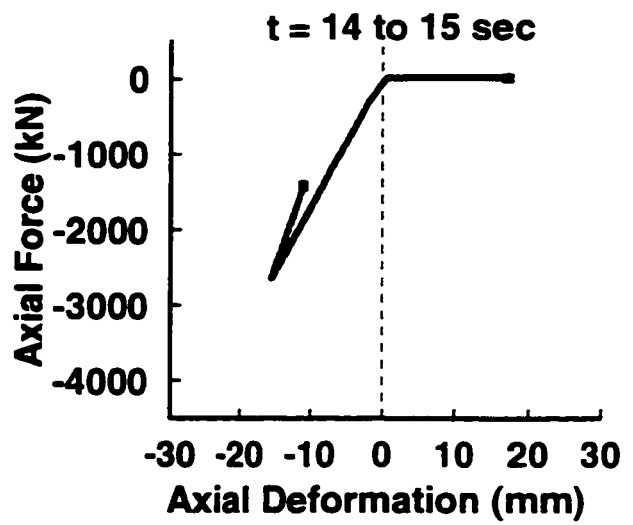
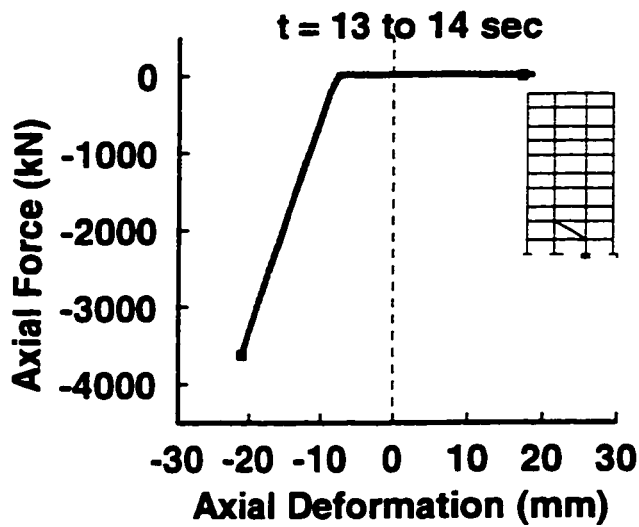


Fig. 5.45 (Cont'd)

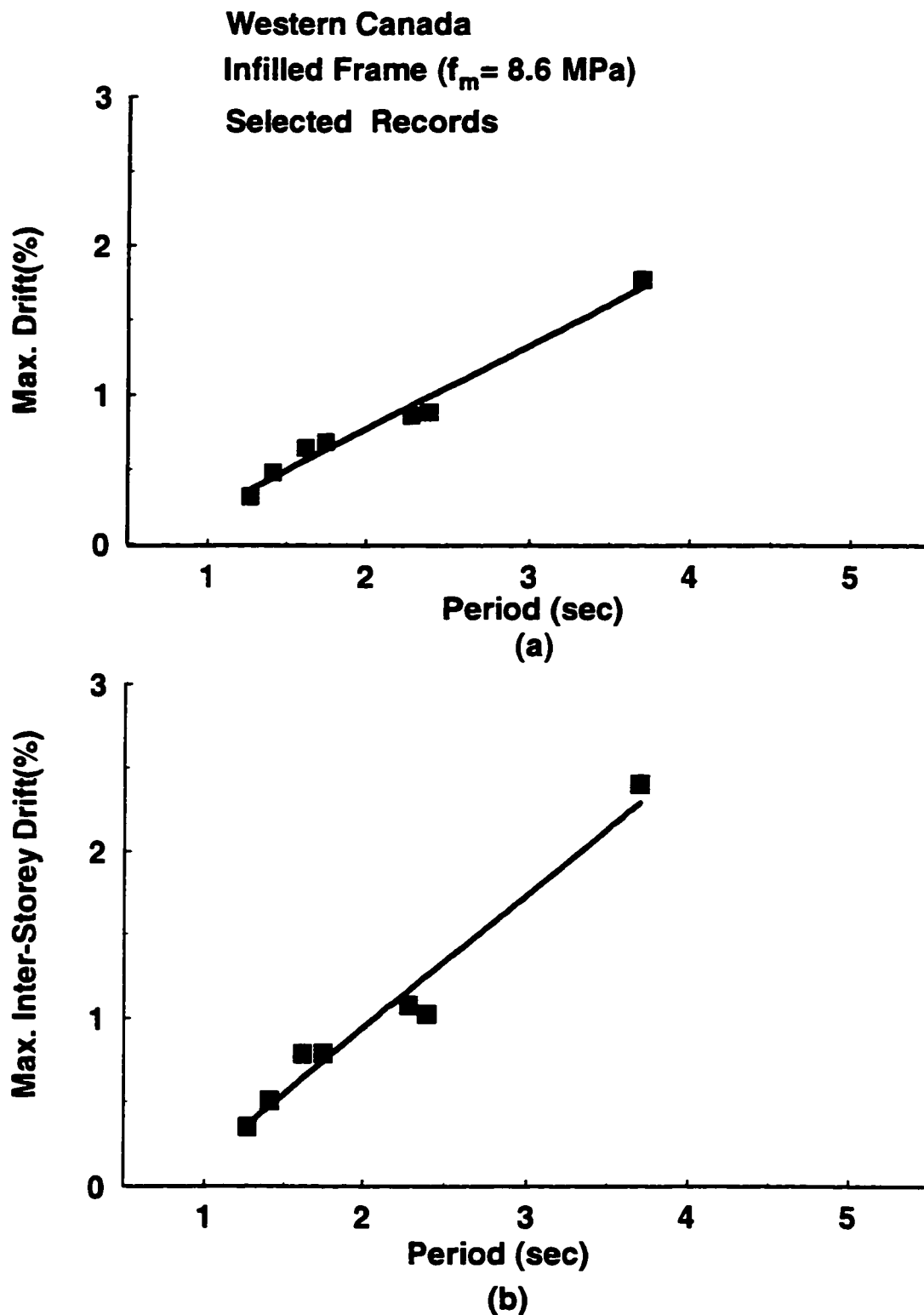


Fig. 5.46 The Relationship Between Maximum Drift of The Fundamental Period of Infilled Frames Subjected to Selected Records ($f_m=8.6$ MPa).

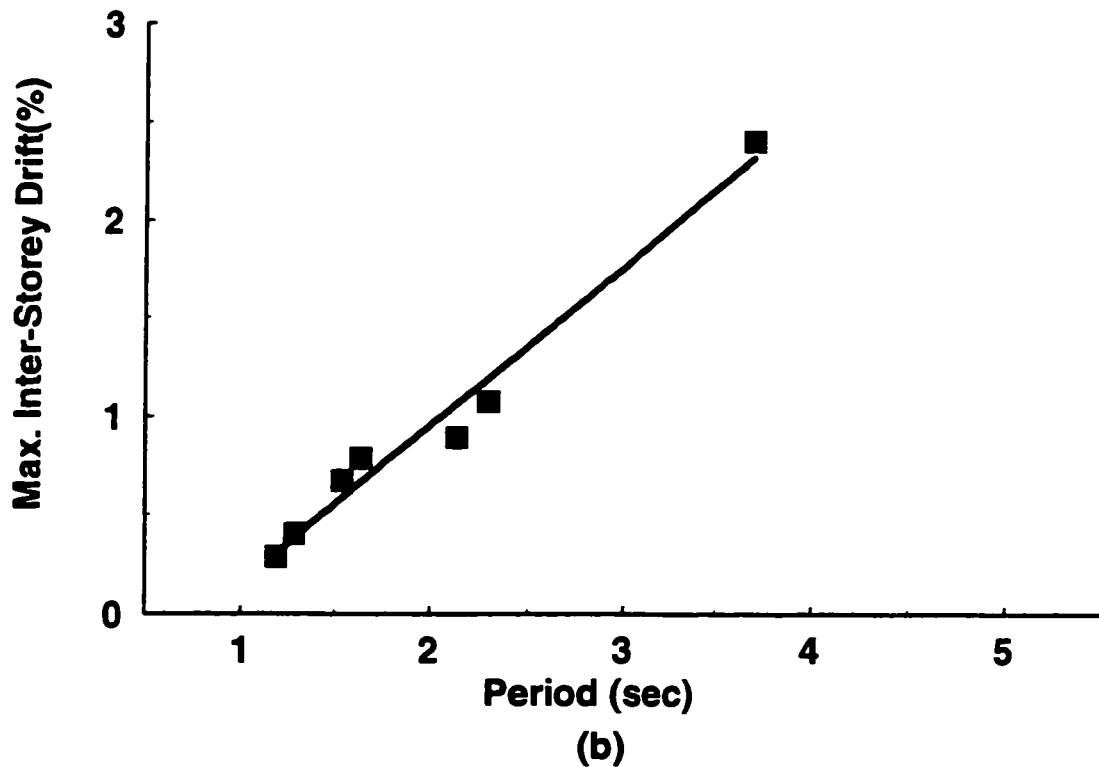
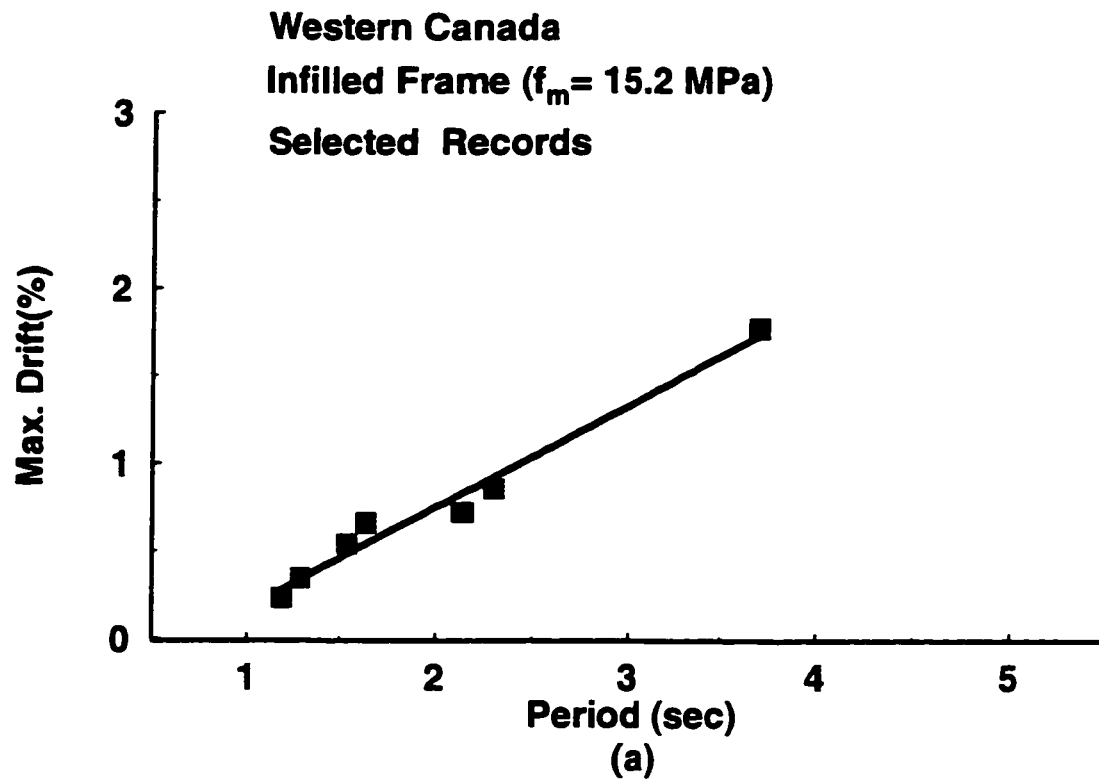
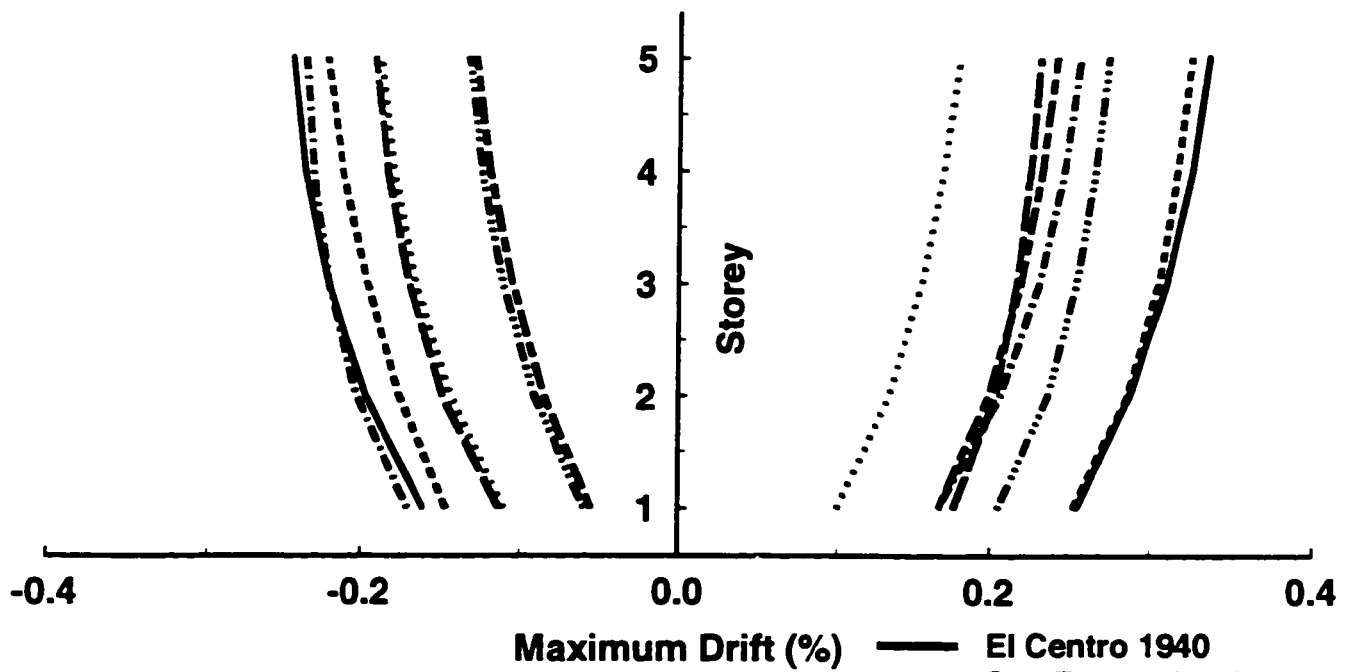
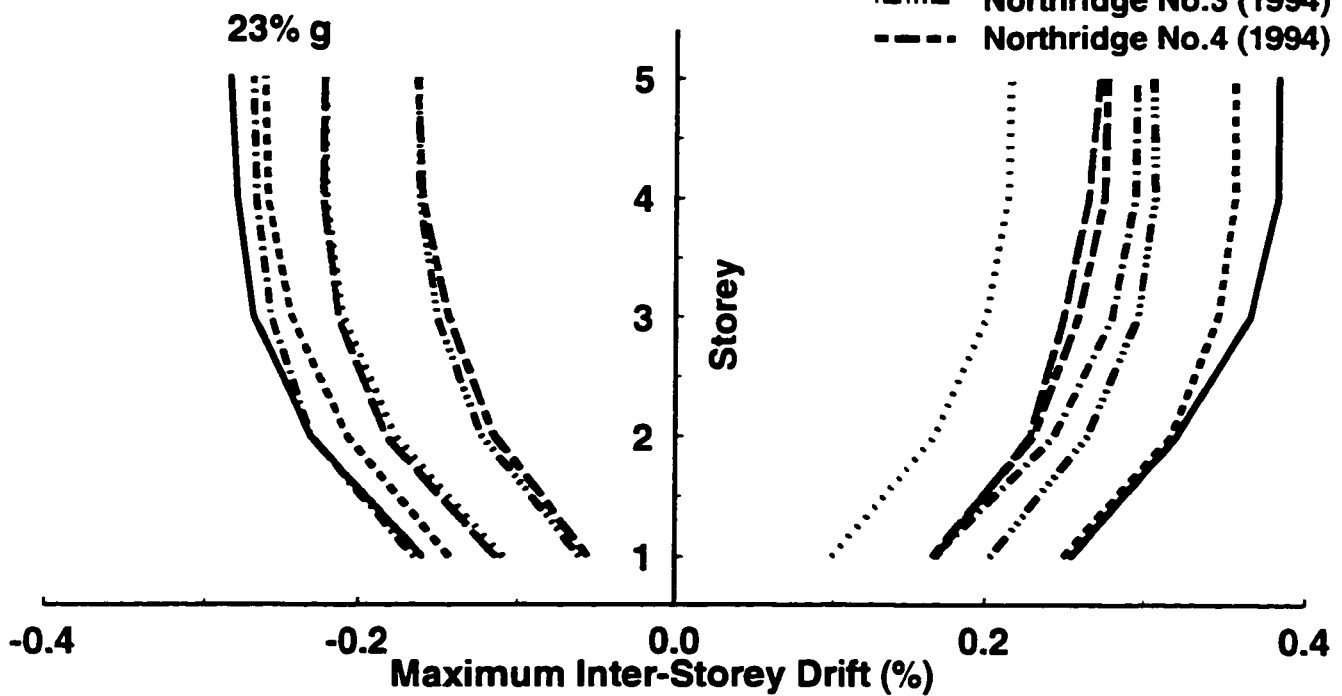


Fig. 5.47 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Selected Records ($f_m = 15.2$ MPa).



(a)



(b)

Fig. 5.48 Drift Response for 5-Storey SHW Subjected to All Selected Records for Western Canada.

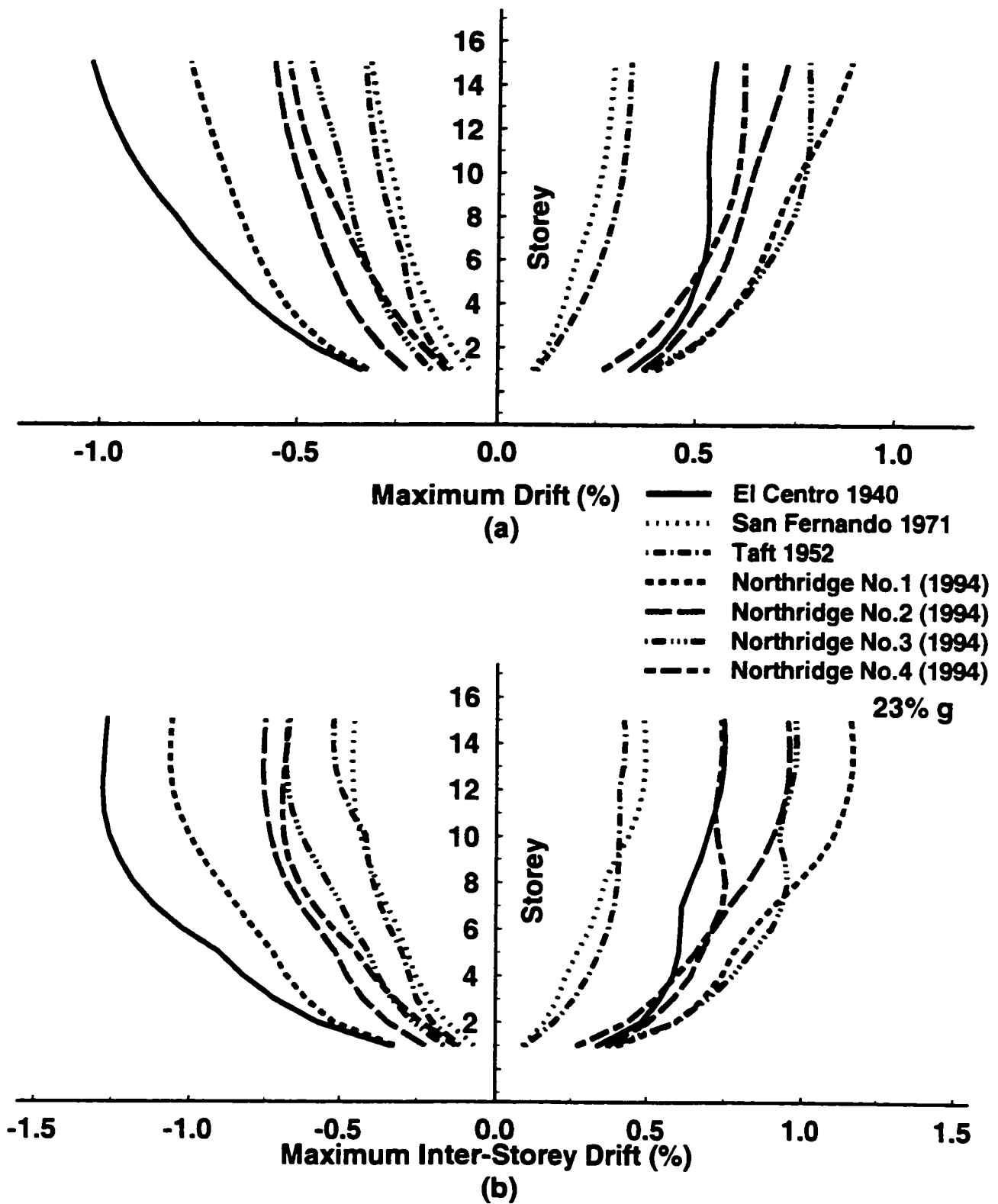
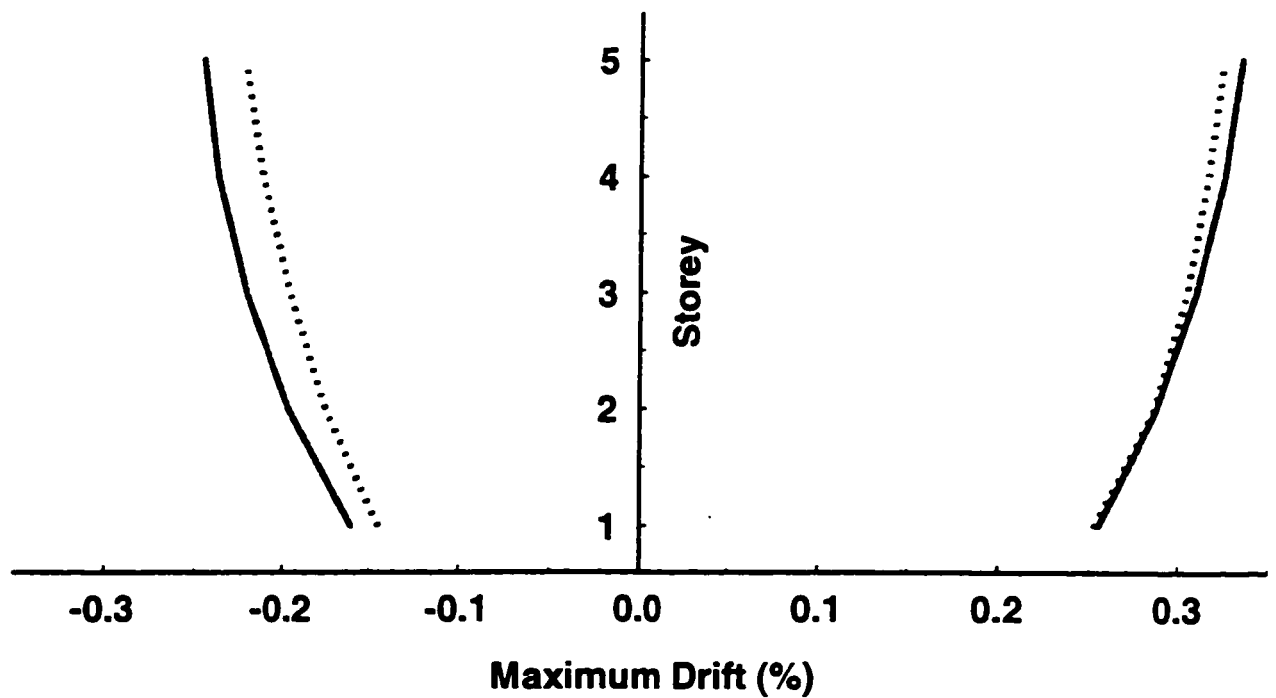


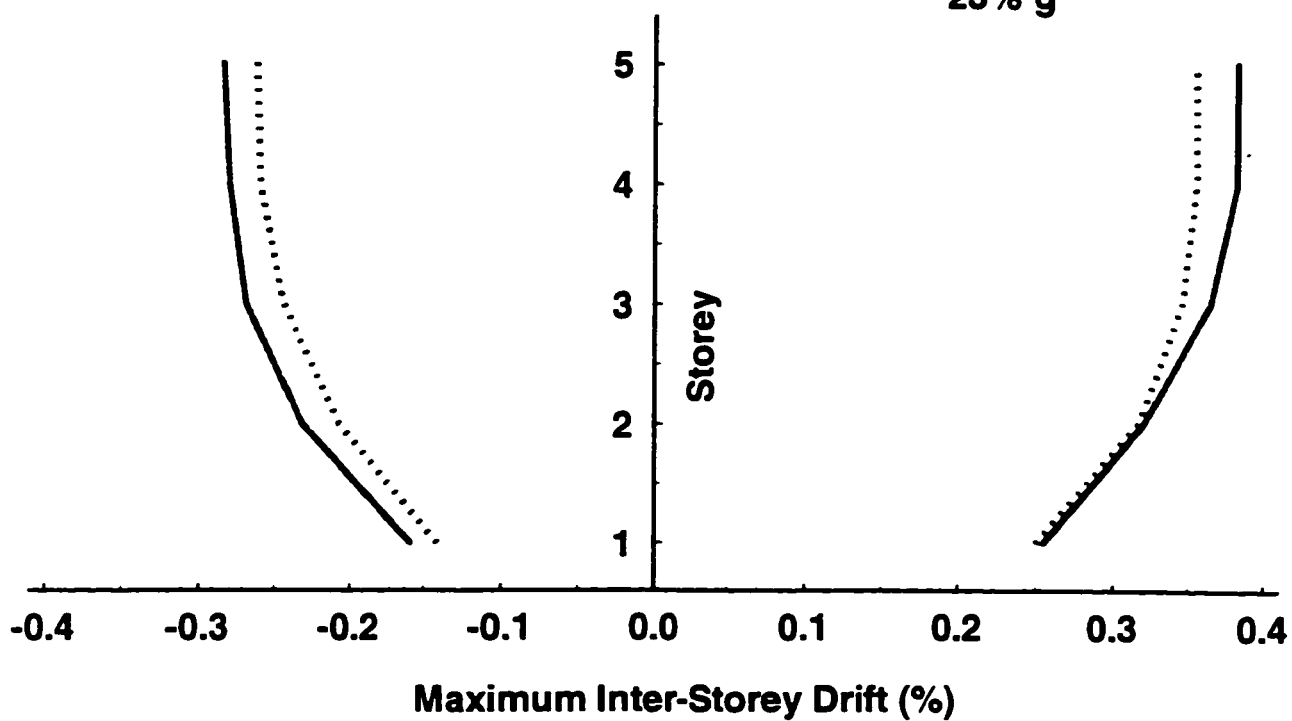
Fig. 5.49 Drift Response for 15-Storey SHW Subjected to All Selected Records for Western Canada.



(a)

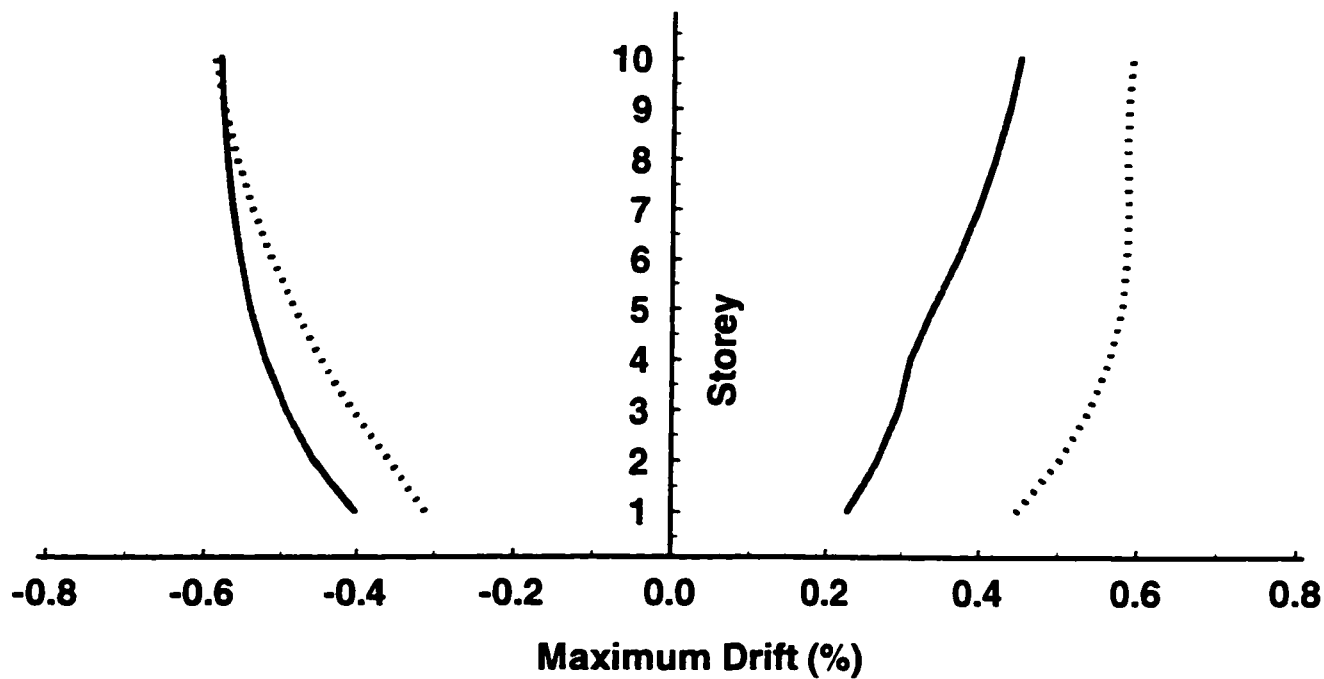
— El Centro 1940
 Northridge No.1 (1994)

23% g



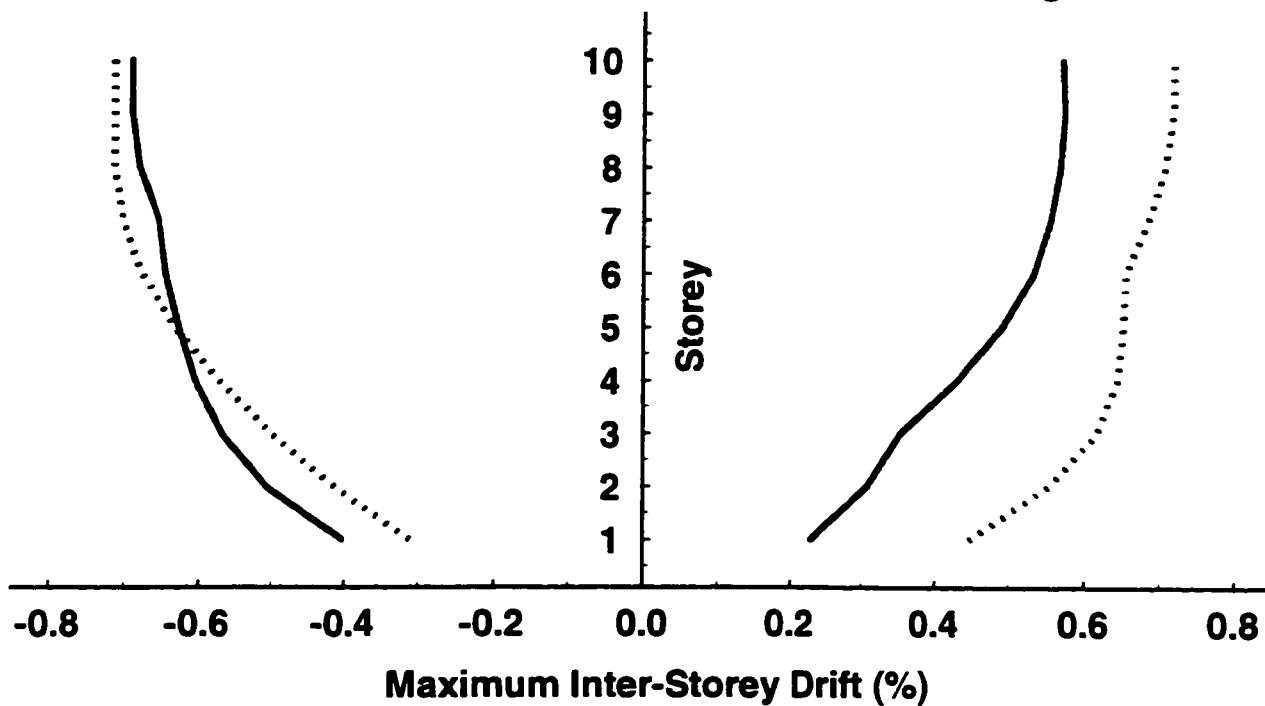
(b)

Fig. 5.50 Drift Response for 5-Storey SHW Subjected to Actual Records for Western Canada.



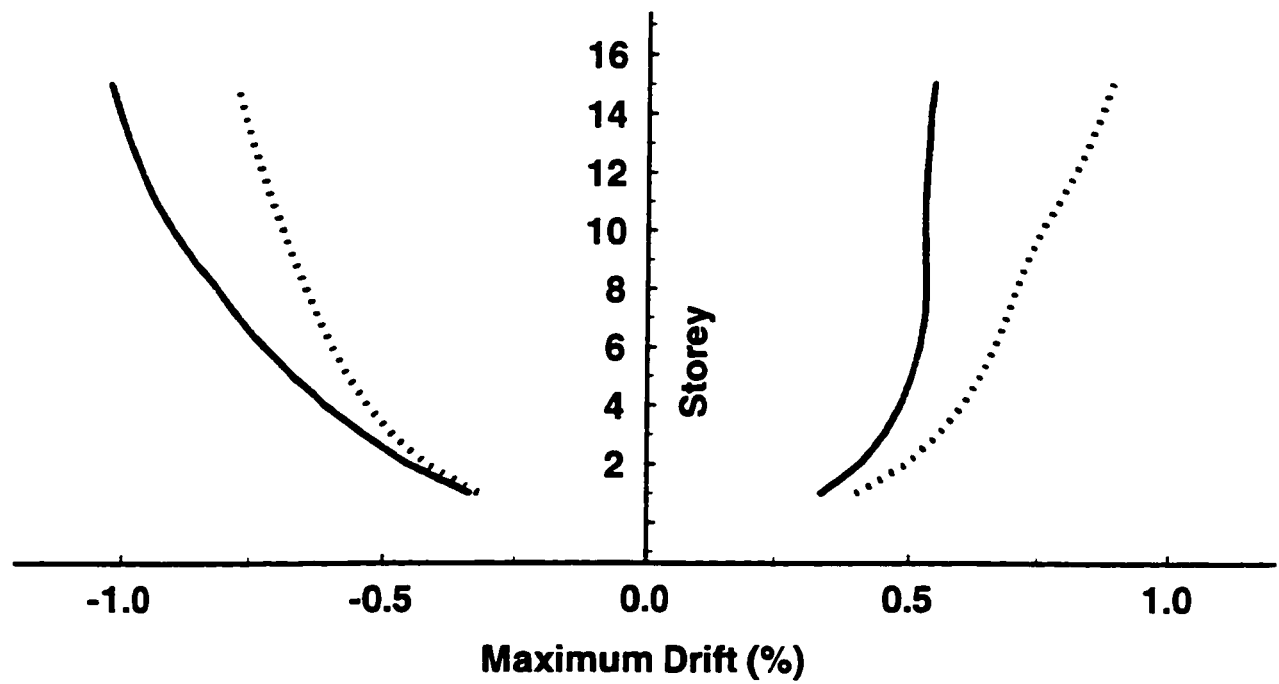
(a)

— El Centro 1940
 Northridge No.1 (1994)
 23% g



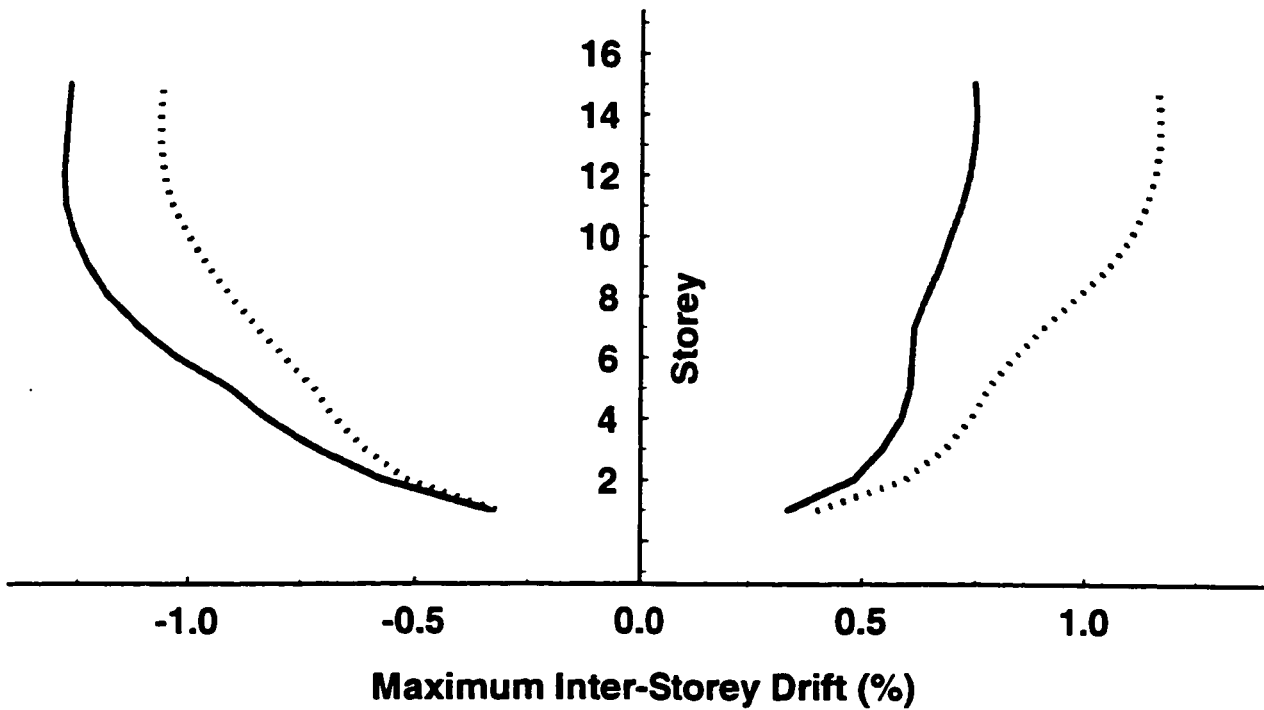
(b)

Fig. 5.51 Drift Response for 10-Storey SHW Subjected to Actual Records for Western Canada.



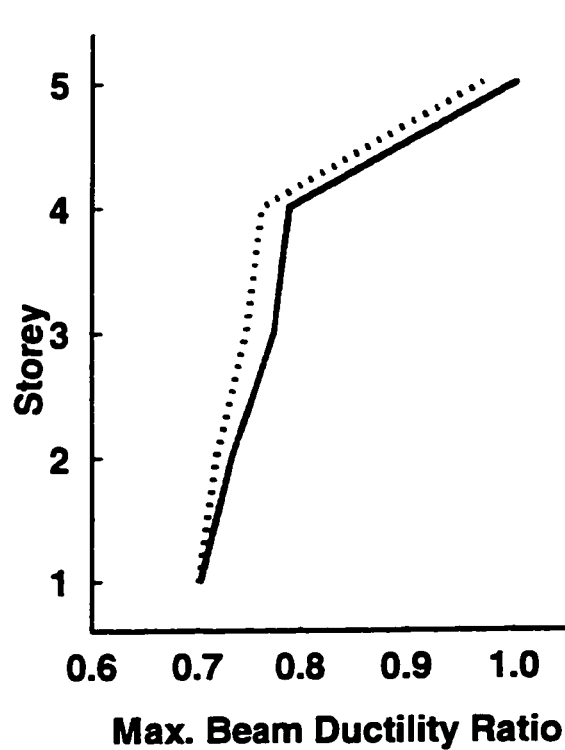
(a)

— El Centro 1940
 Northridge No.1 (1994)
 23% g

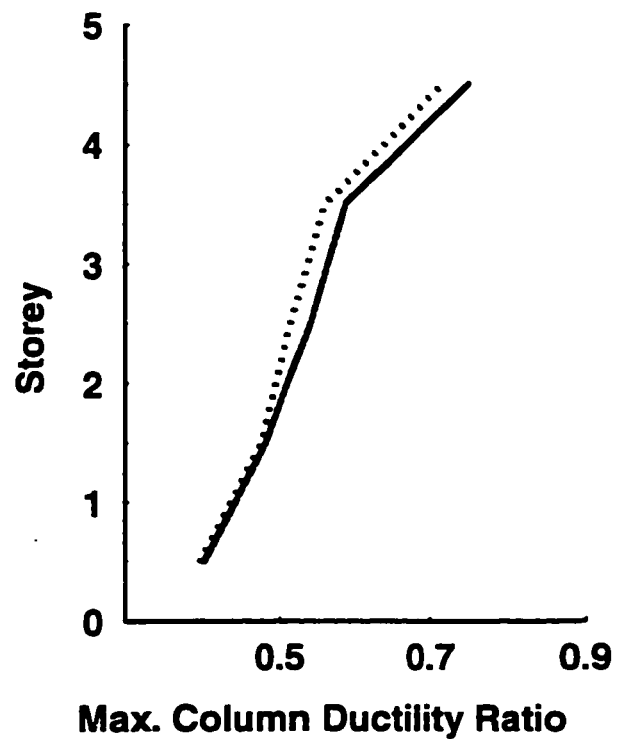


(b)

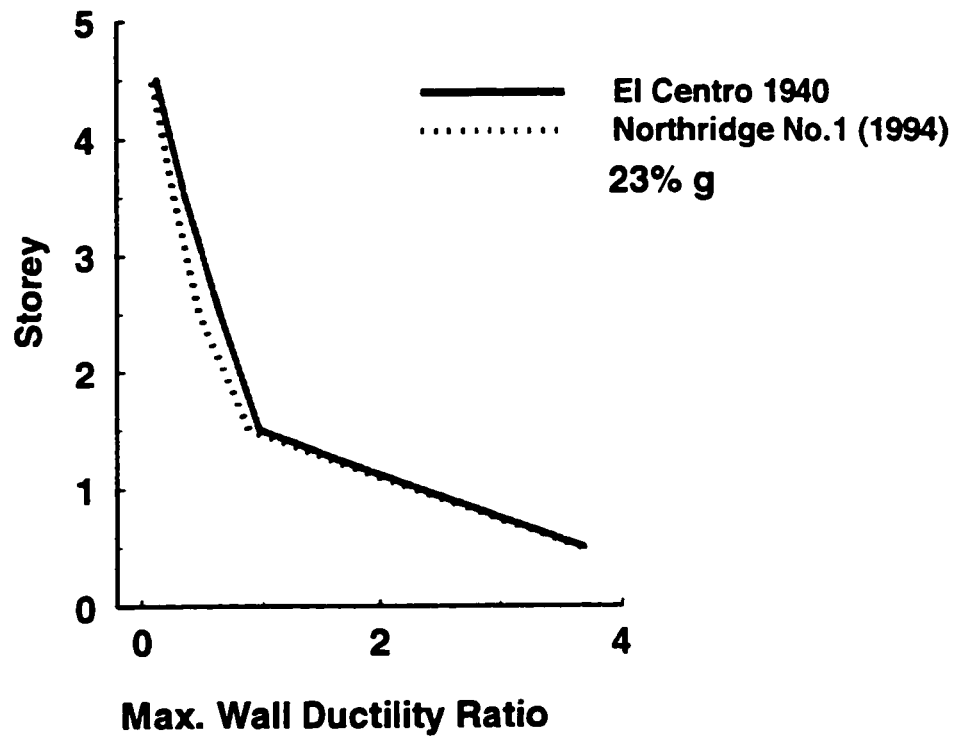
Fig. 5.52 Drift Response for 15-Storey SHW Subjected to Actual Records for Western Canada.



(a)

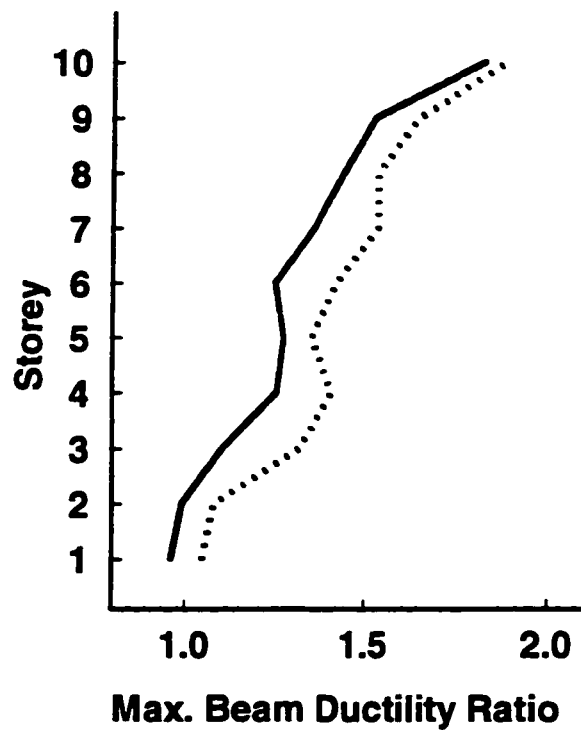


(b)

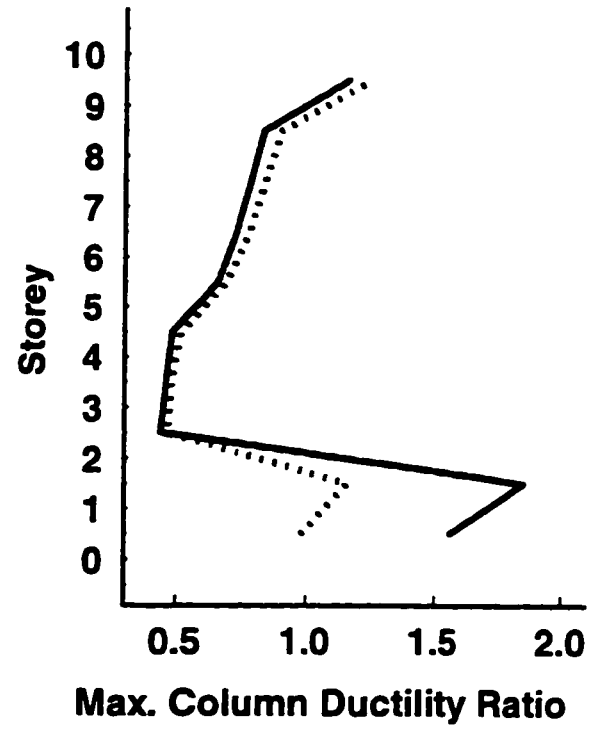


(c)

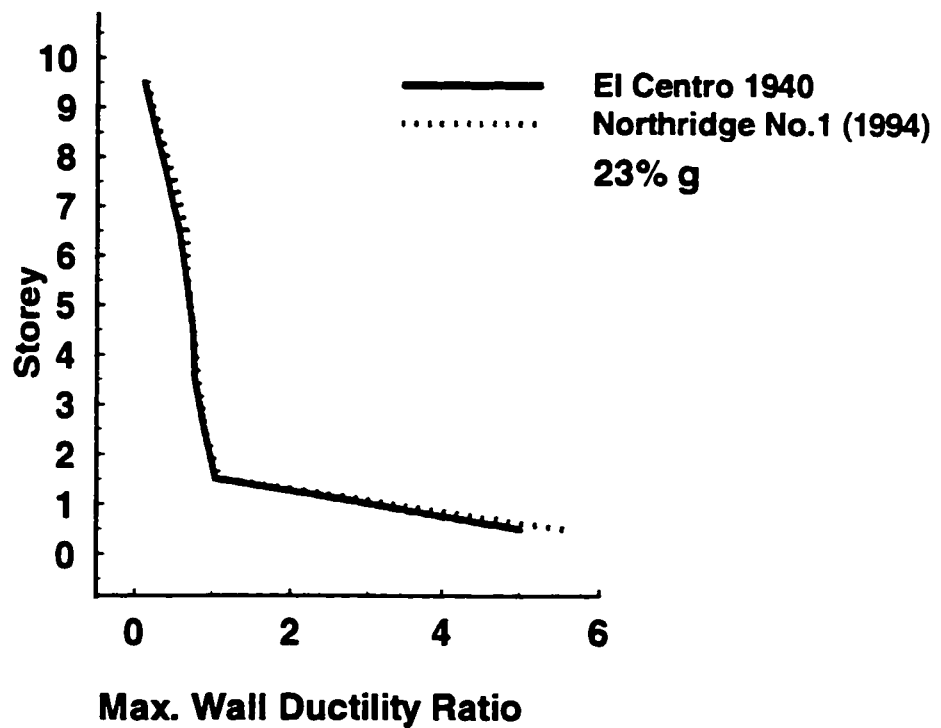
Fig. 5.53 Ductility Response for 5-Storey SHW Subjected to Actual Records for Western Canada.



(a)



(b)



(c)

Fig. 5.54 Ductility Response for 10-Storey SHW Subjected to Actual Records for Western Canada.

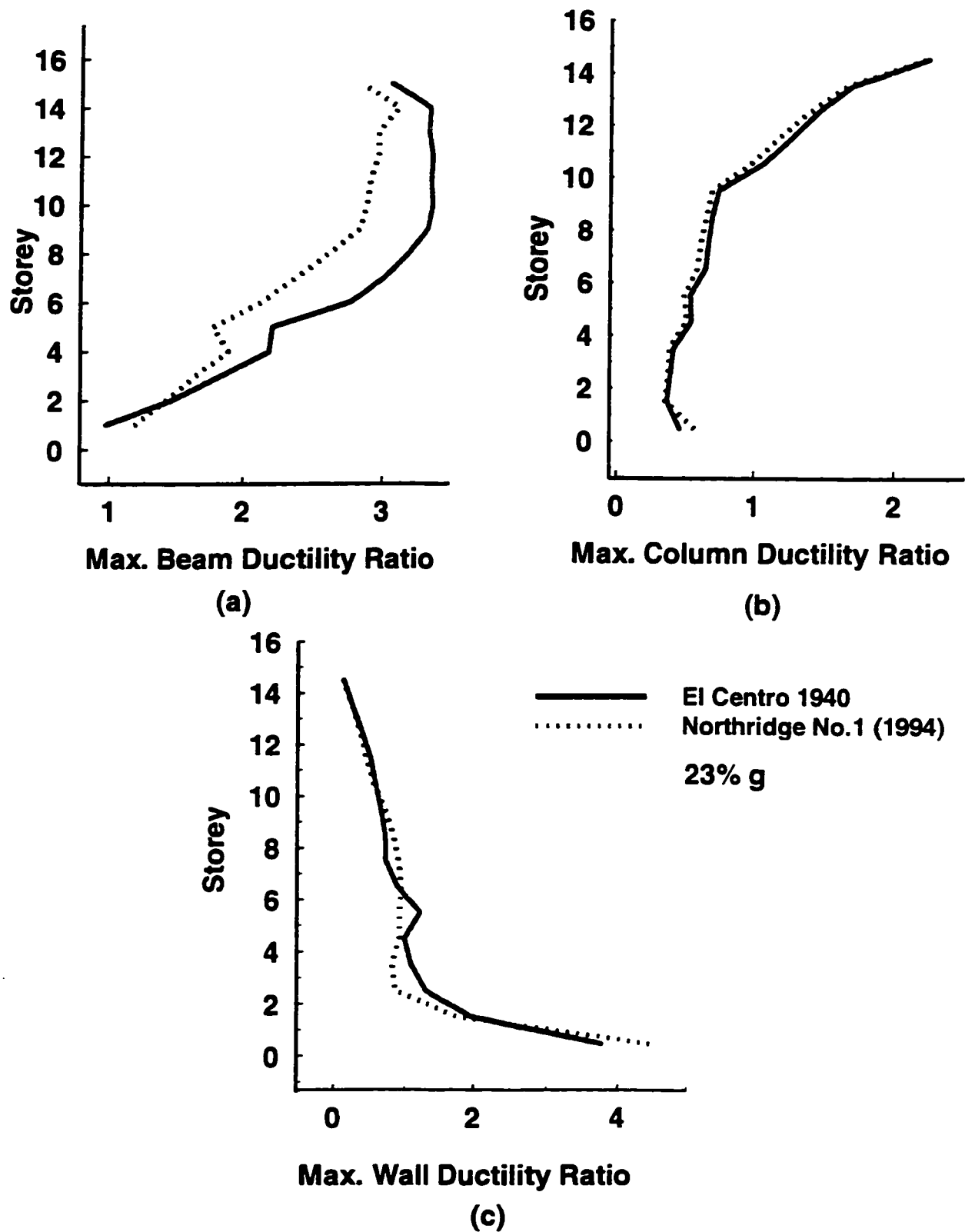


Fig. 5.55 Ductility Response for 15-Storey SHW Subjected to Actual Records for Western Canada.

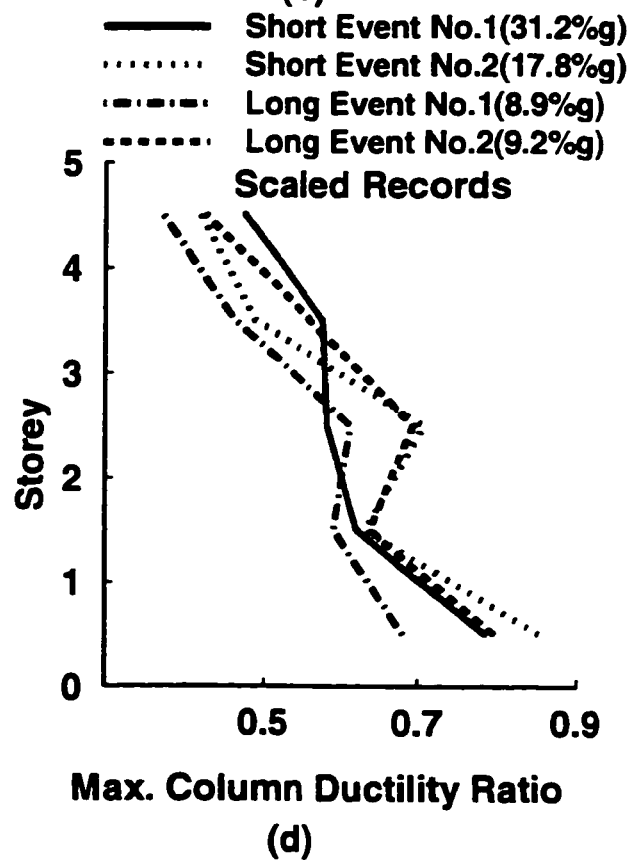
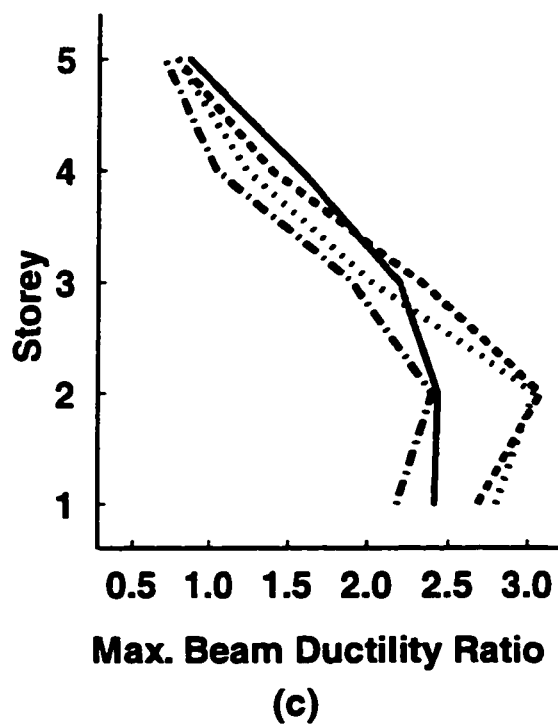
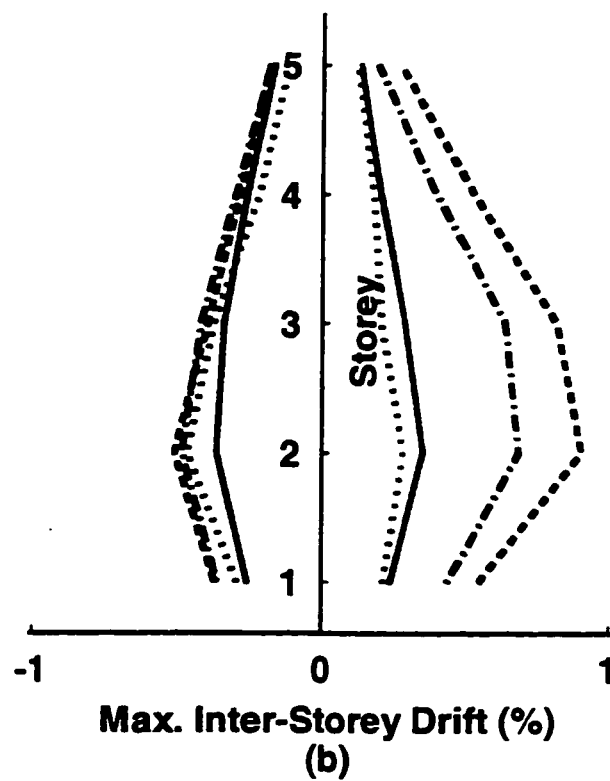
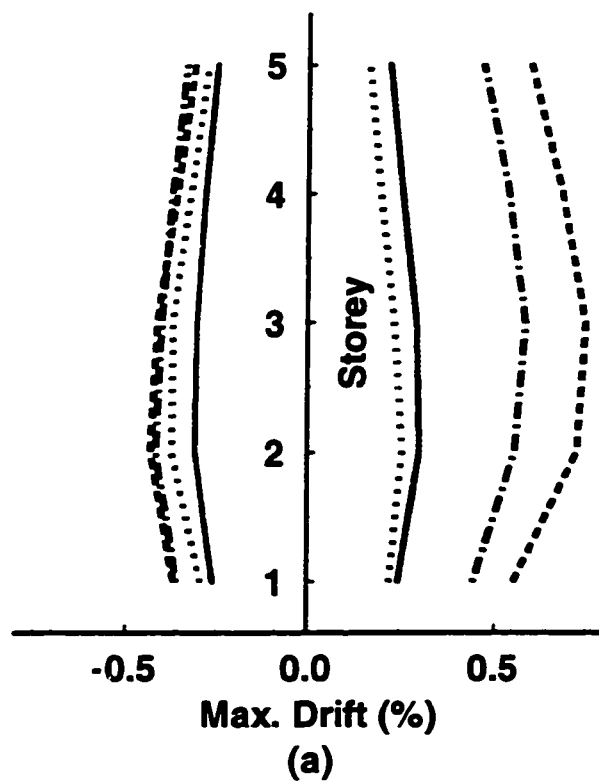
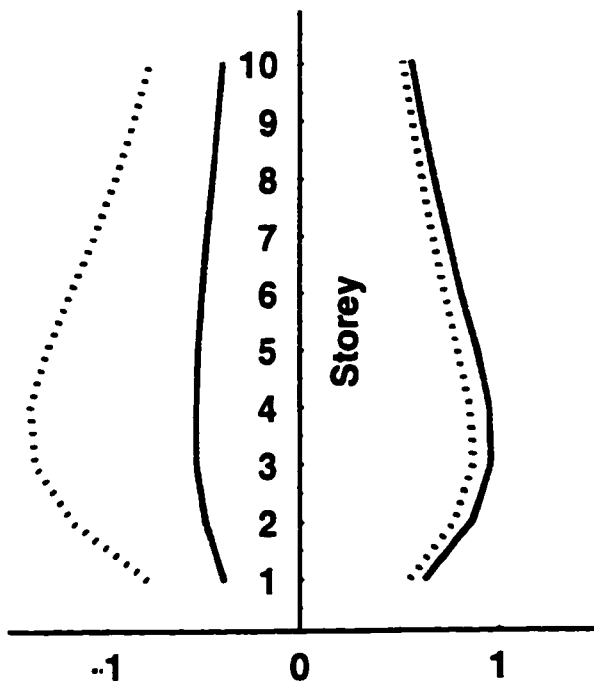
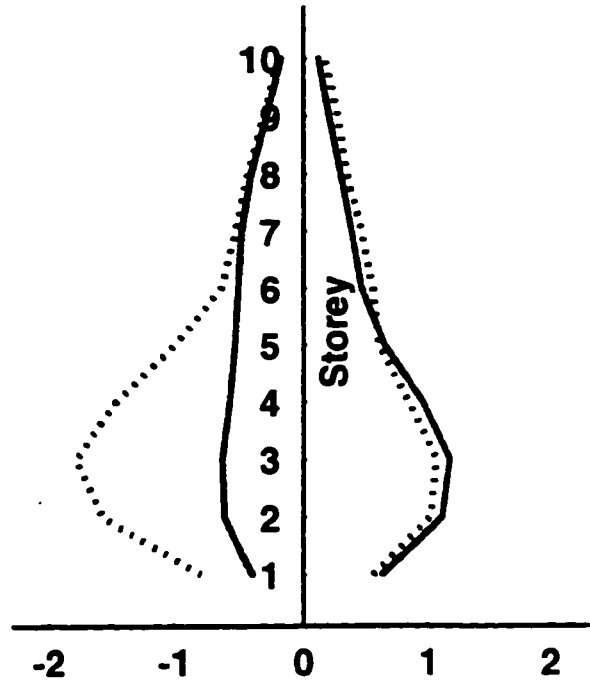


Fig. 5.56 Drift and Ductility Response for 5-Storey MRF Subjected to Scaled Artificial Records for Western Canada.



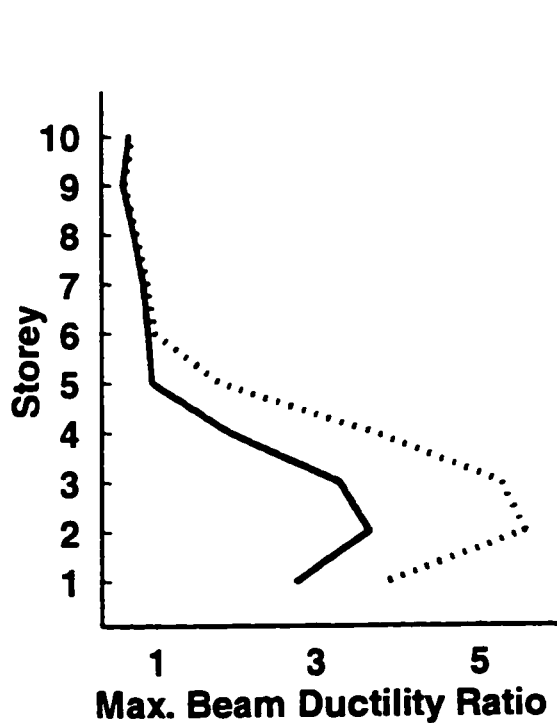
Max. Drift (%)

(a)



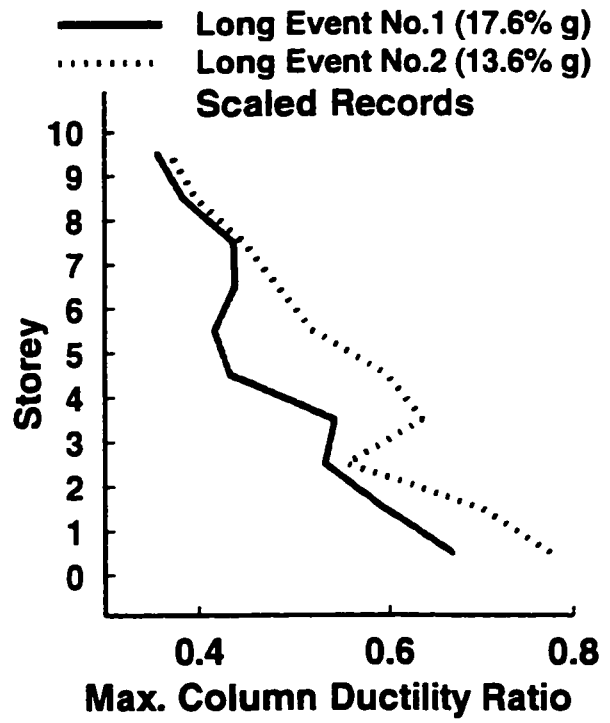
Max. Inter-Storey Drift (%)

(b)



Max. Beam Ductility Ratio

(c)



Max. Column Ductility Ratio

(d)

Fig. 5.57 Drift and Ductility Response for 10-Storey MRF Subjected to Scaled Artificial Records for Western Canada.

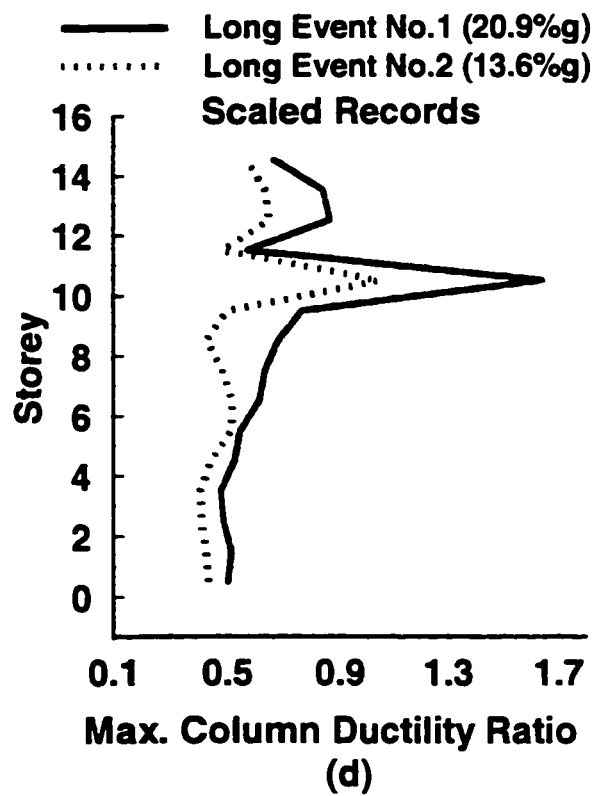
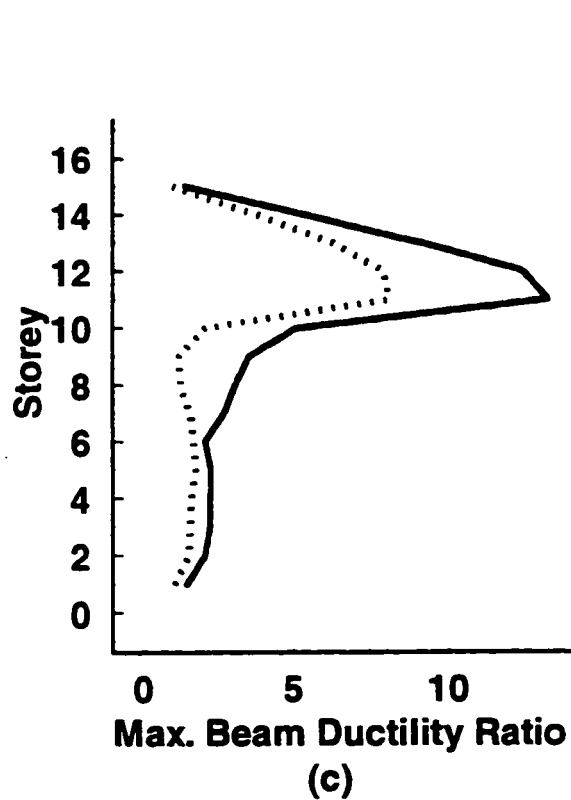
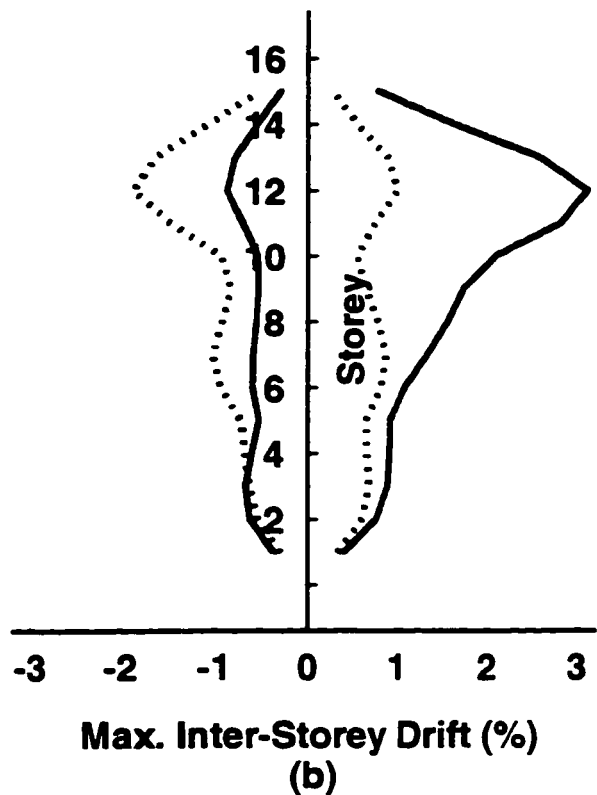
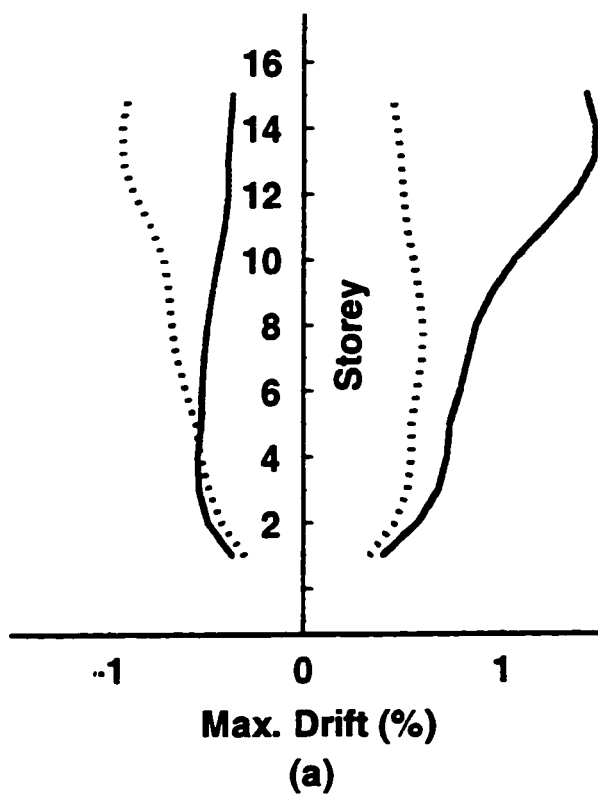


Fig. 5.58 Drift and Ductility Response for 15-Storey MRF Subjected to Scaled Artificial Records for Western Canada.

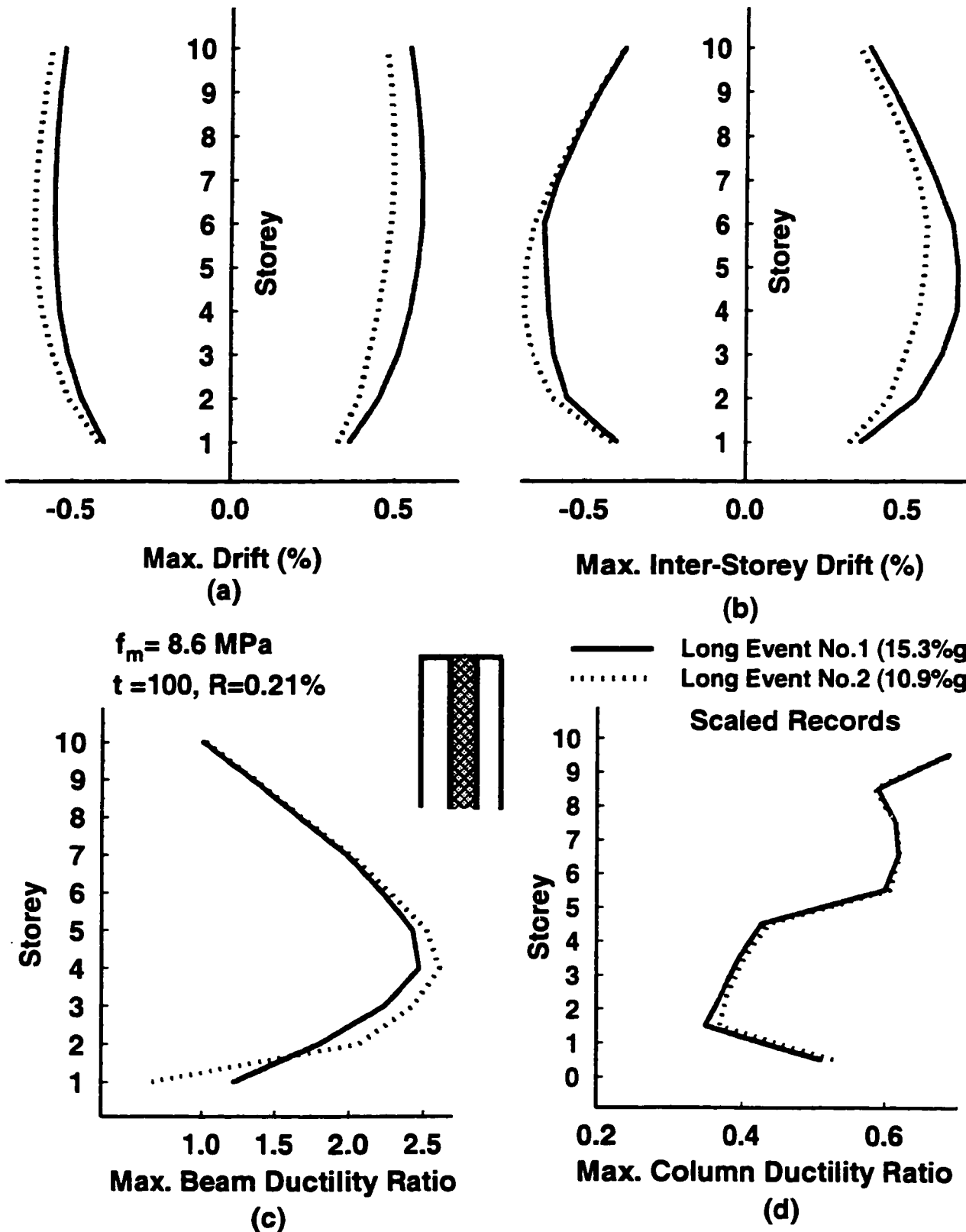


Fig. 5.59 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=100$ mm, $f_m=8.6$ MPa).

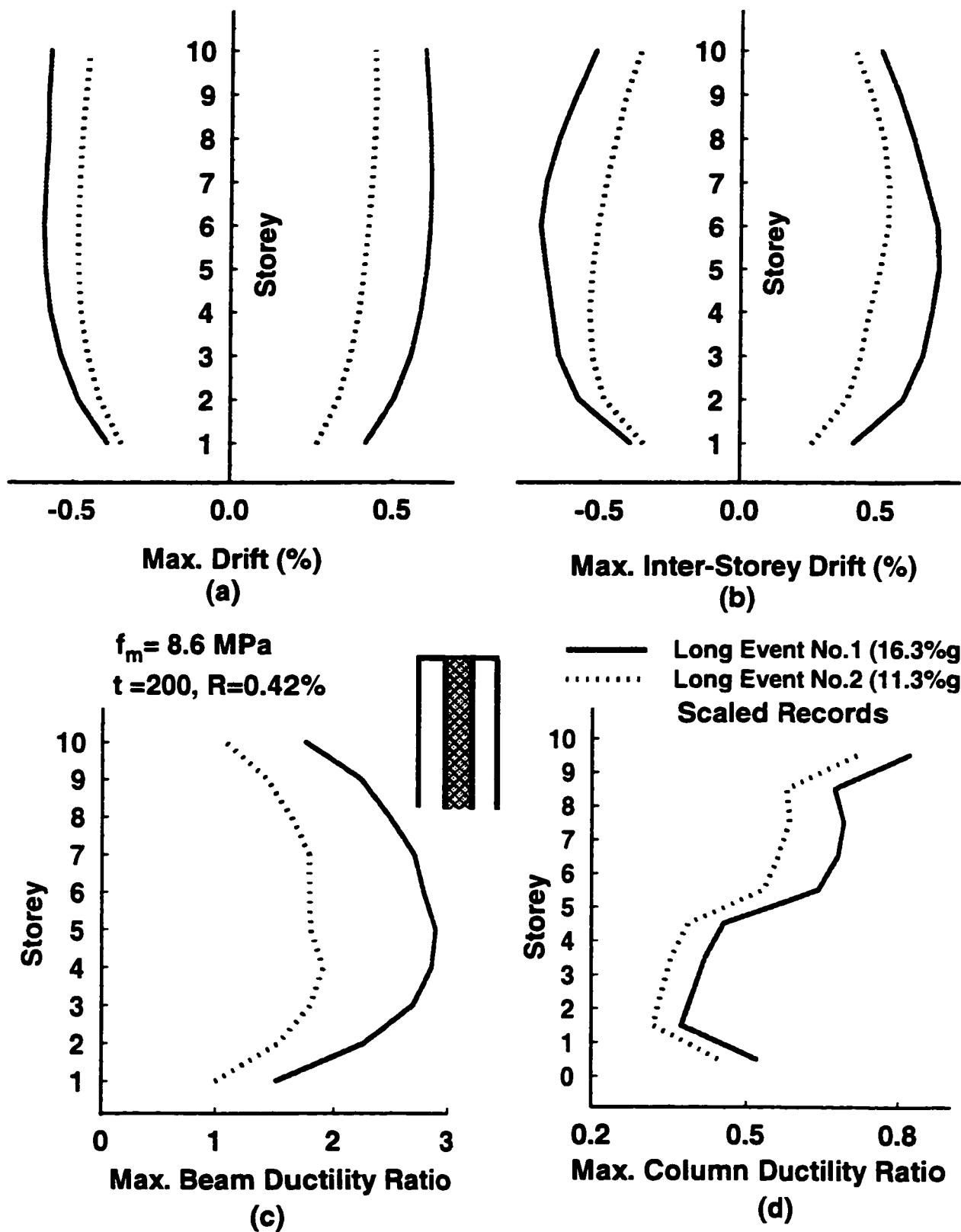


Fig. 5.60 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

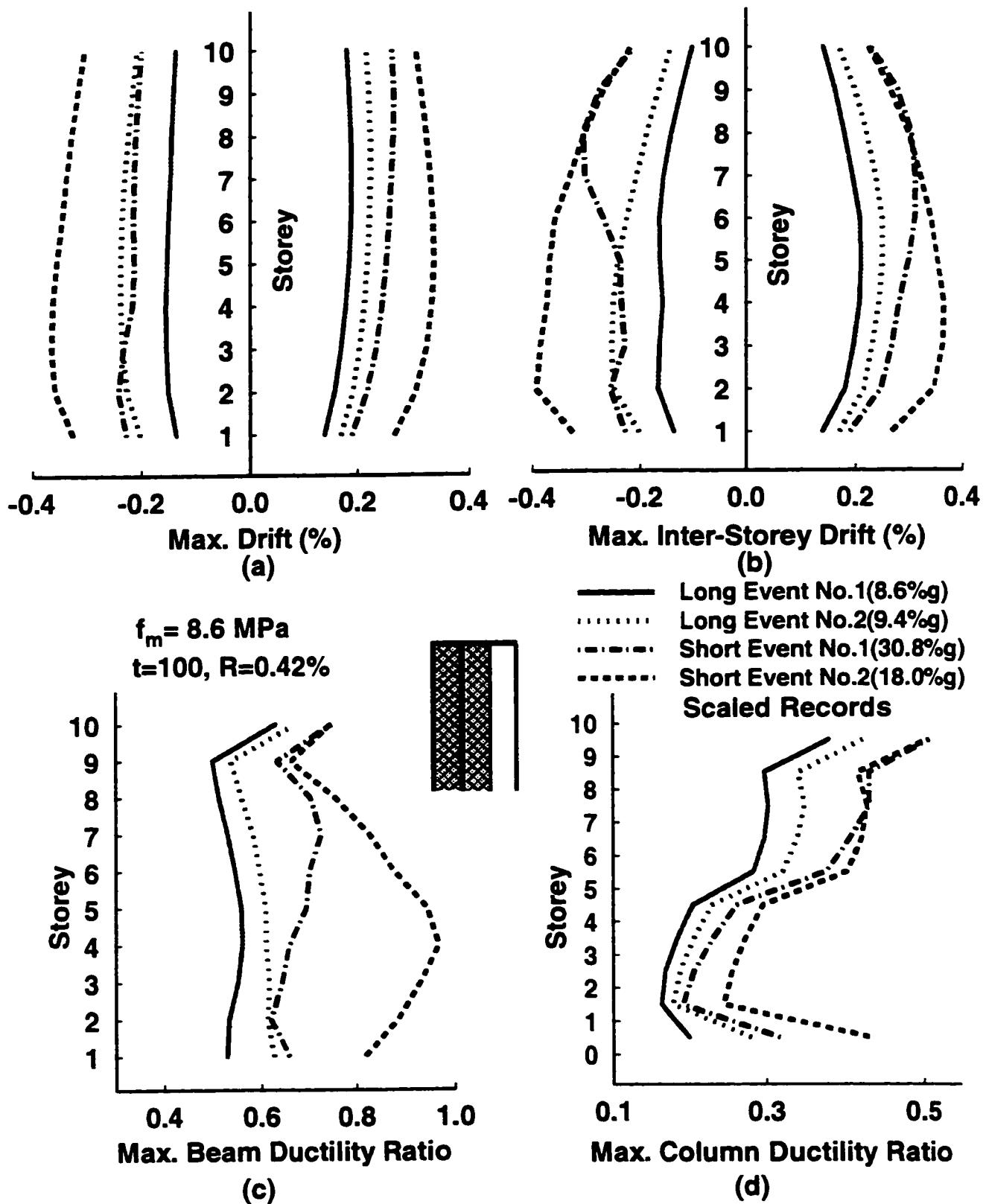


Fig. 5.61 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Two bays, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

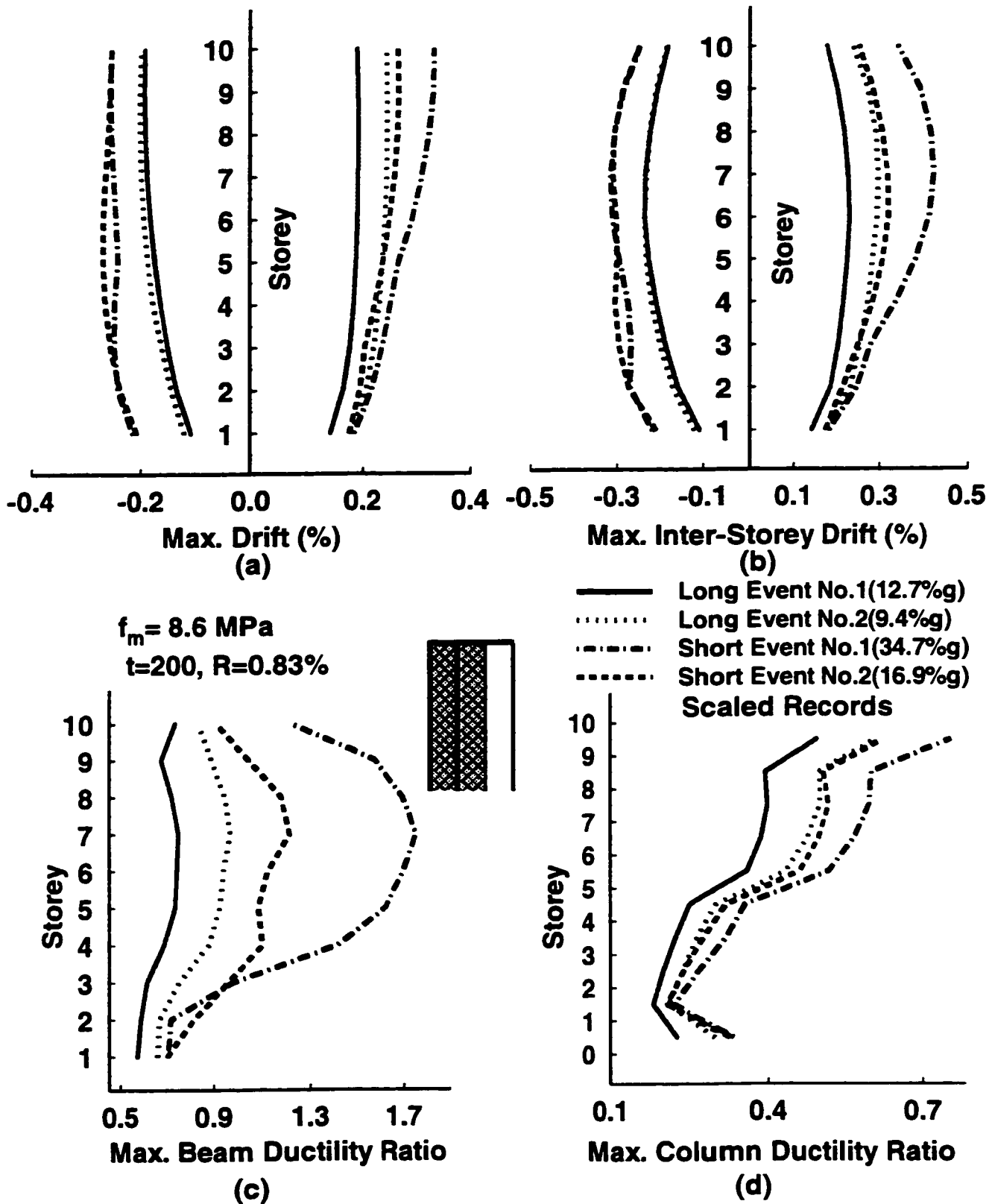


Fig. 5.62 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Two bays, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

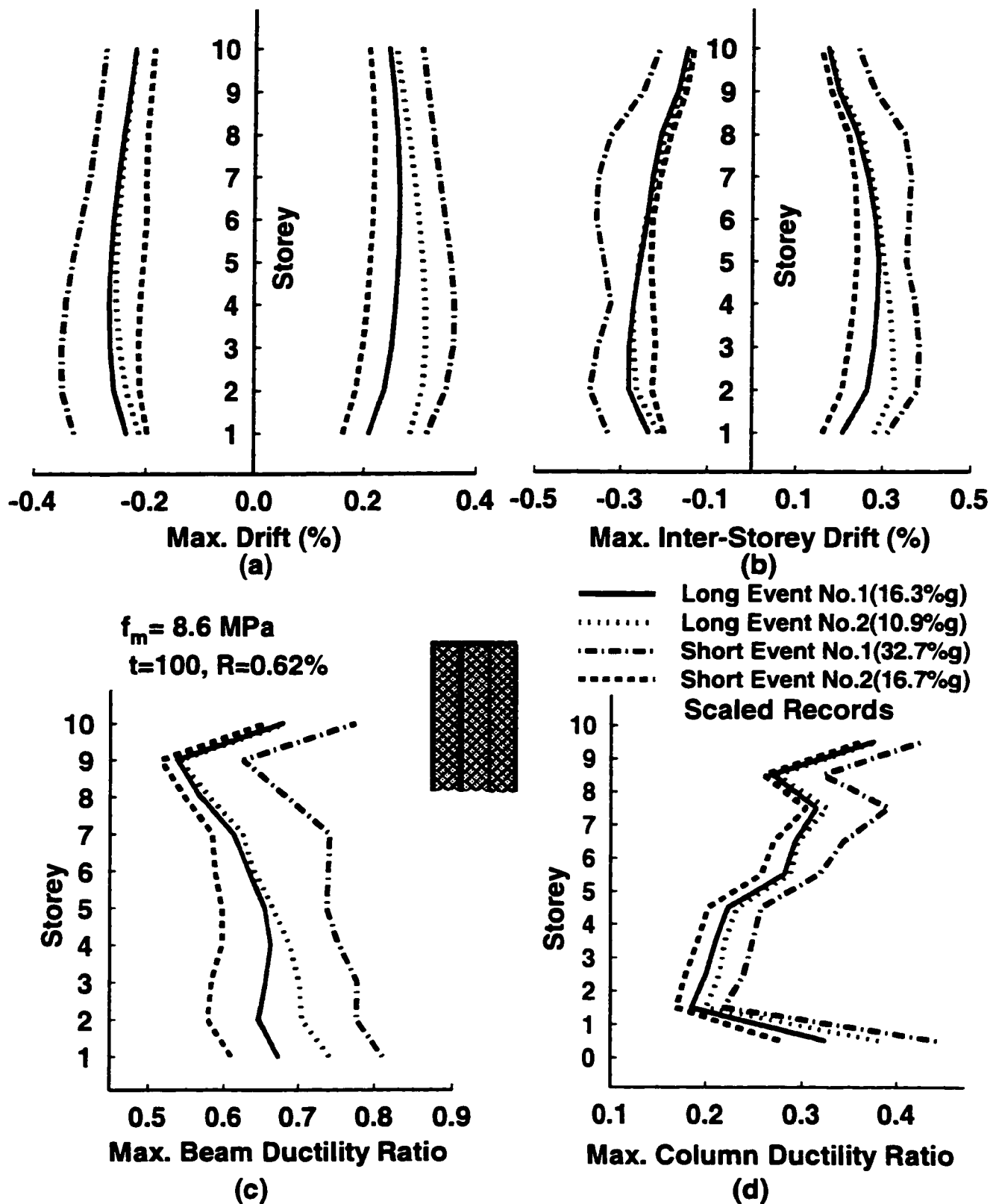


Fig. 5.63 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Three bays, $t=100 \text{ mm}$, $f_m=8.6 \text{ MPa}$).

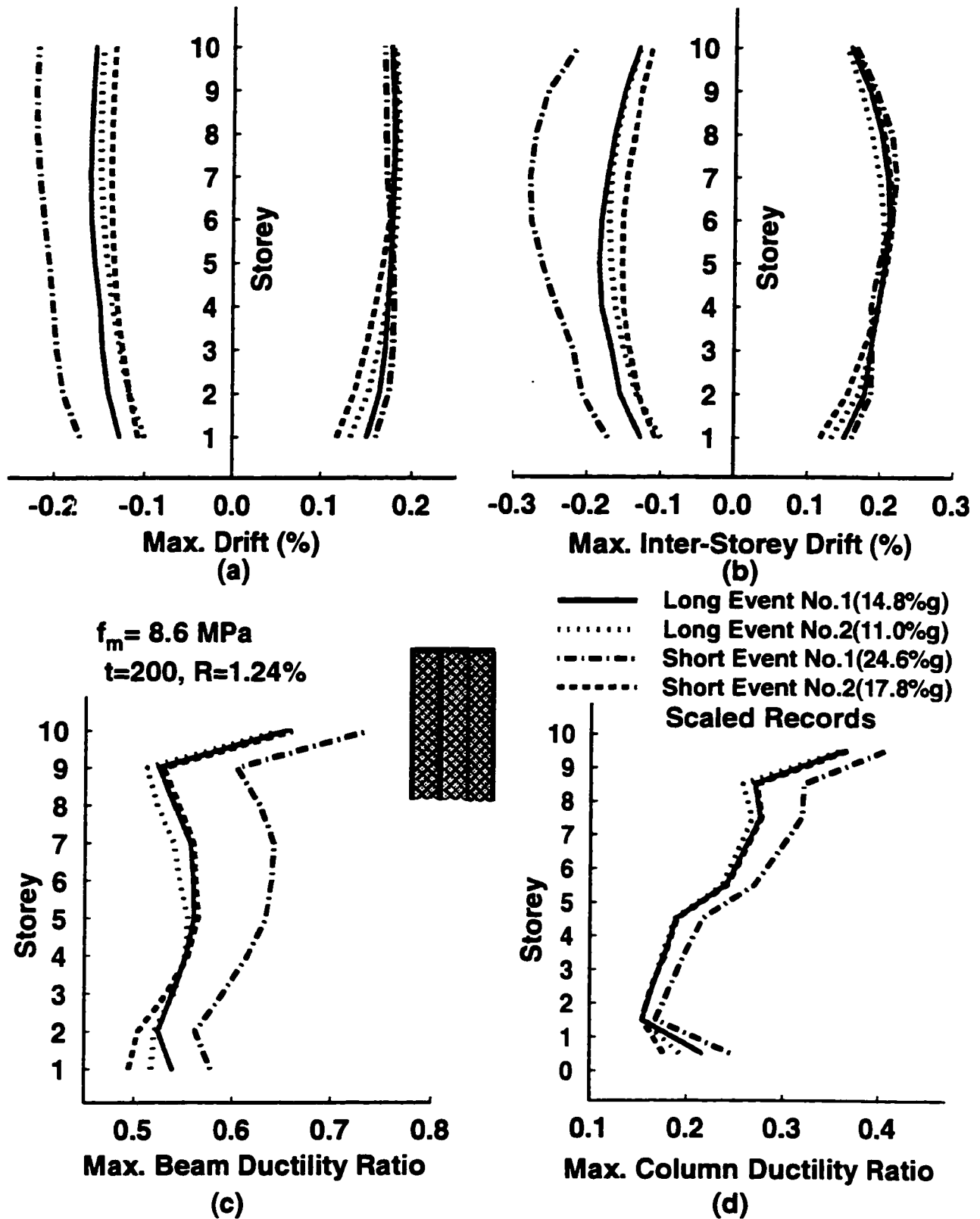
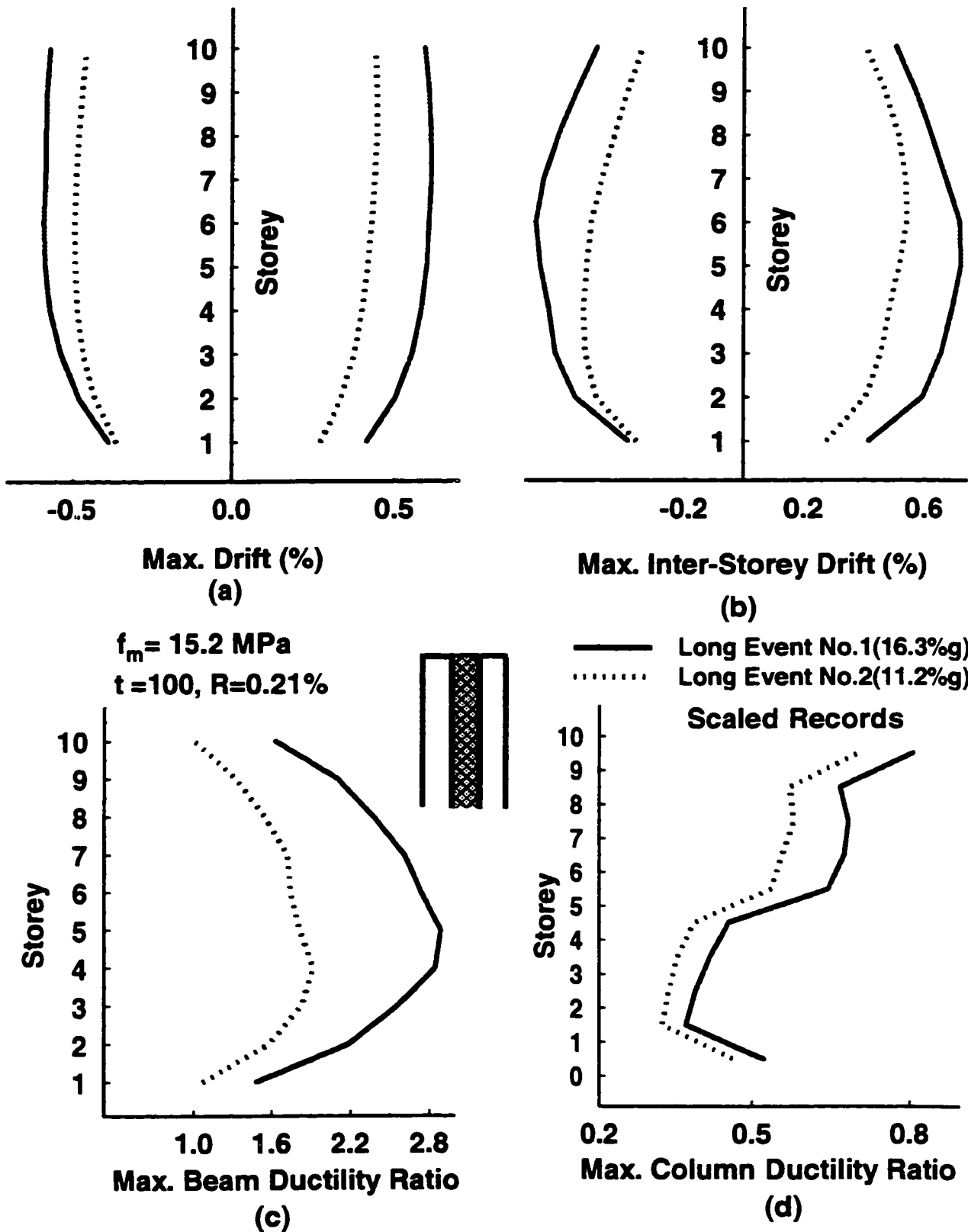


Fig. 5.64 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Three bays, $t=200 \text{ mm}$, $f_m=8.6 \text{ MPa}$).



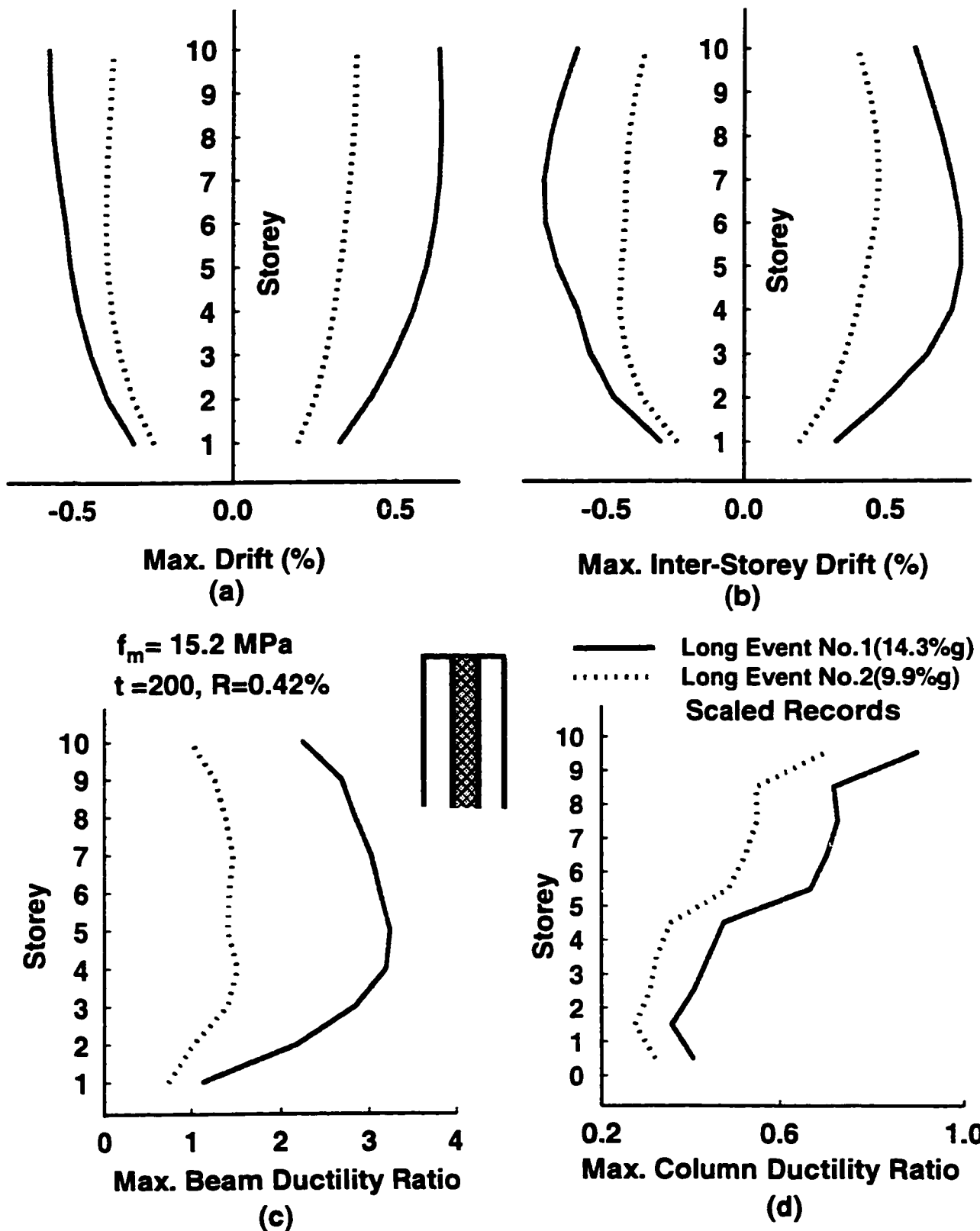


Fig. 5.66 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (One bay, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

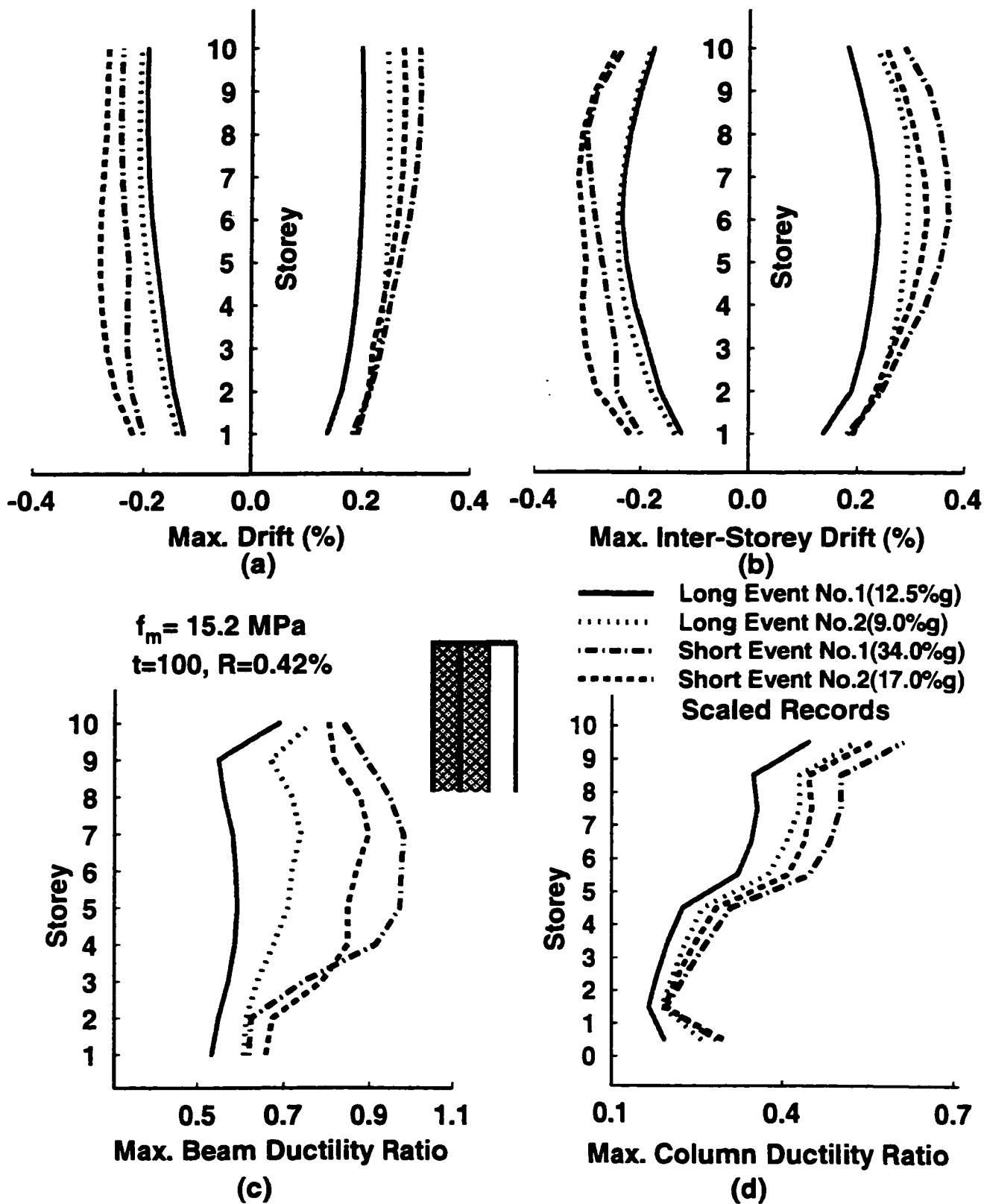


Fig. 5.67 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Two bays, $t=100 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

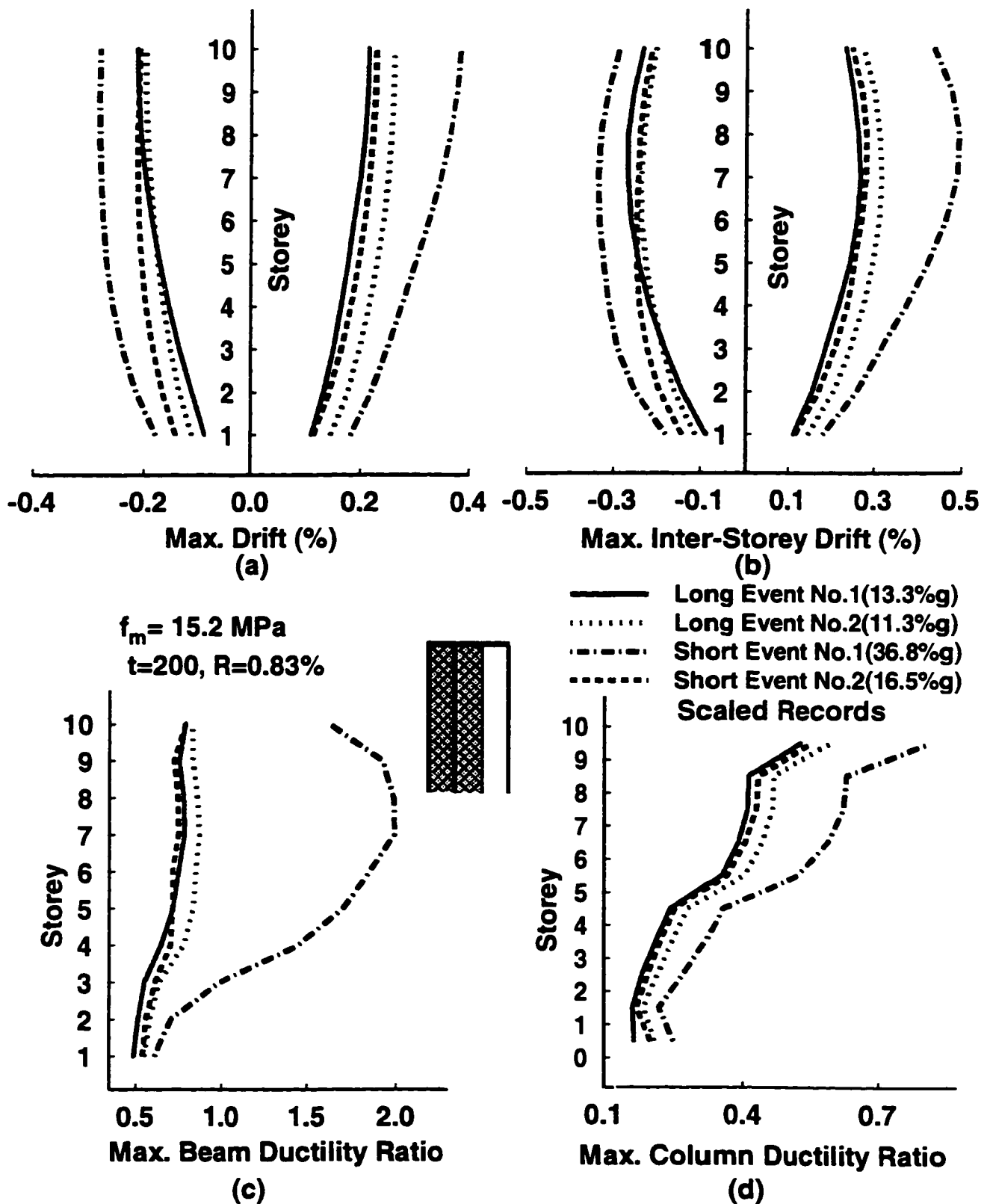


Fig. 5.68 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Two bays, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

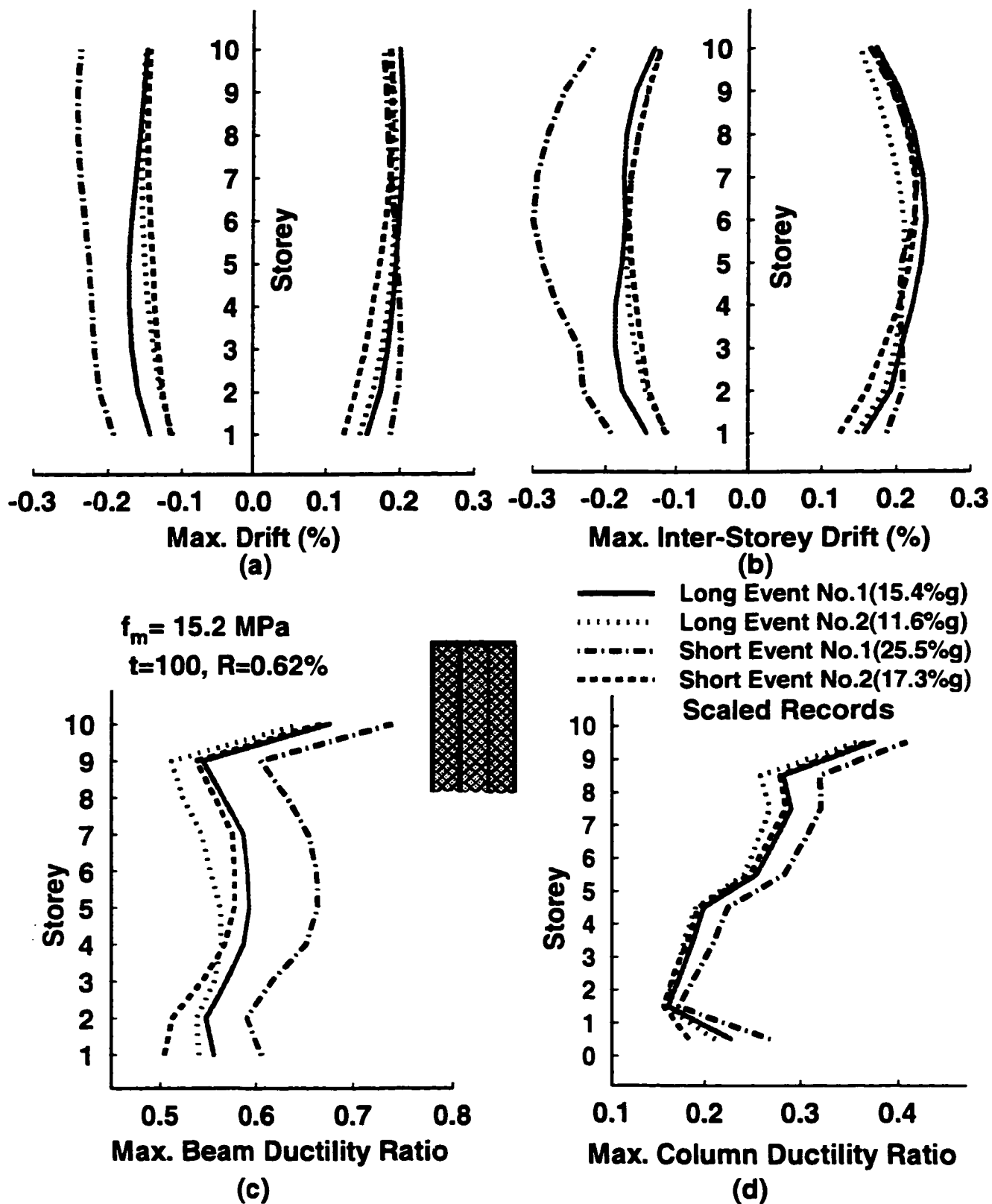


Fig. 5.69 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Scaled Artificial Records (Three bays, $t=100 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

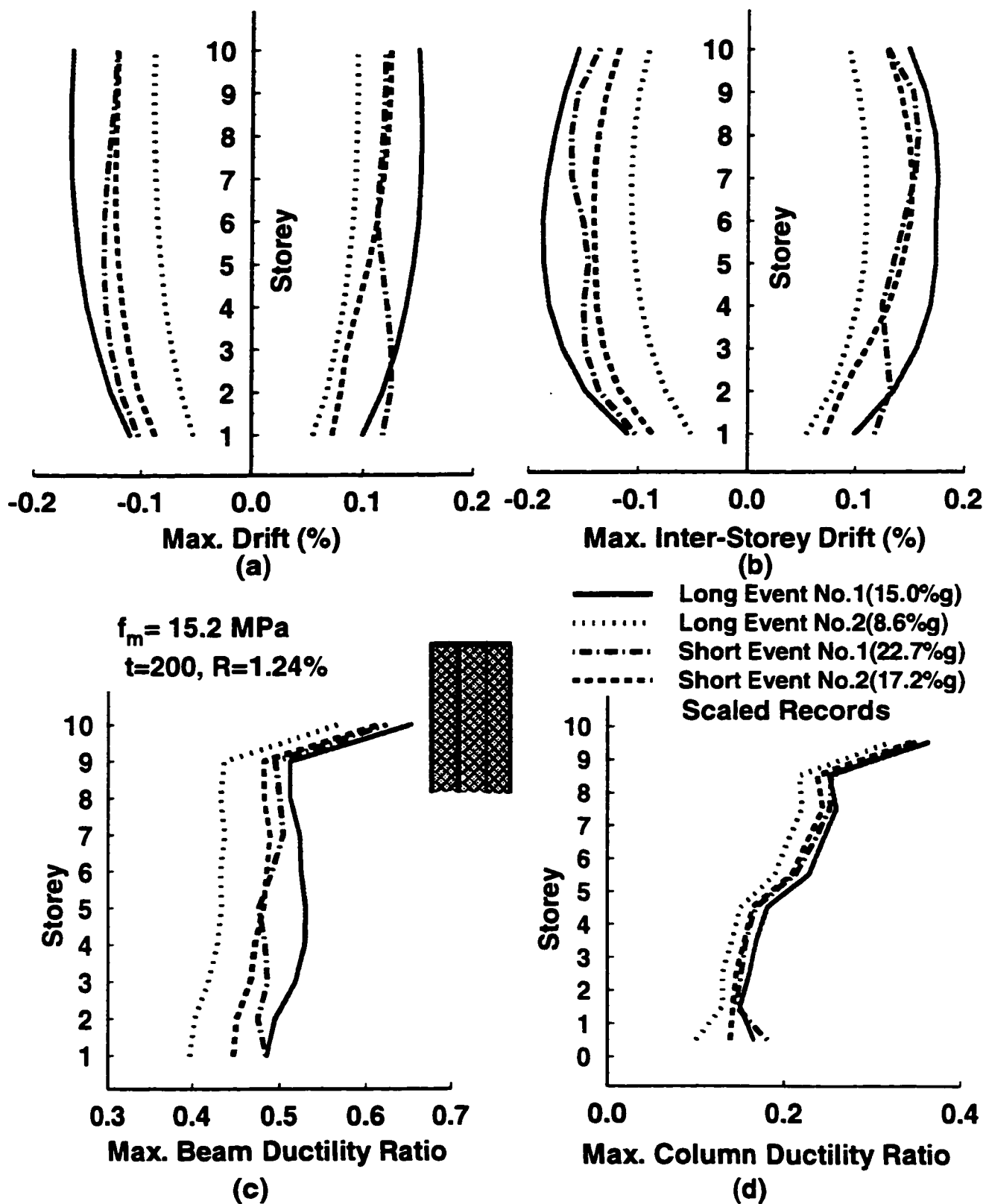


Fig. 5.70 Drift and Ductility Response for 10-Storey Infilled Frame Subjected to Artificial Records (Three bays, $t=200 \text{ mm}$, $f_m=15.2 \text{ MPa}$).

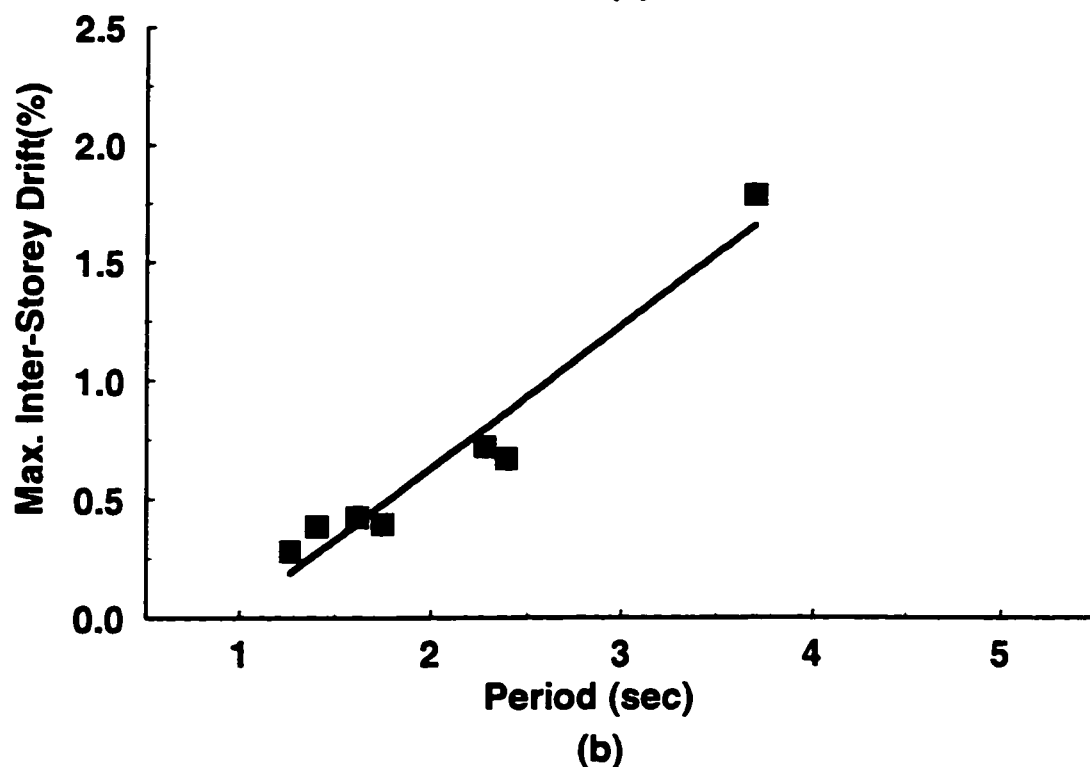
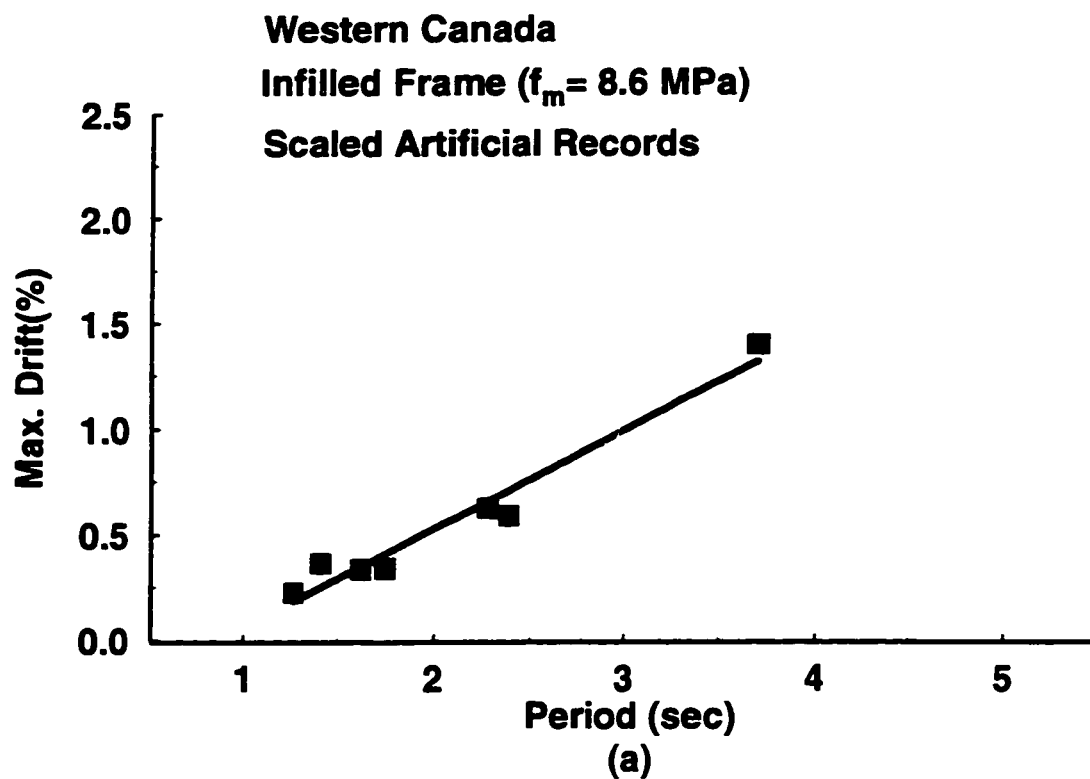


Fig. 5.71 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Scaled Artificial Records ($f_m=8.6$ MPa).

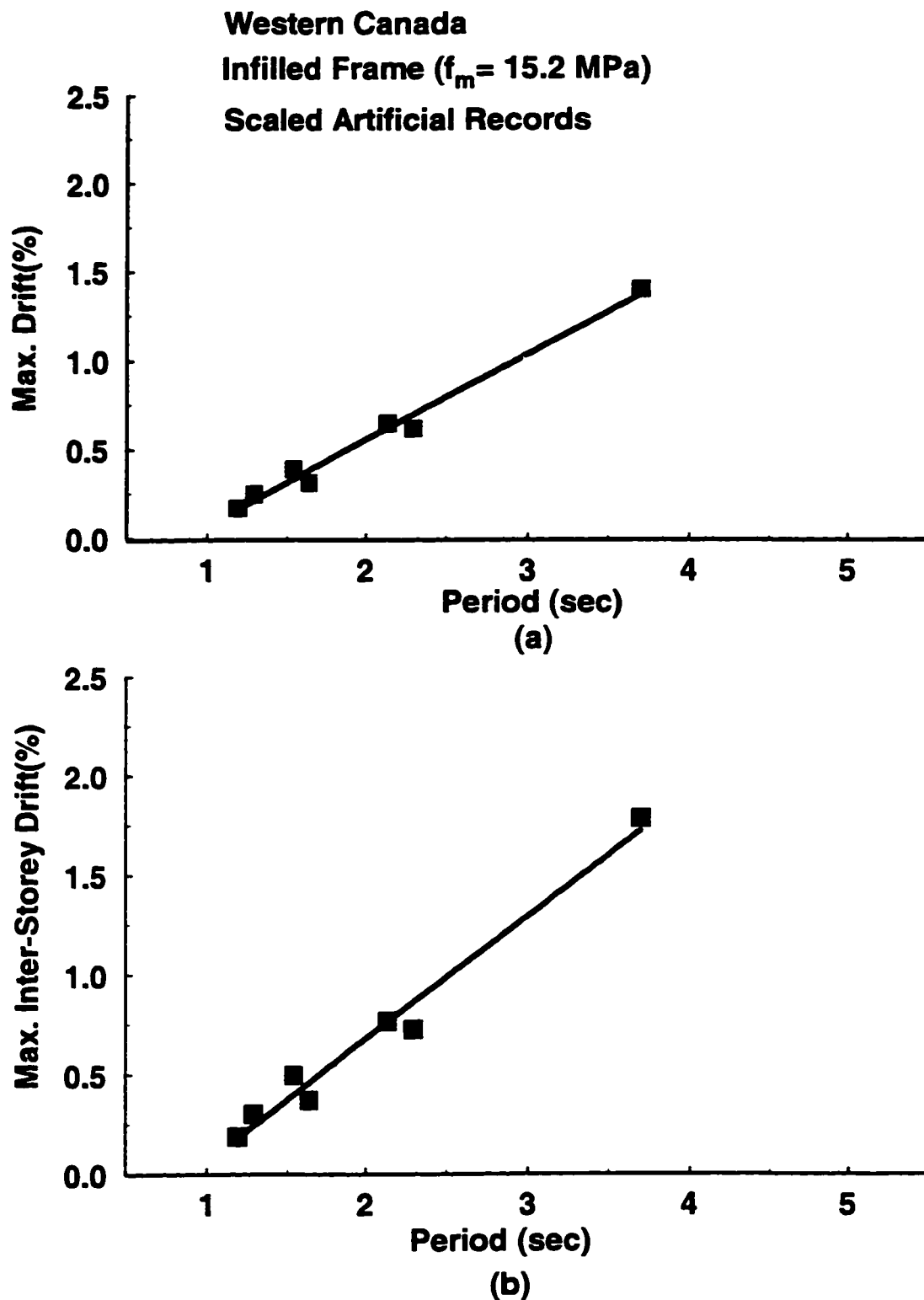
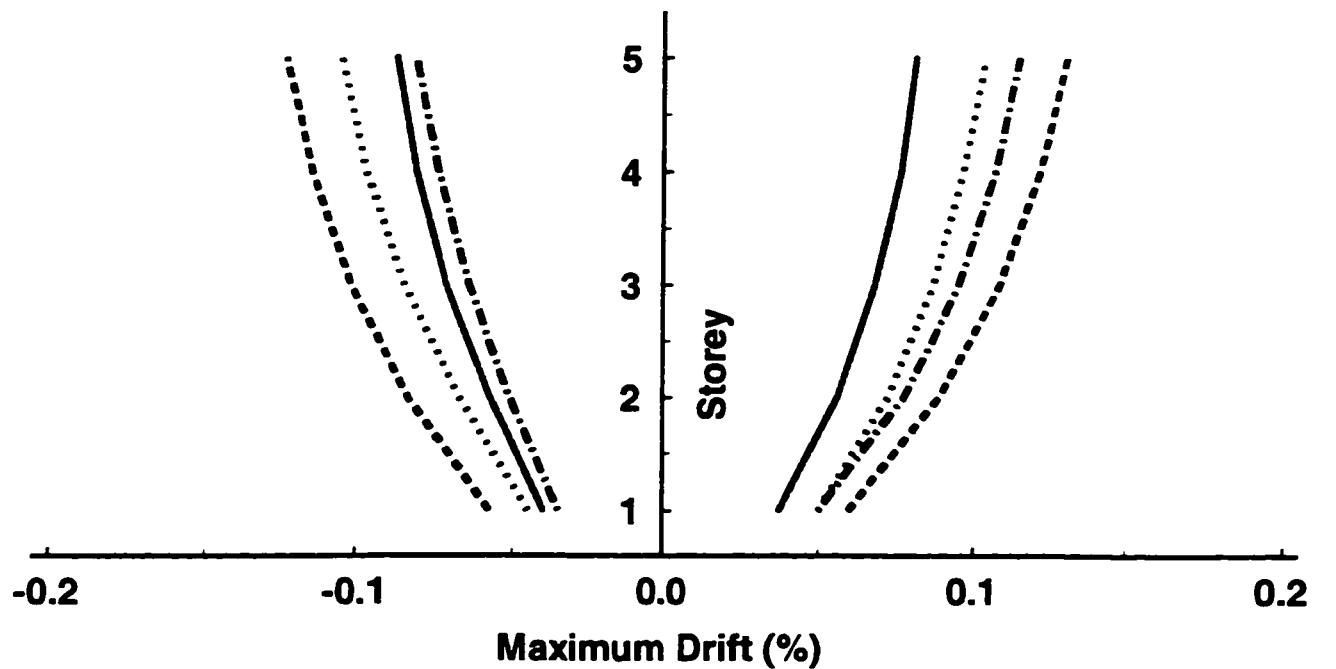
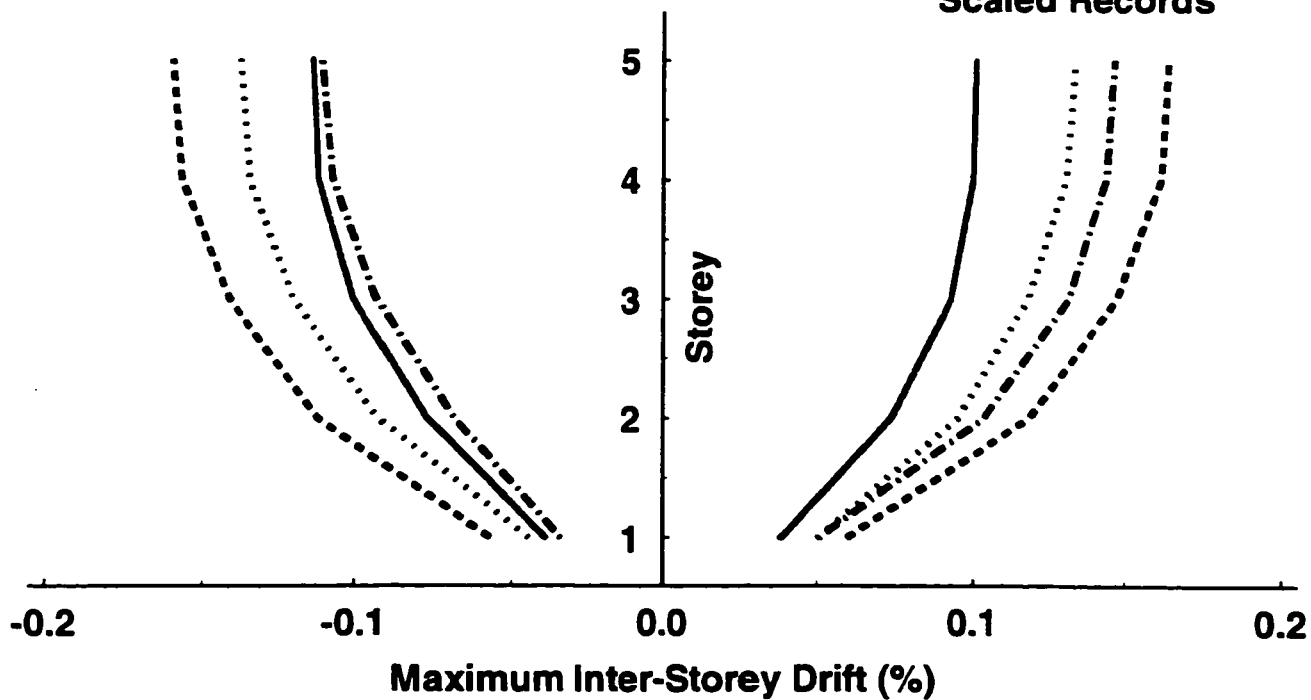


Fig. 5.72 The Relationship Between Maximum Drift and The Fundamental Period of Infilled Frames Subjected to Scaled Artificial Records ($f_m=15.2$ MPa).



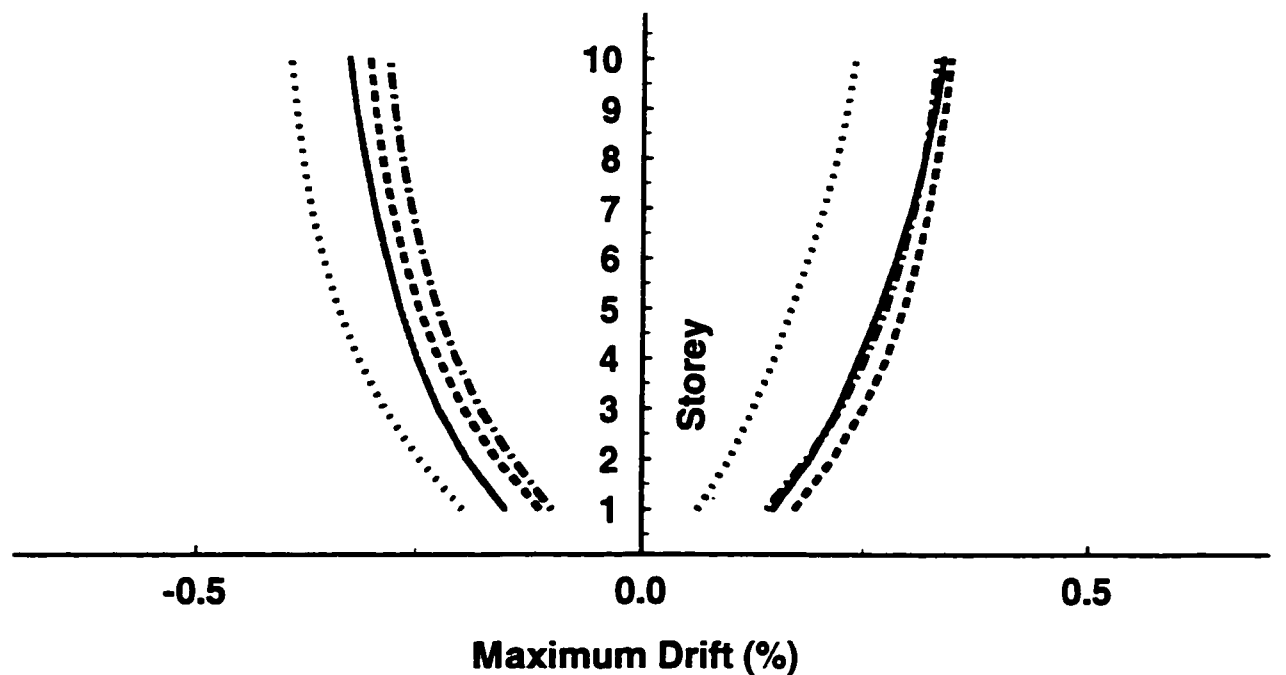
(a)

— Short Event No.1(11.5%g)
 Short Event No.2(18.9%g)
 -.-.- Long Event No.1 (11.7%g)
 ---- Long Event No.2 (16.2%g)
Scaled Records



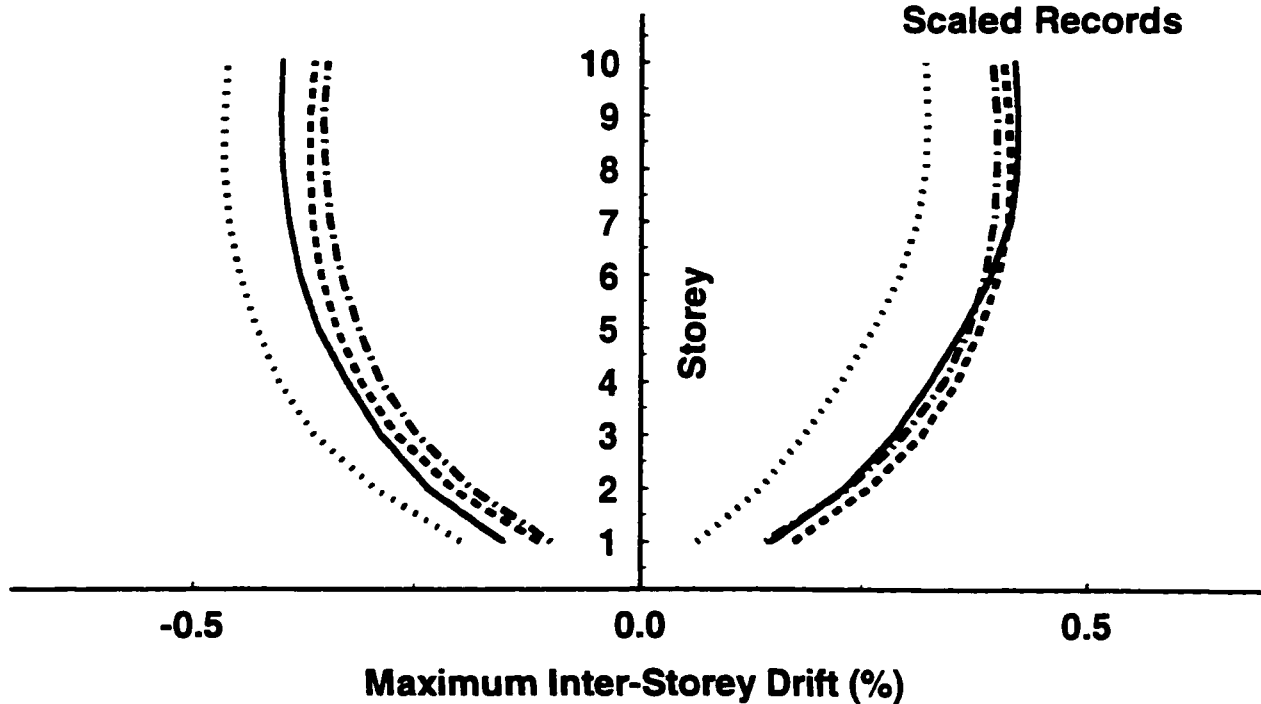
(b)

Fig. 5.73 Drift Response for 5-Storey SHW Subjected to Scaled Artificial Records for Western Canada.



(a)

- Short Event No.1(30.4%g)
 - Short Event No.2(18.2%g)
 - .-.- Long Event No.1 (8.4%g)
 - Long Event No.2 (9.6%g)
- Scaled Records**



(b)

Fig. 5.74 Drift Response for 10-Storey SHW Subjected to Scaled Artificial Records for Western Canada.

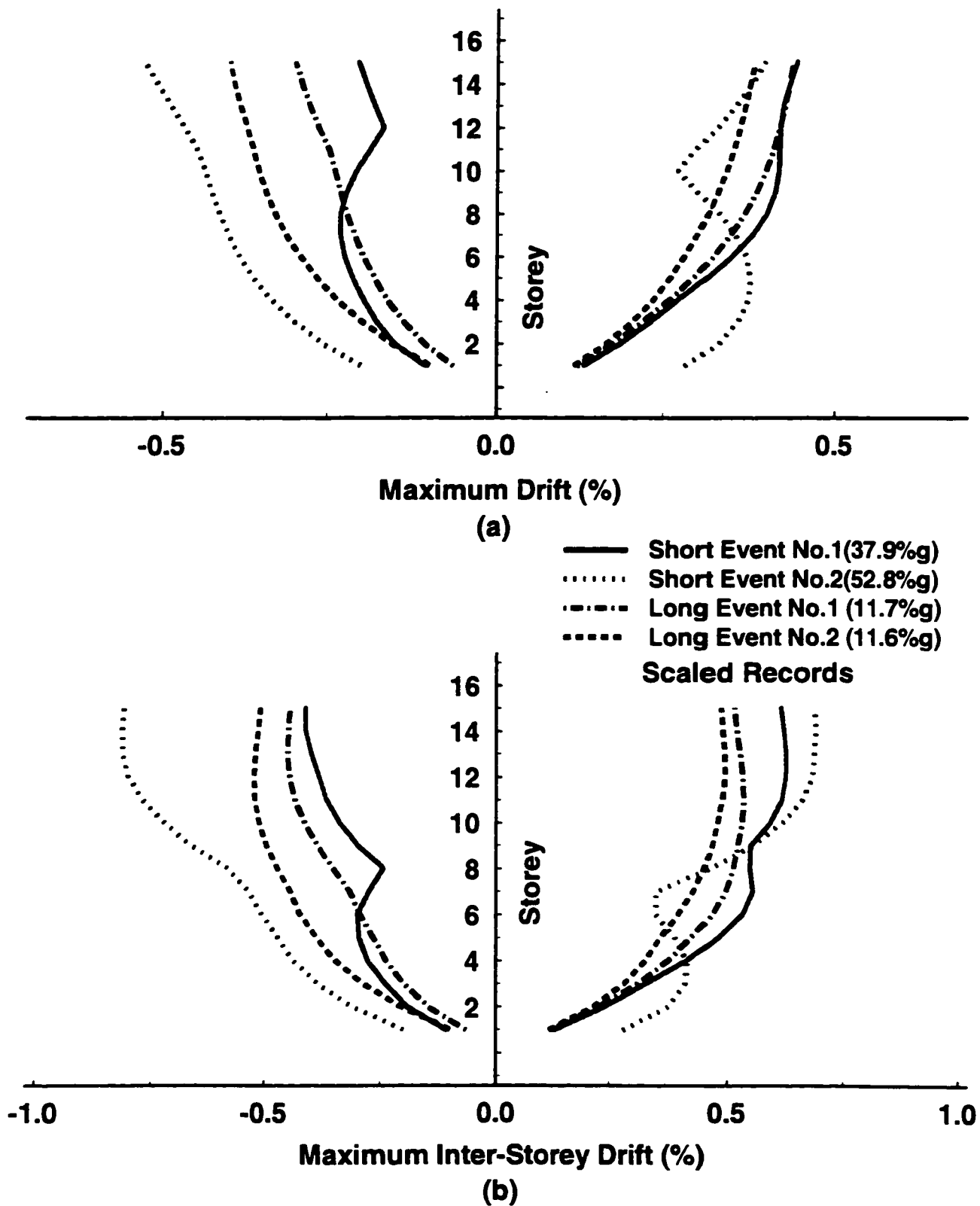


Fig. 5.75 Drift Response for 15-Storey SHW Subjected to Scaled Artificial Records for Western Canada.

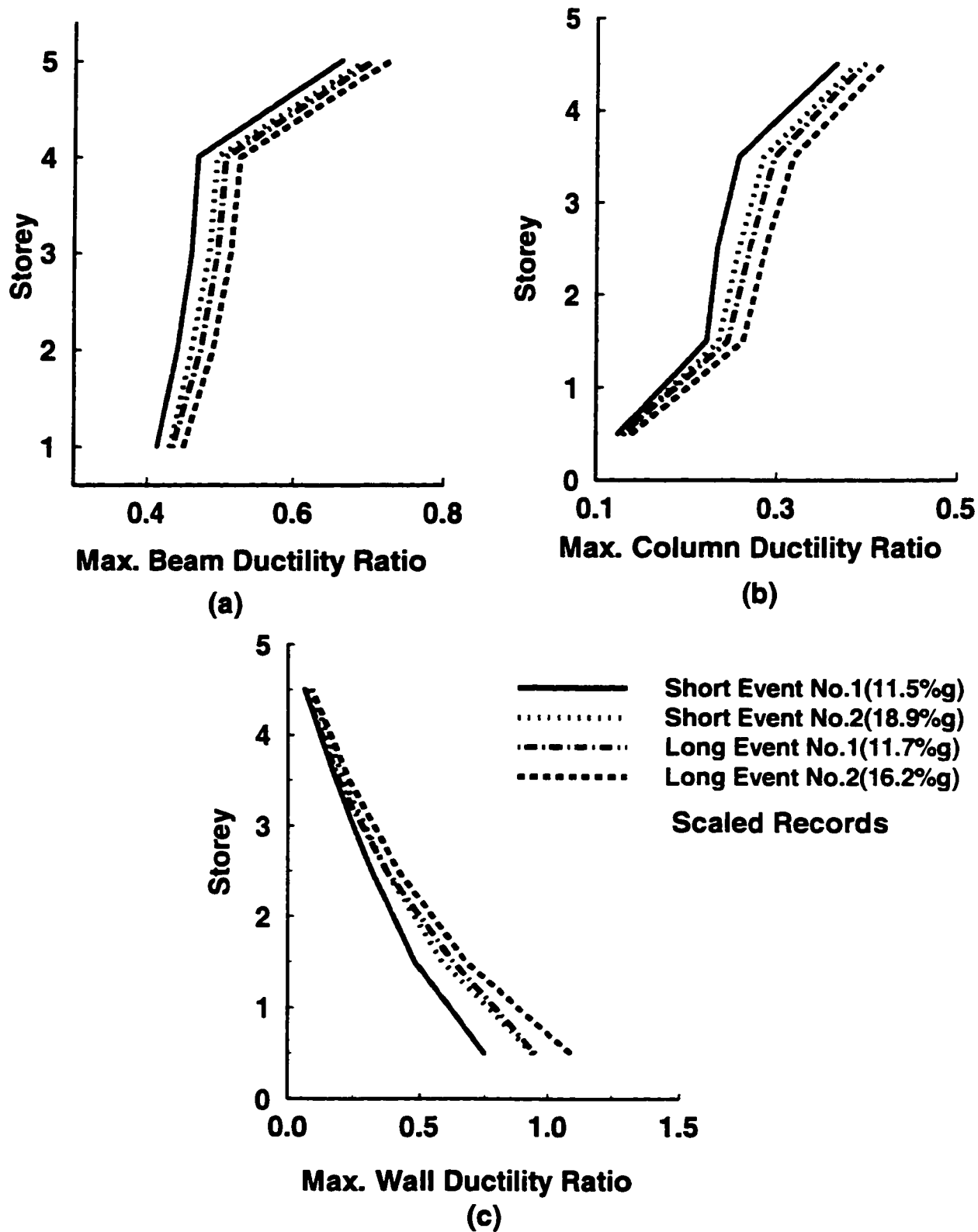


Fig. 5.76 Ductility Response for 5-Storey SHW Subjected to Scaled Artificial Records for Western Canada.

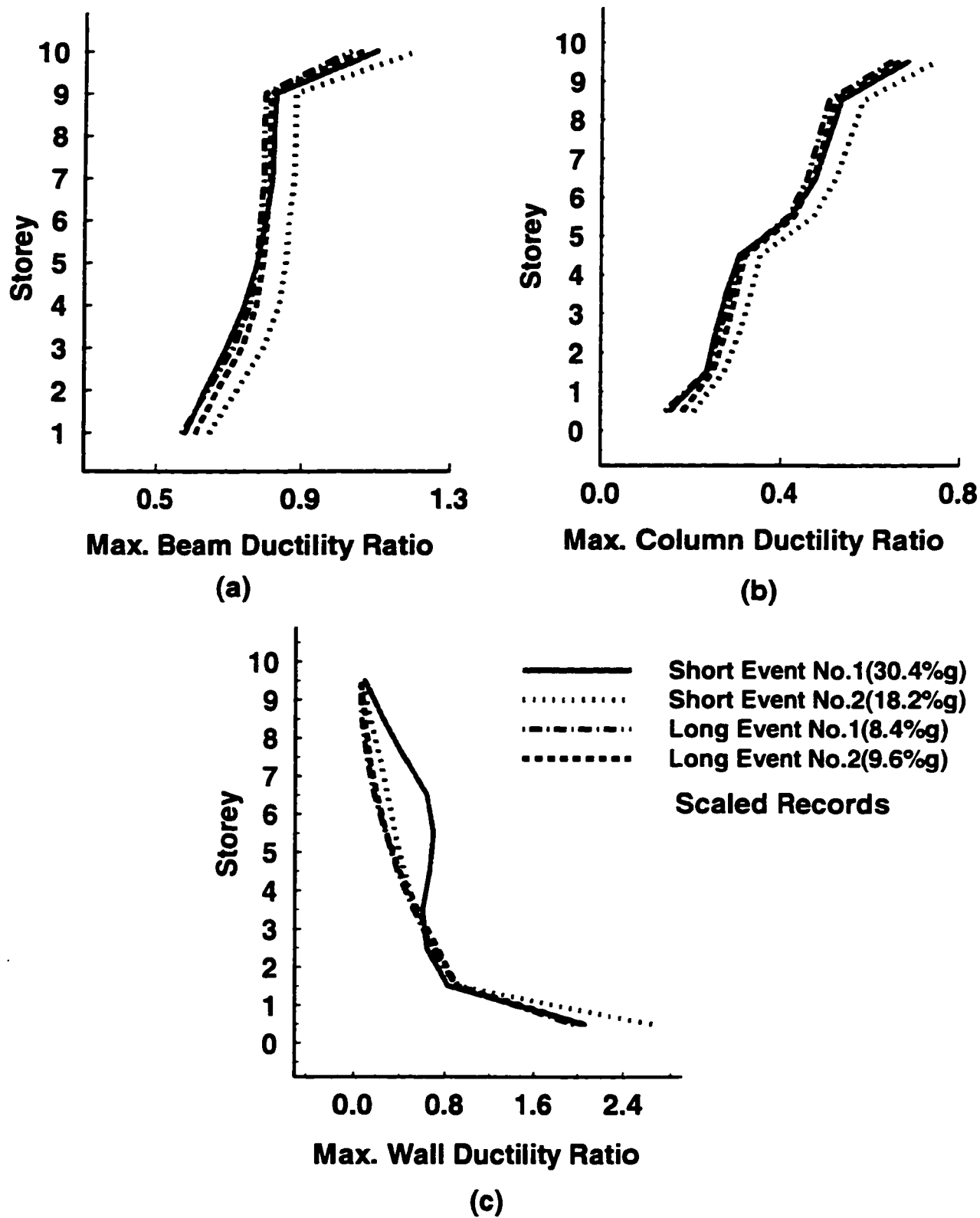


Fig. 5.77 Ductility Response for 10-Storey SHW Subjected to Scaled Artificial Records for Western Canada.

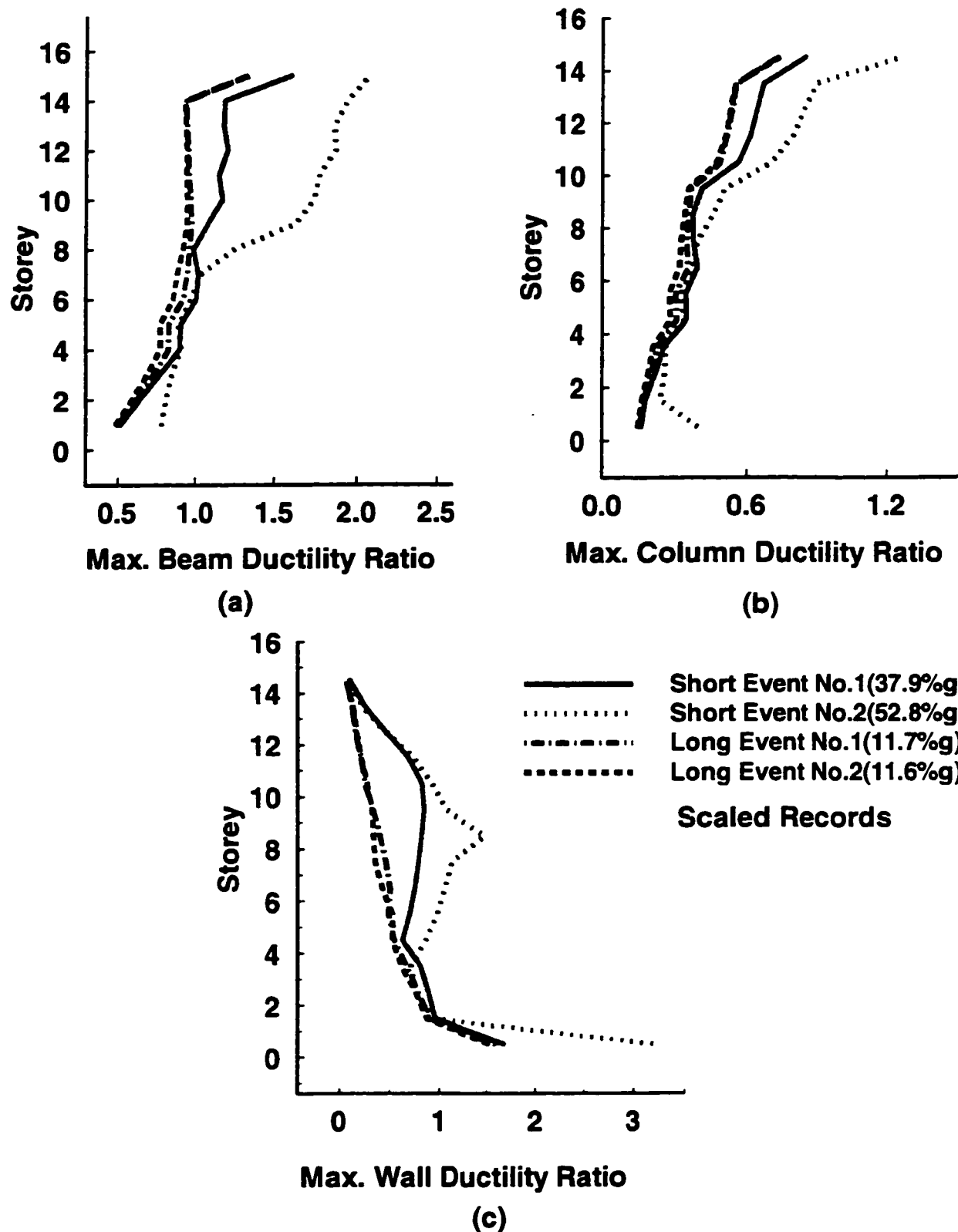


Fig. 5.78 Ductility Response for 15-Storey SHW Subjected to Scaled Artificial Records for Western Canada.

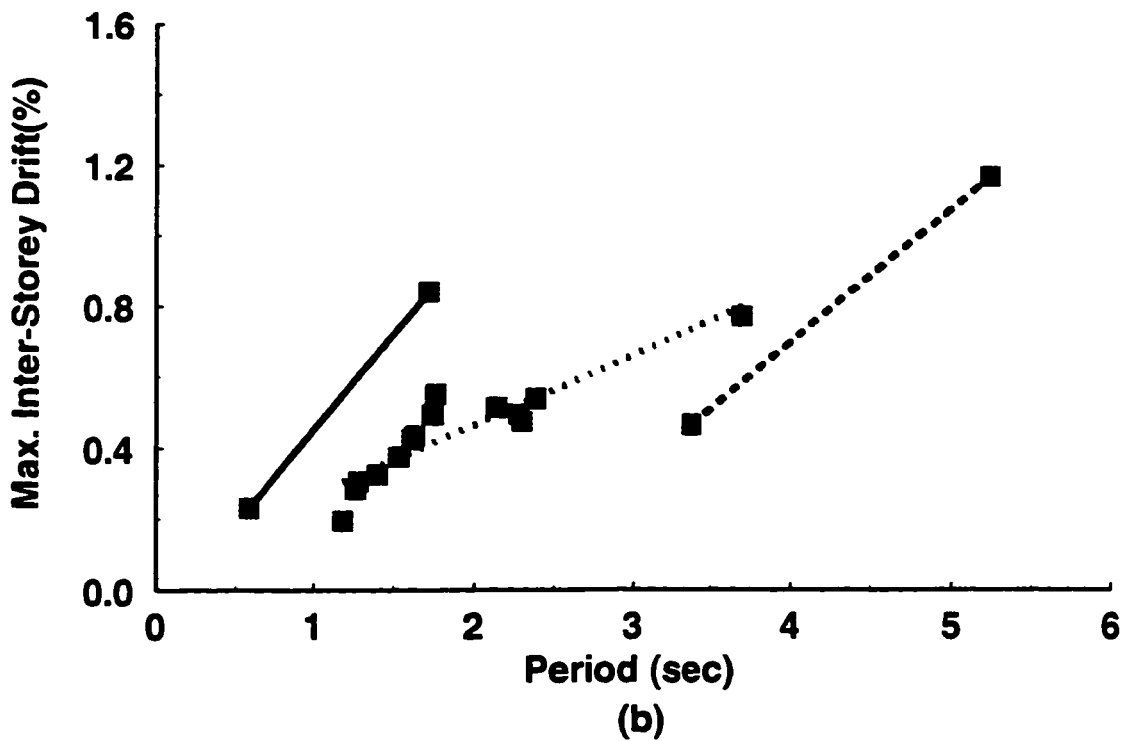
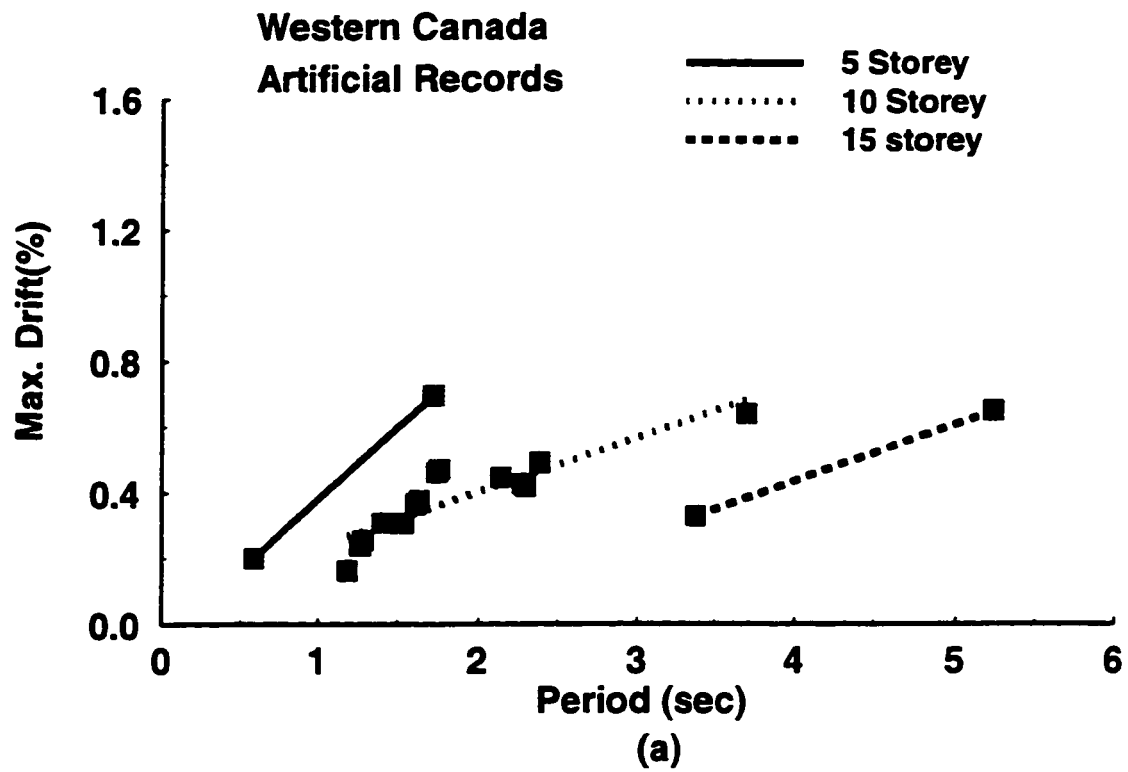


Fig. 5.79 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Artificial Records for Western Canada.

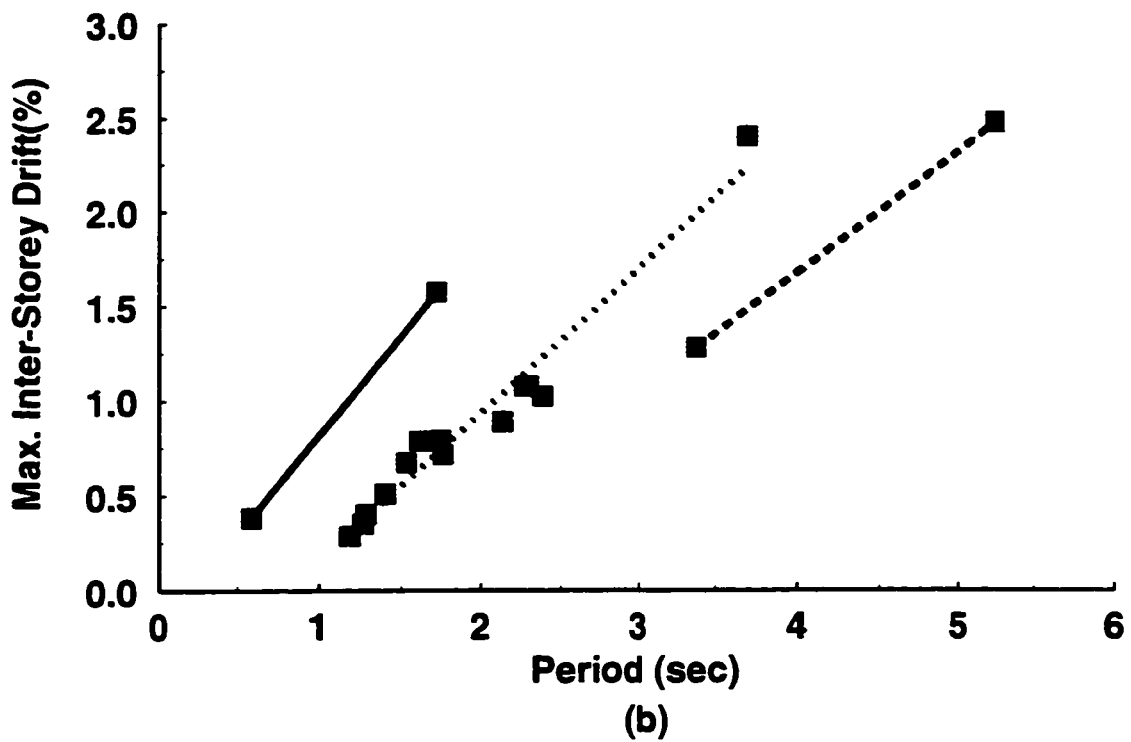
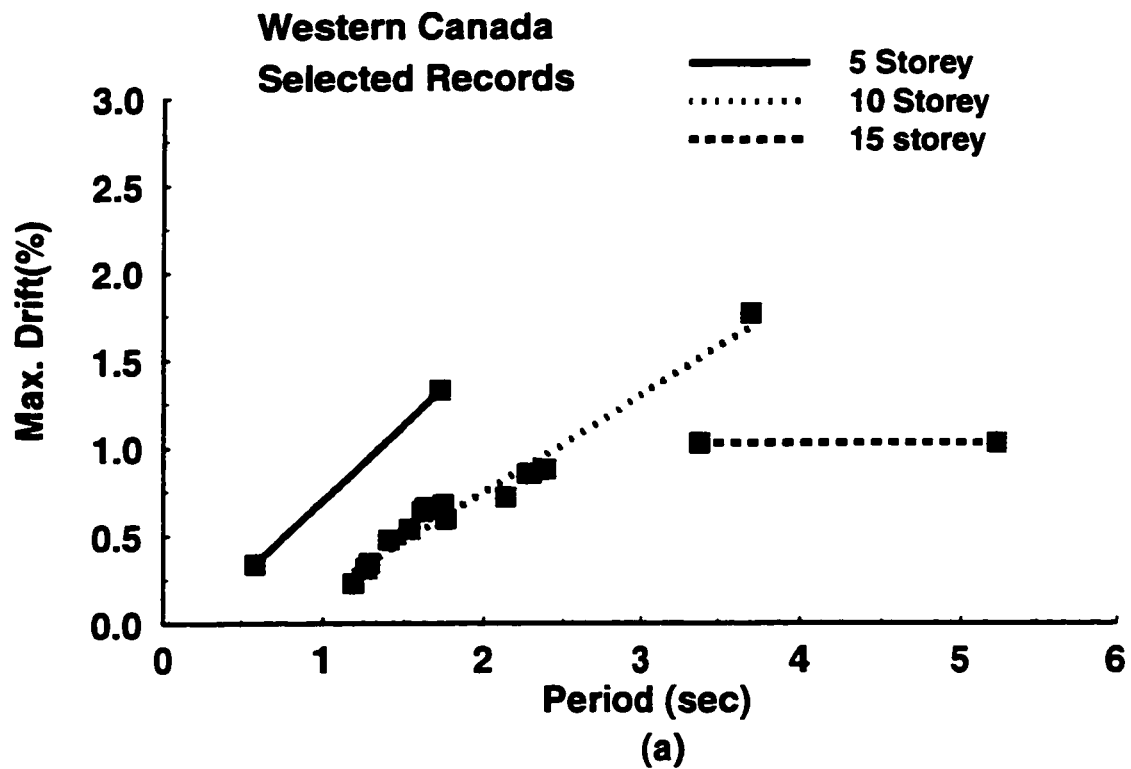


Fig. 5.80 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Selected Records for Western Canada.

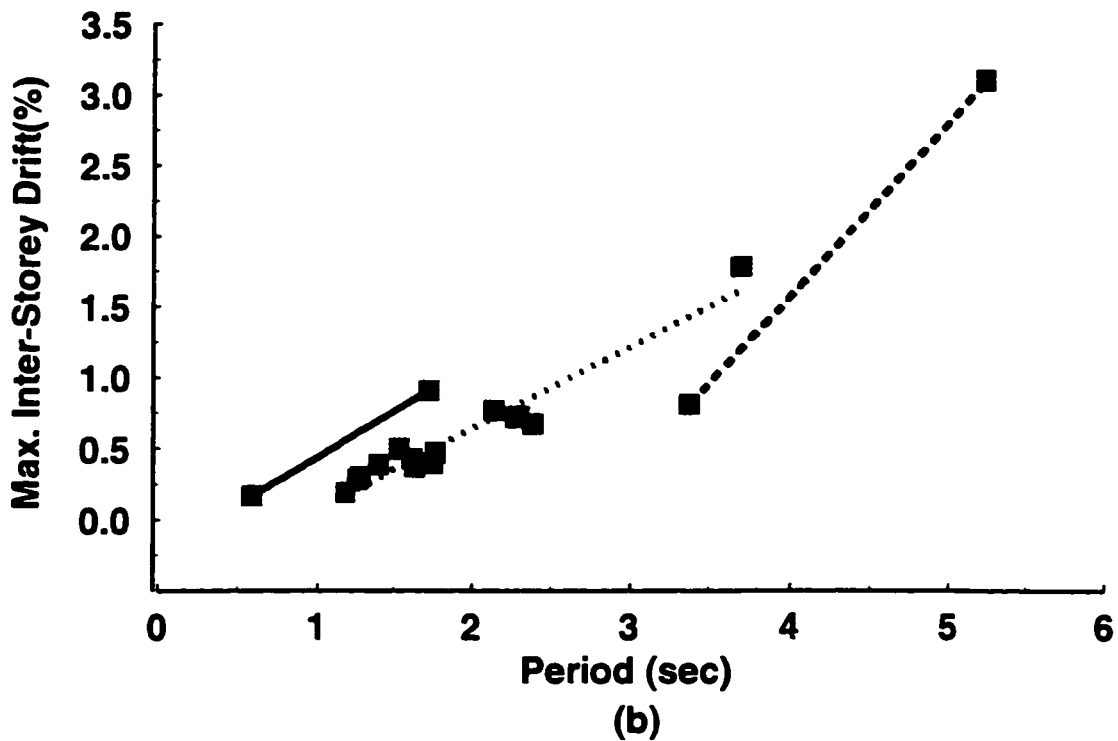
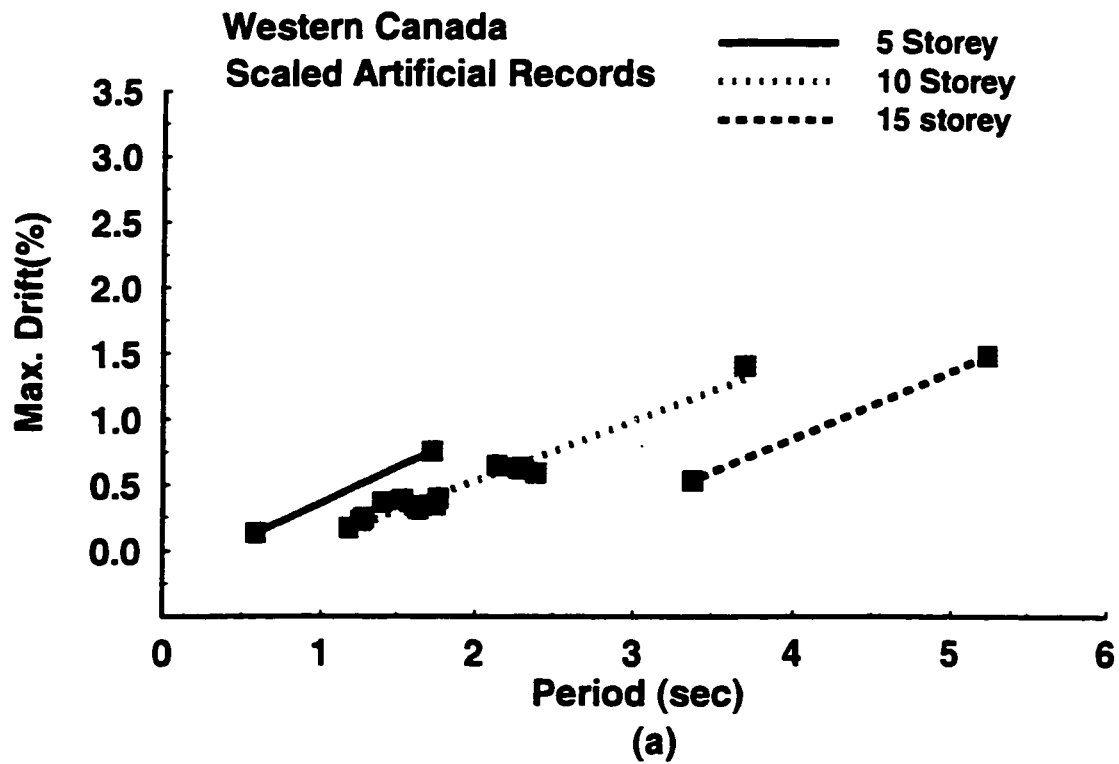


Fig. 5.81 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Scaled Artificial Records for Western Canada.

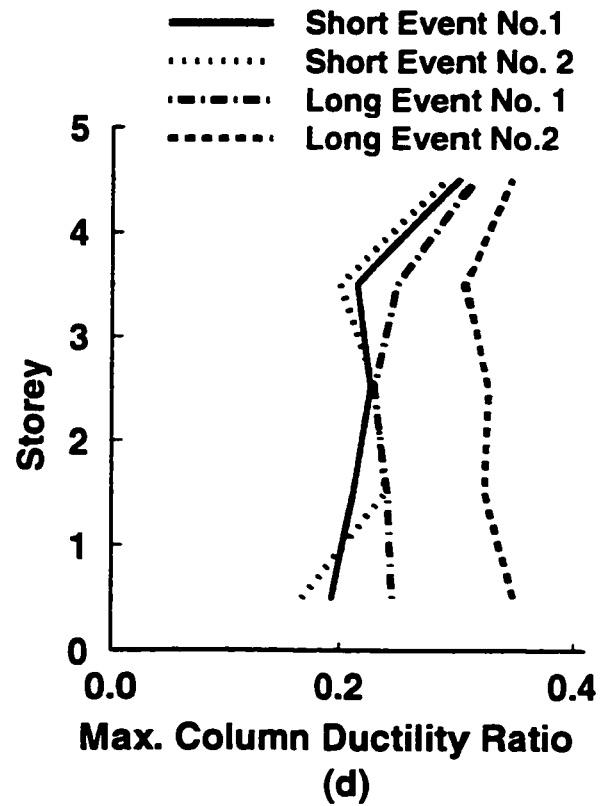
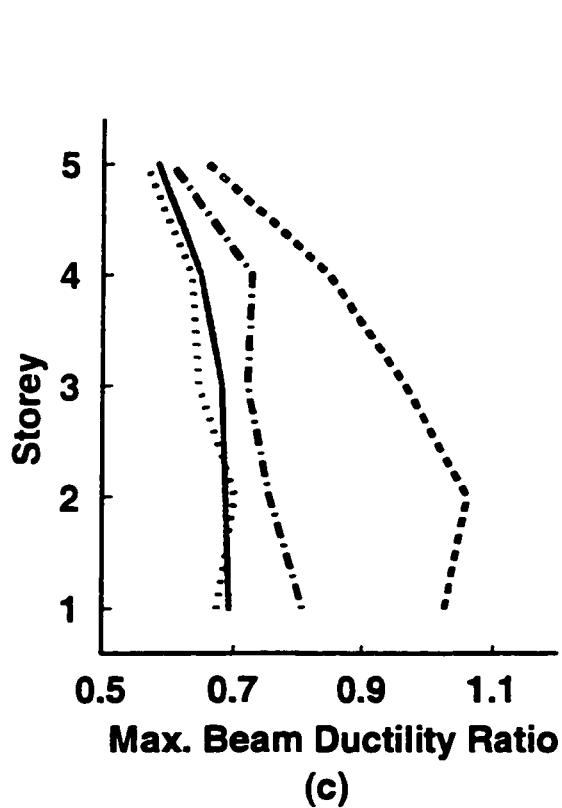
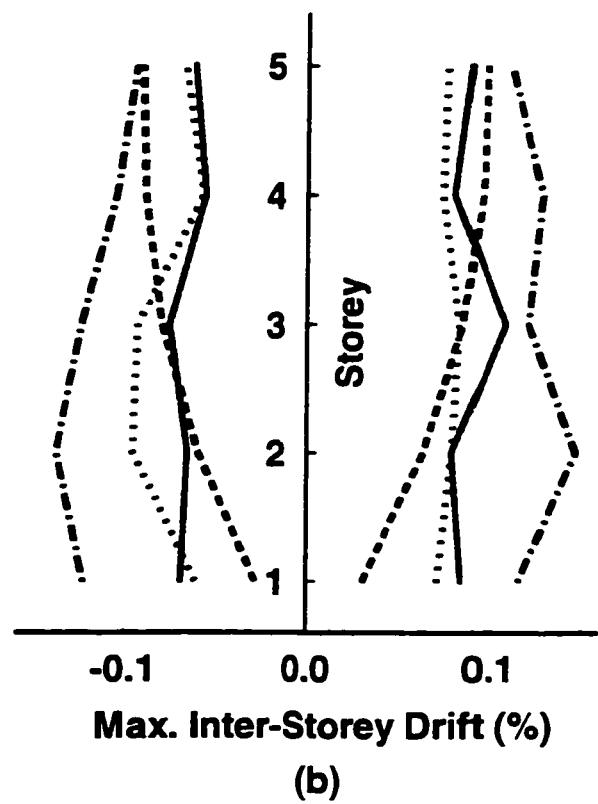
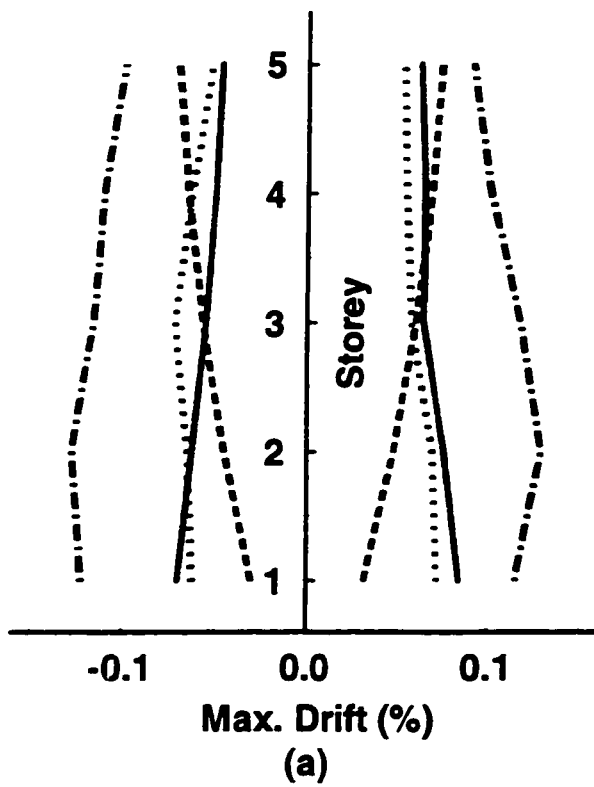


Fig. 5.82 Drift and Ductility Response for 5-Storey MRF Subjected to Artificial Records for Eastern Canada.

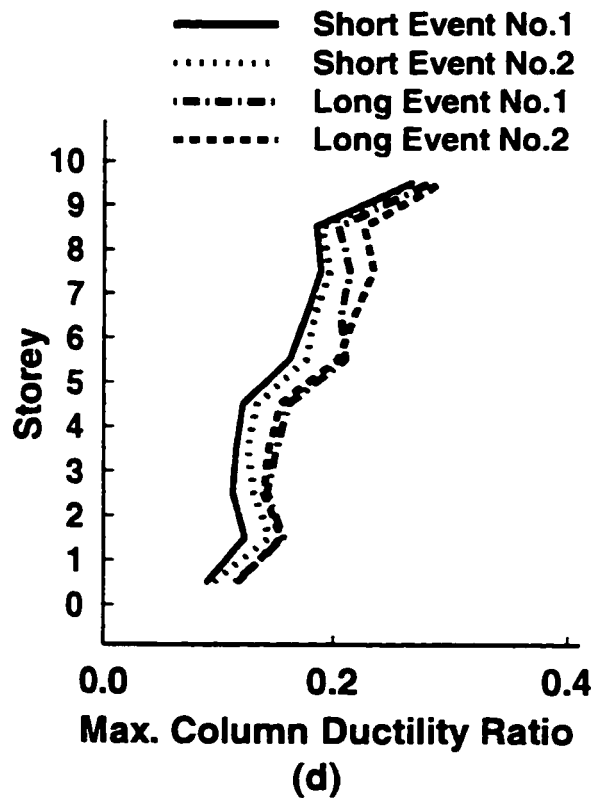
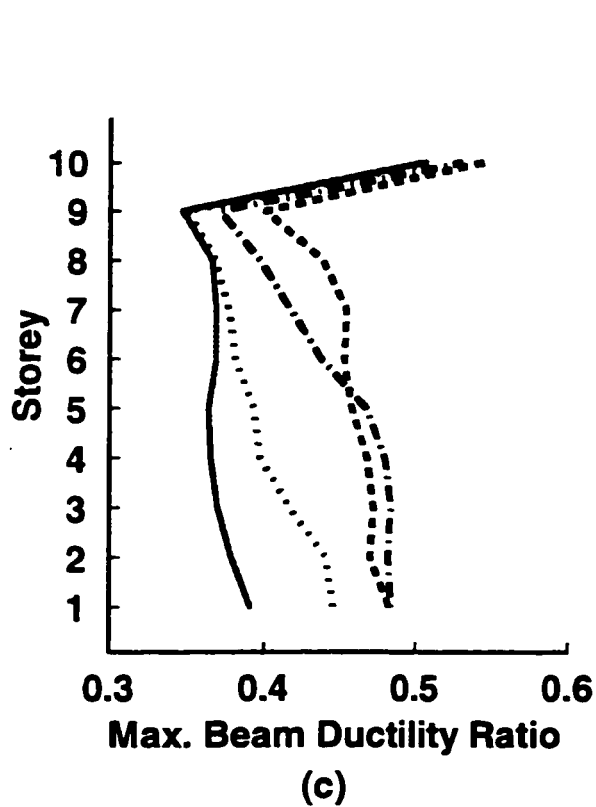
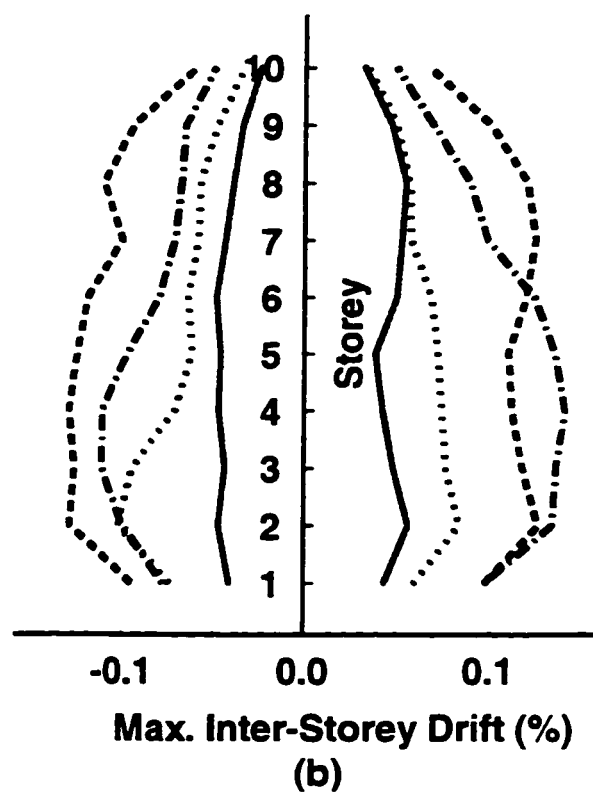
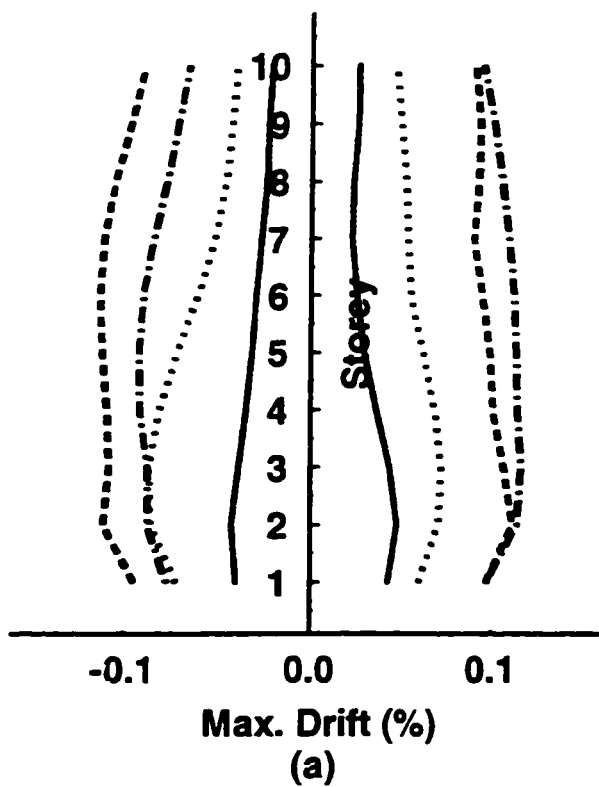


Fig. 5.83 Drift and Ductility Response for 10-Storey MRF Subjected to Artificial Records for Eastern Canada.

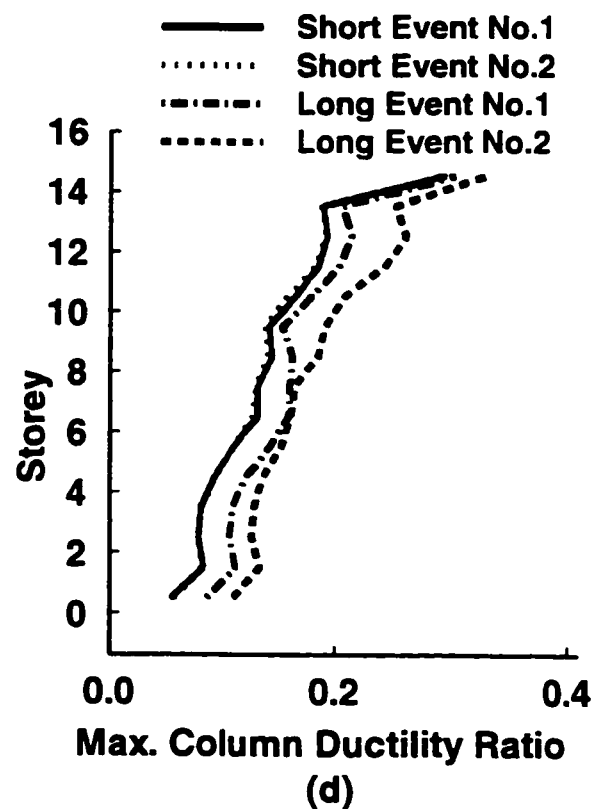
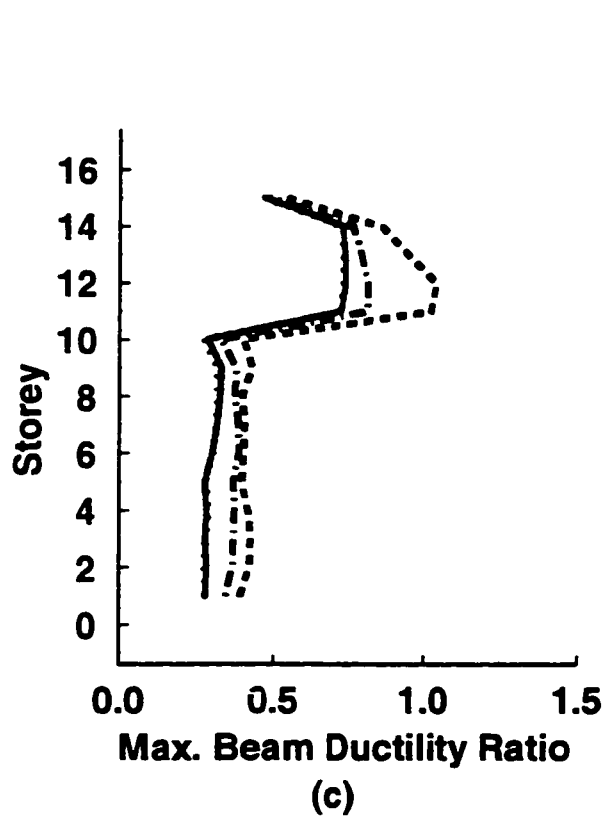
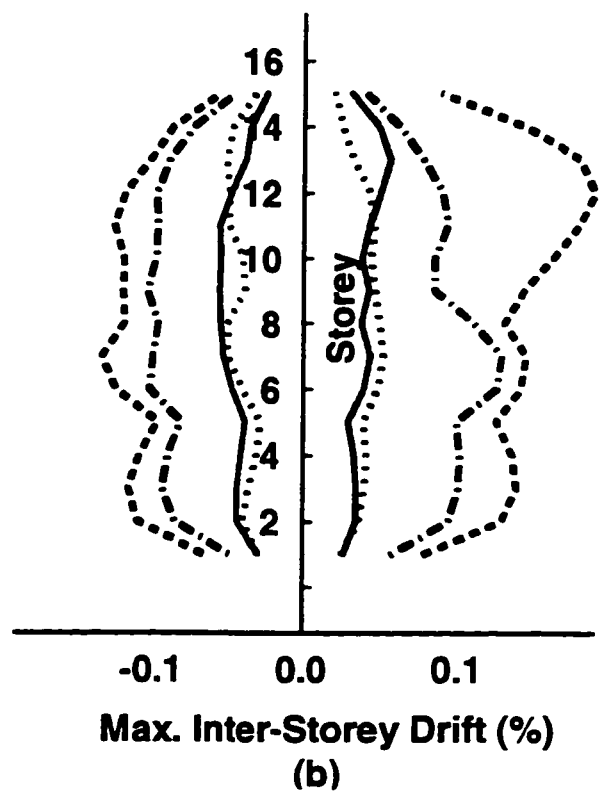
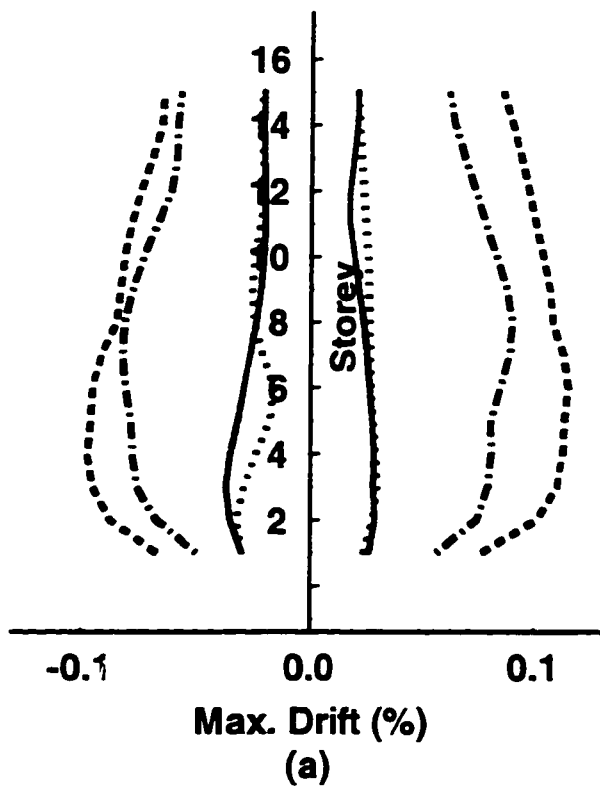
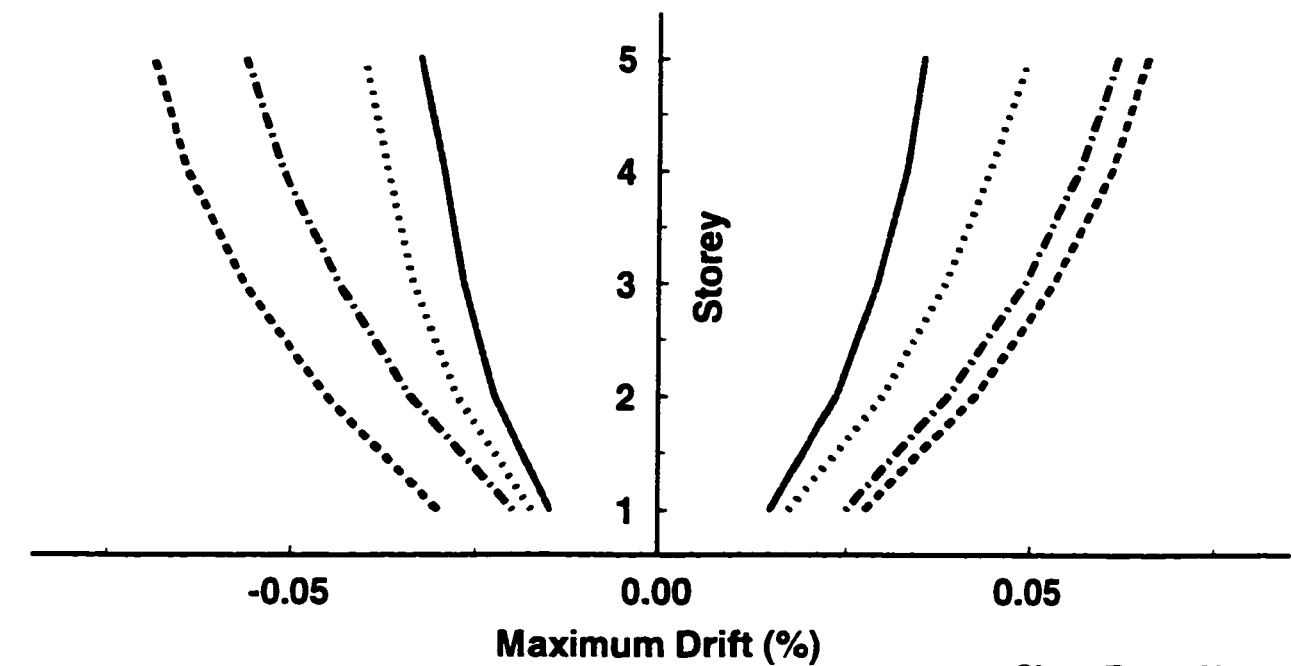
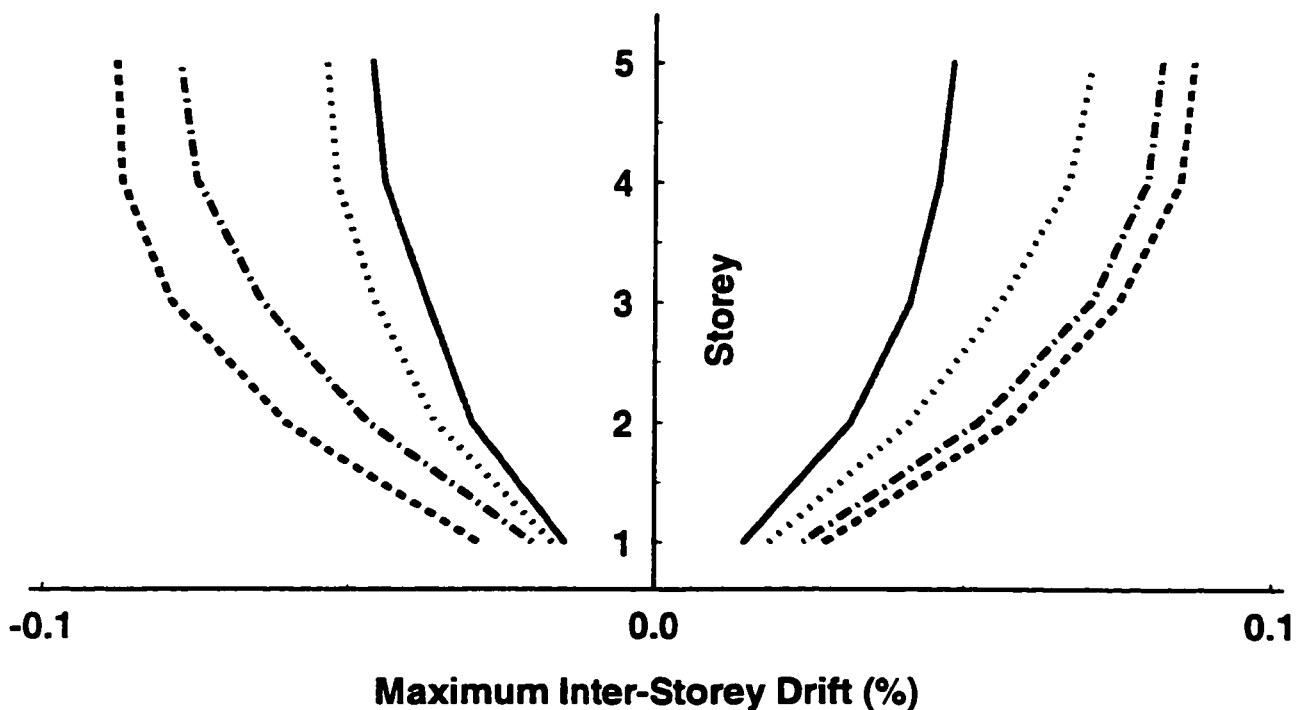


Fig. 5.84 Drift and Ductility Response for 15-Storey MRF Subjected to Artificial Records for Eastern Canada.



(a)

- Short Event No. 1
- Short Event No. 2
- . - Long Event No. 1
- - - Long Event No. 2



(b)

Fig. 5.85 Drift Response for 5-Storey SHW Subjected to Artificial Records for Eastern Canada.

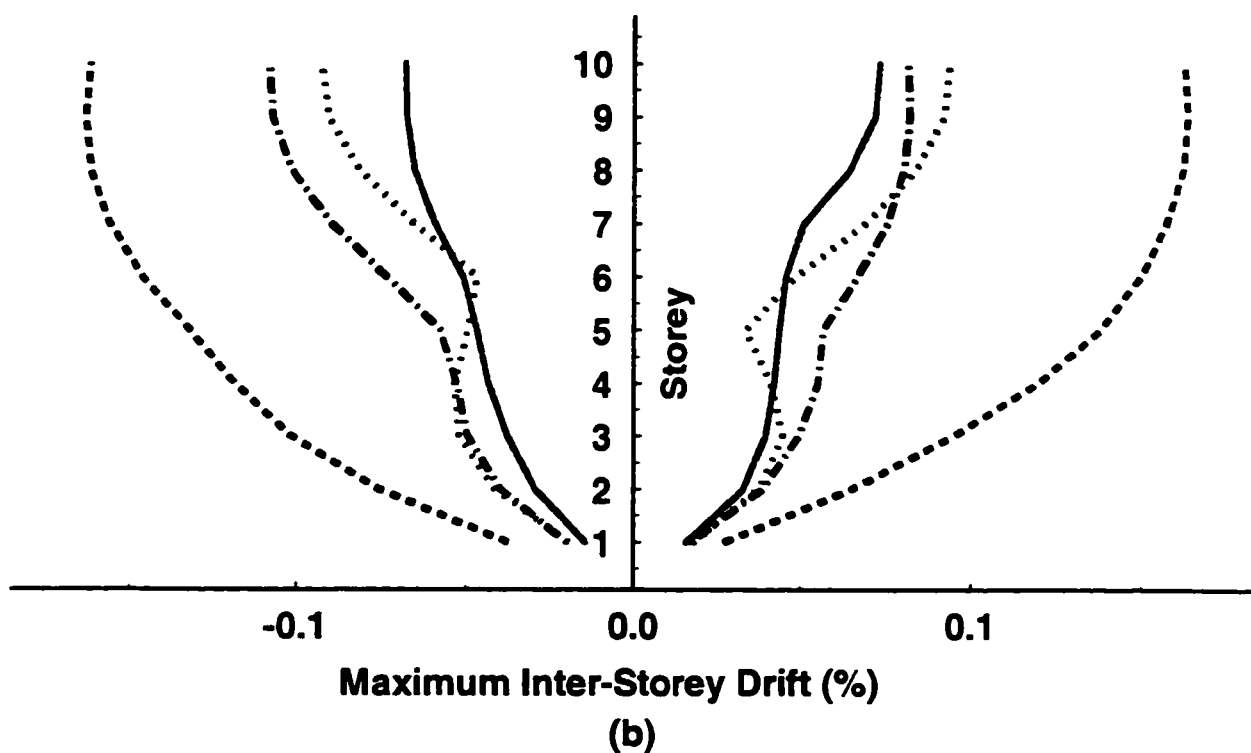
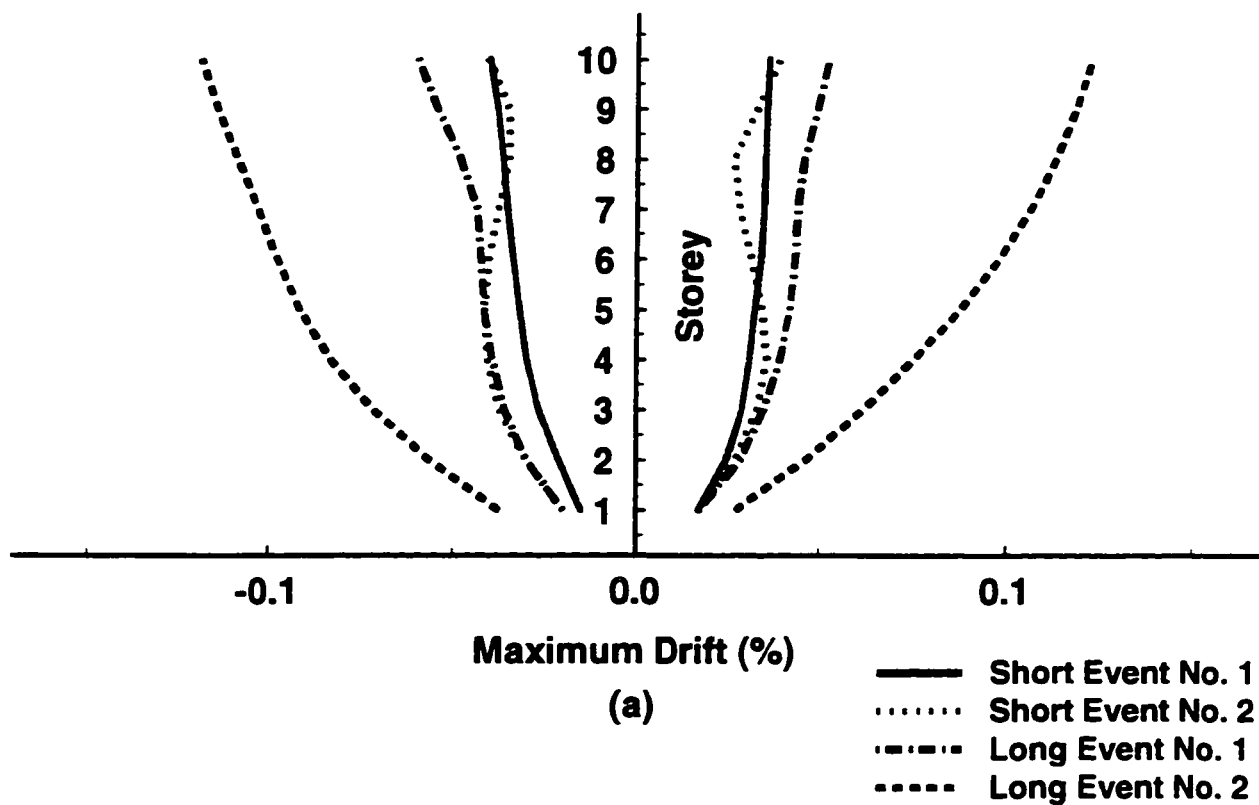


Fig. 5.86 Drift Response for 10-Storey SHW Subjected to Artificial Records for Eastern Canada.

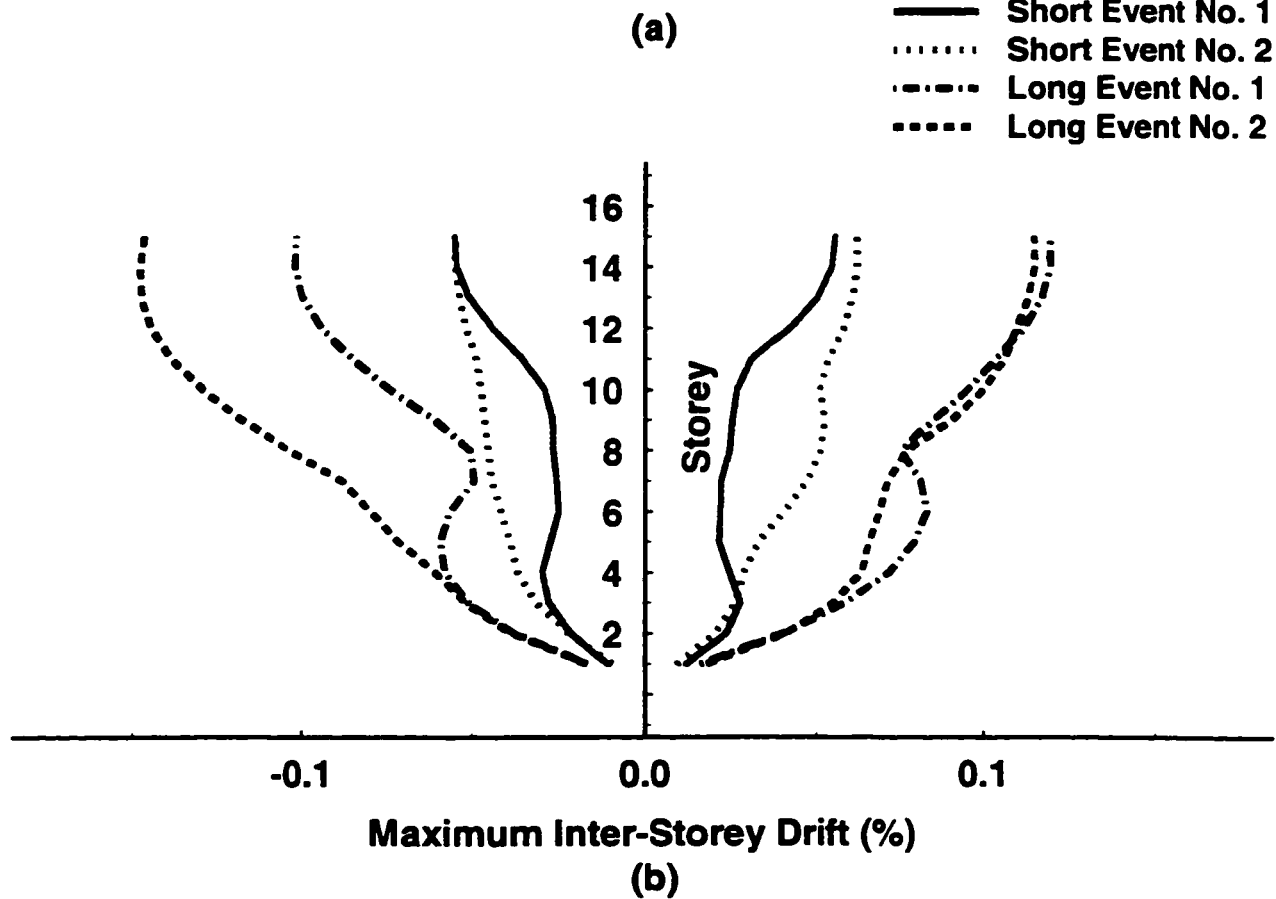
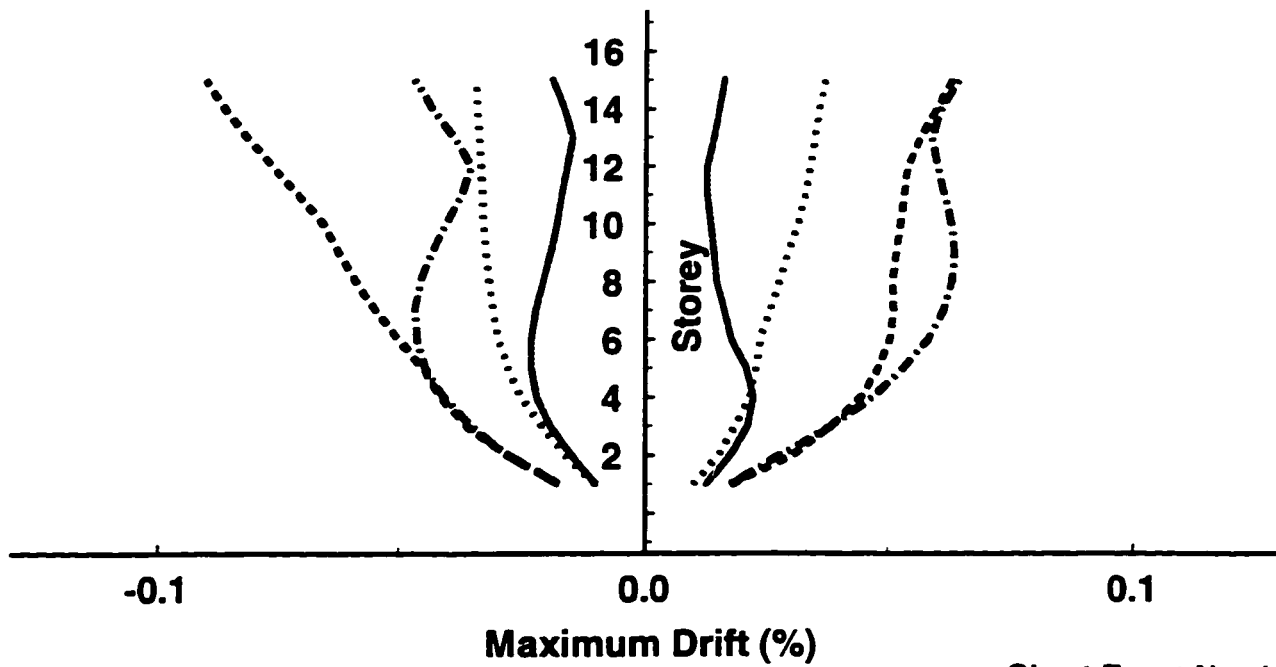
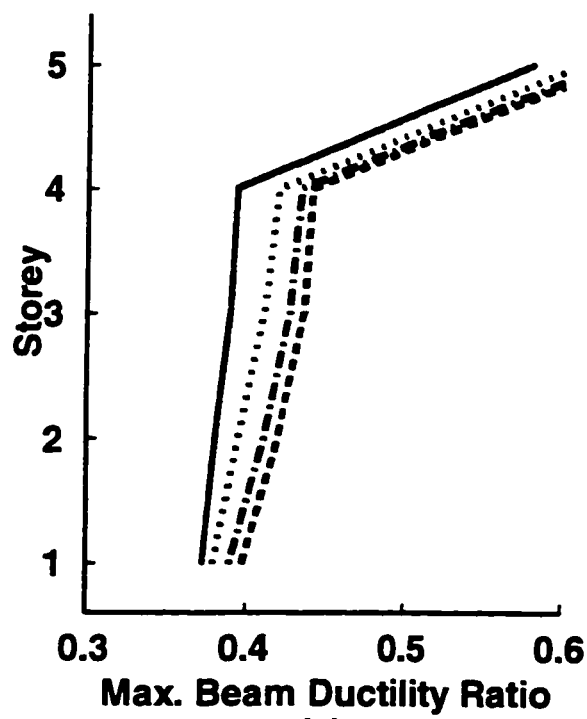
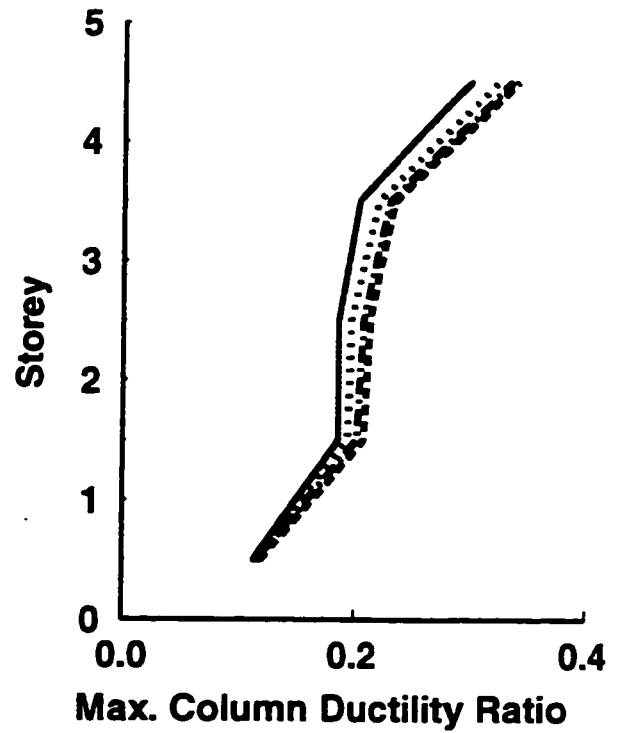


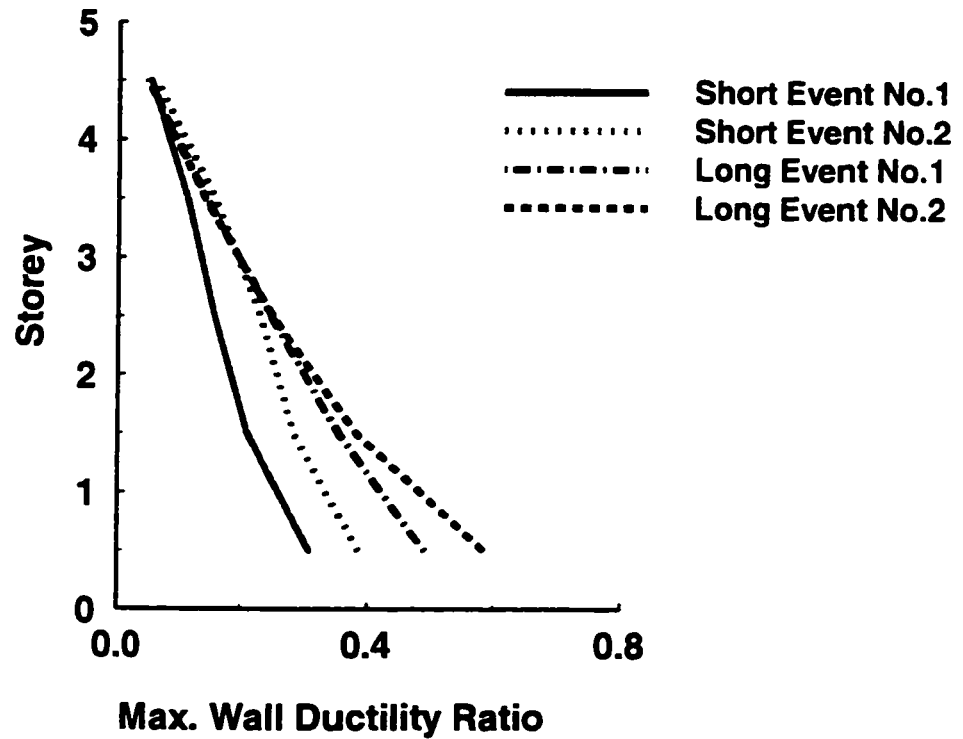
Fig. 5.87 Drift Response for 15-Storey SHW Subjected to Artificial Records for Eastern Canada.



(a)



(b)



(c)

Fig. 5.88 Ductility Response for 5-Storey SHW Subjected to Artificial Records for Eastern Canada.

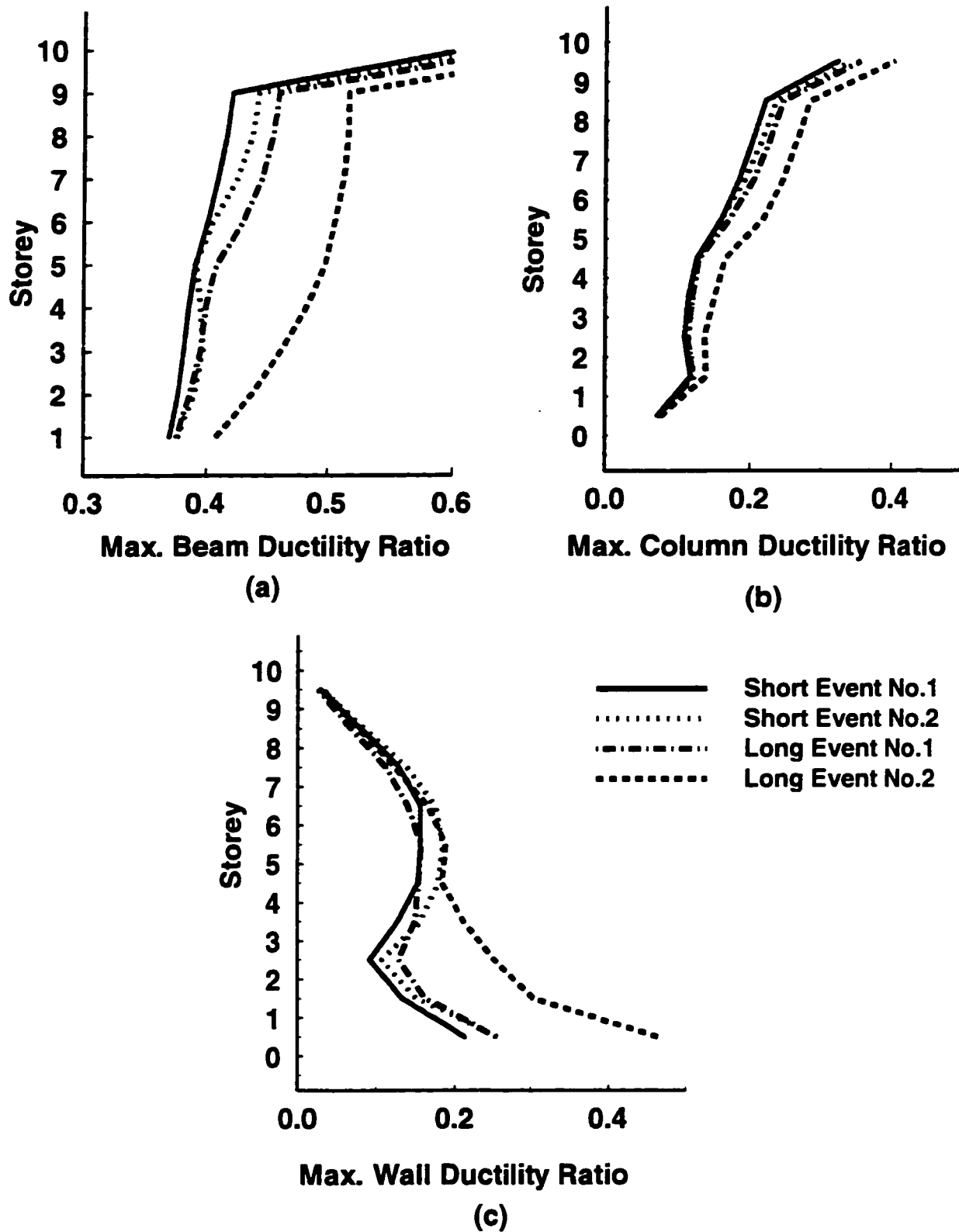


Fig. 5.89 Ductility Response for 10-Storey SHW Subjected to Artificial Records for Eastern Canada.

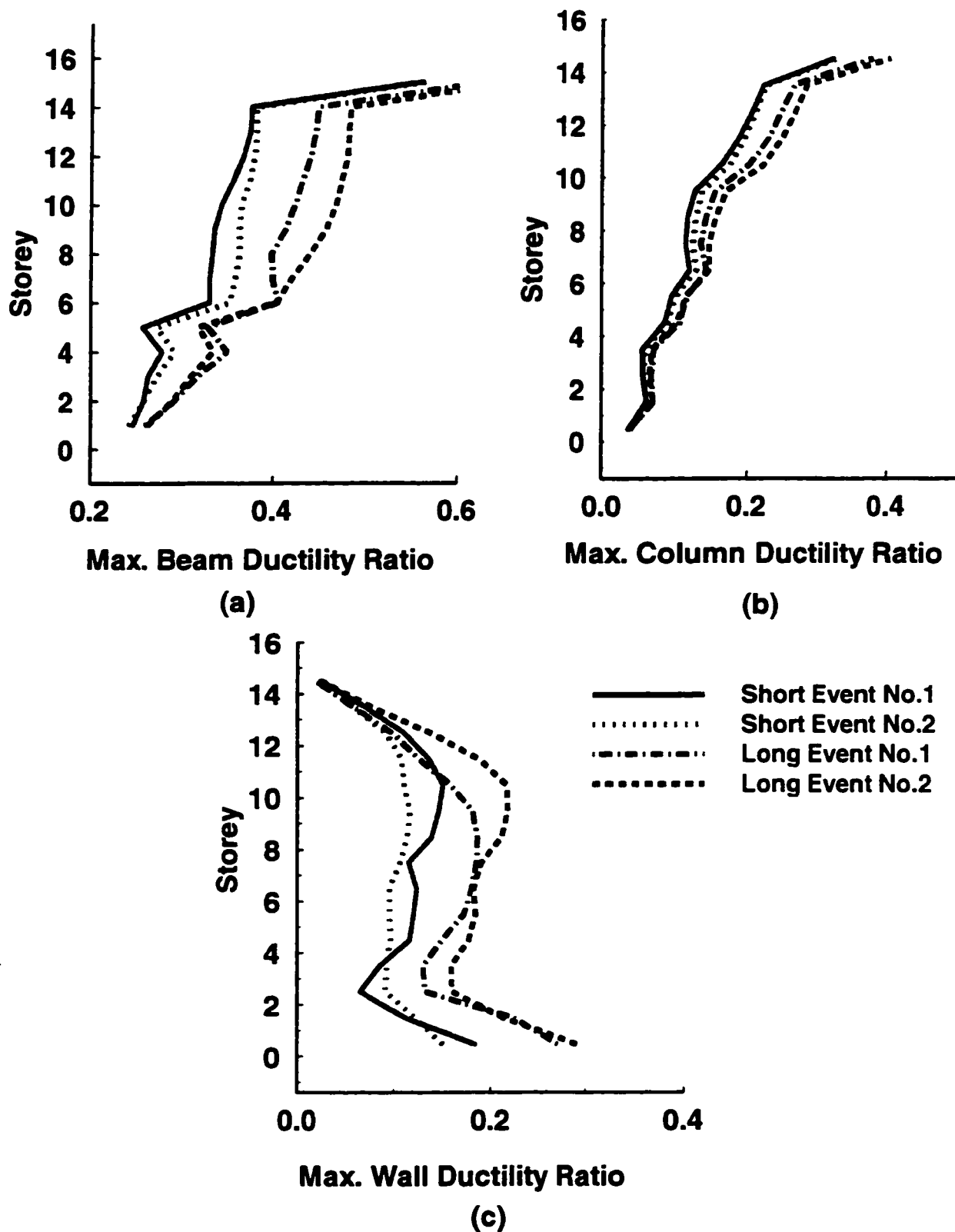


Fig. 5.90 Ductility Response for 15-Storey SHW Subjected to Artificial Records for Eastern Canada.

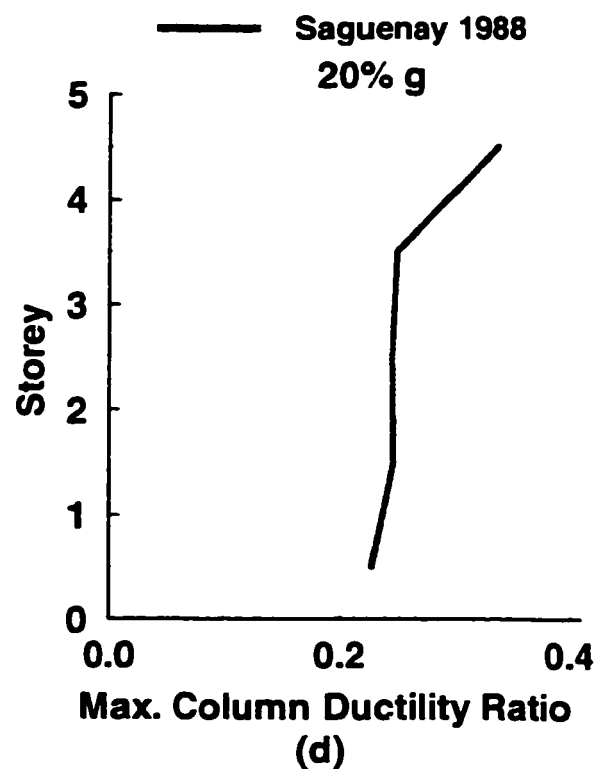
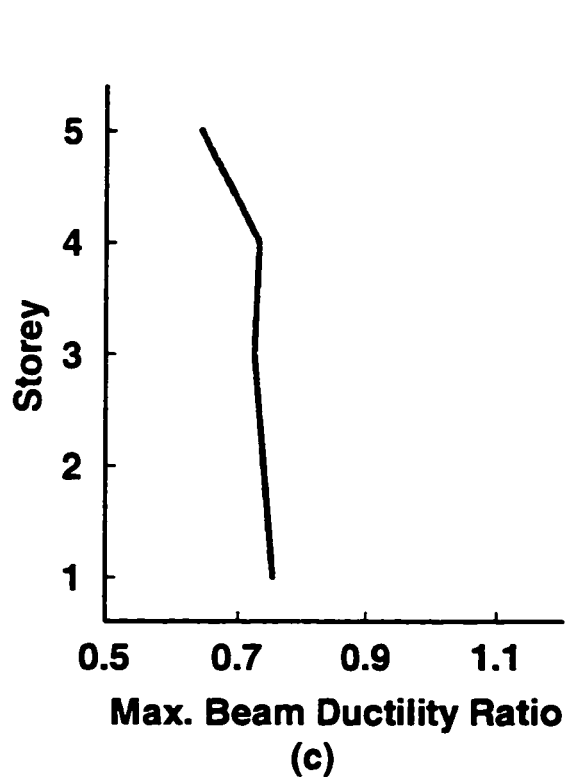
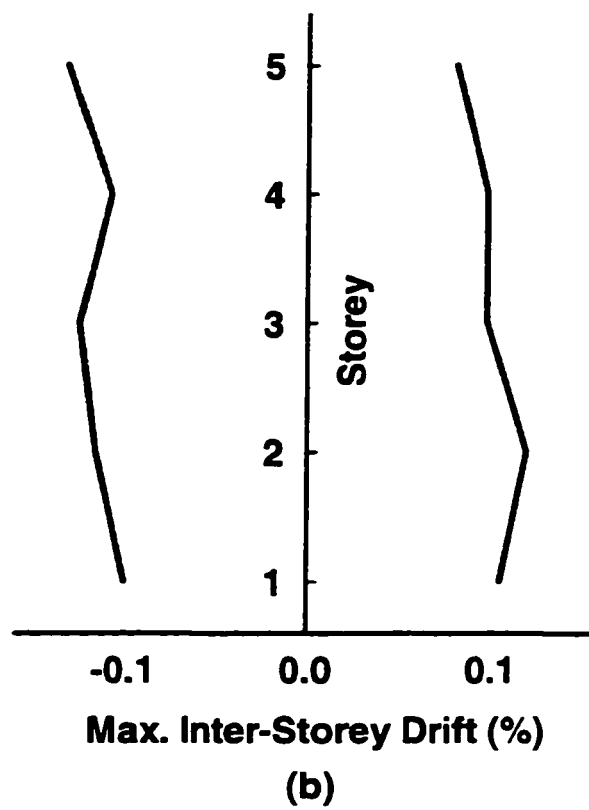
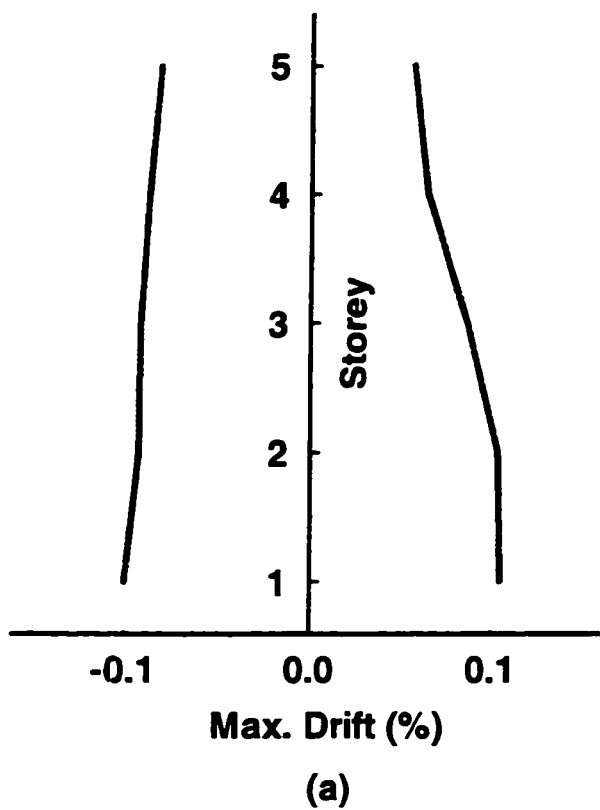


Fig. 5.91 Drift and Ductility Response for 5-Storey MRF Subjected to 1988 Saguenay Record for Eastern Canada.

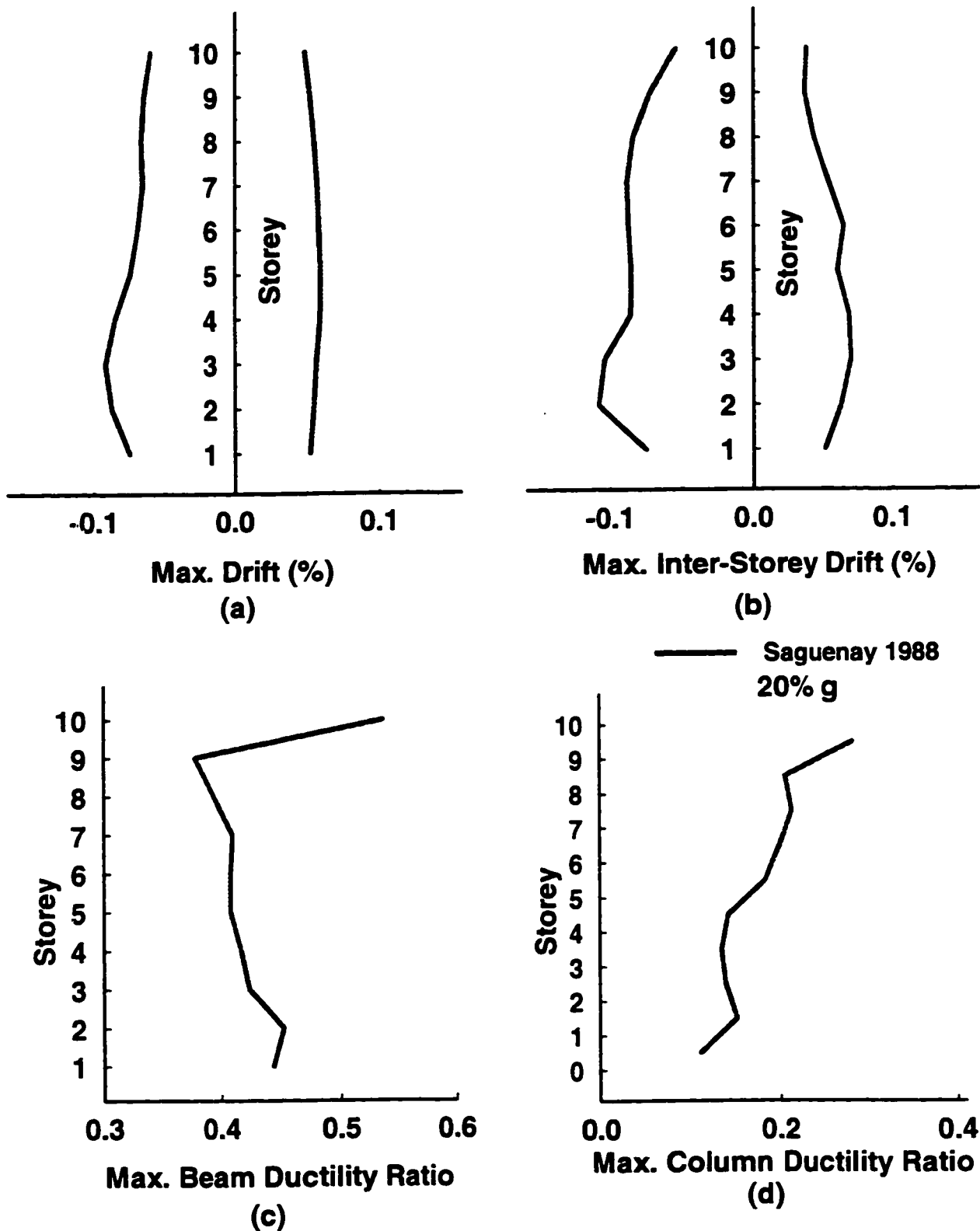


Fig. 5.92 Drift and Ductility Response for 10-Storey MRF Subjected to 1988 Saguenay Record for Eastern Canada.

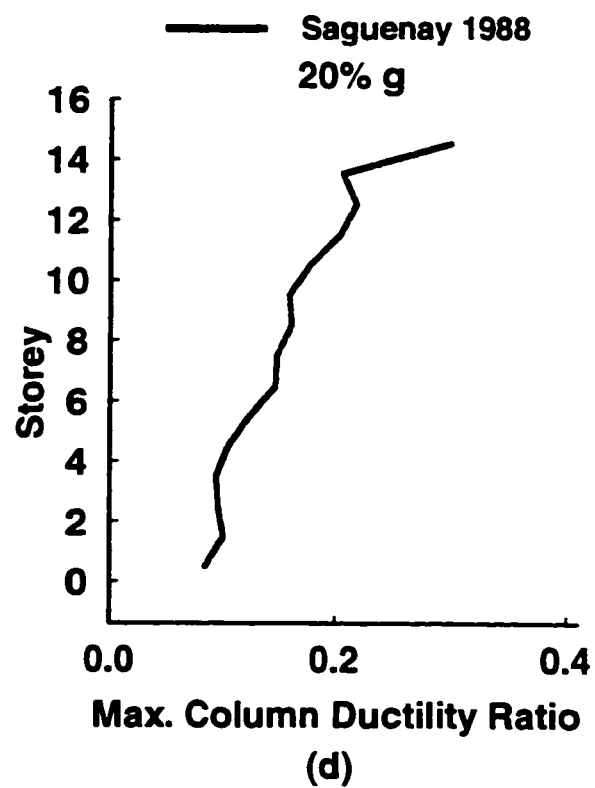
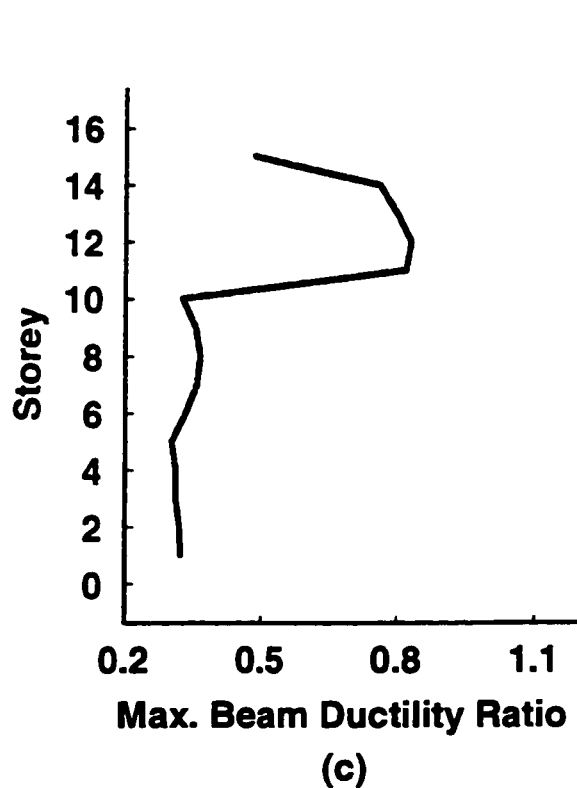
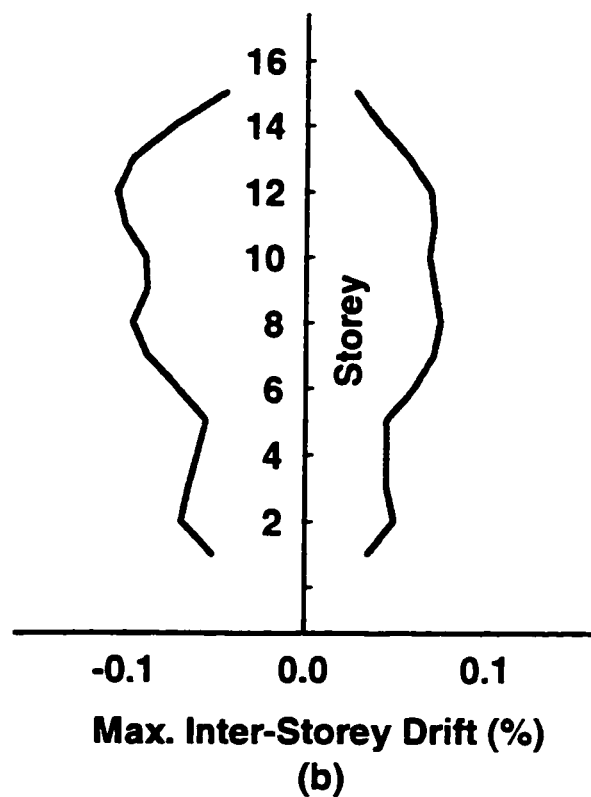
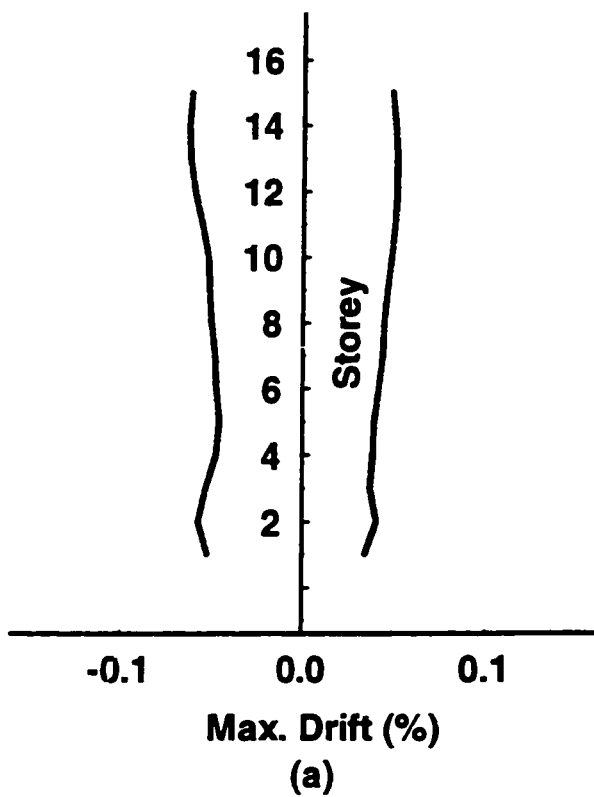
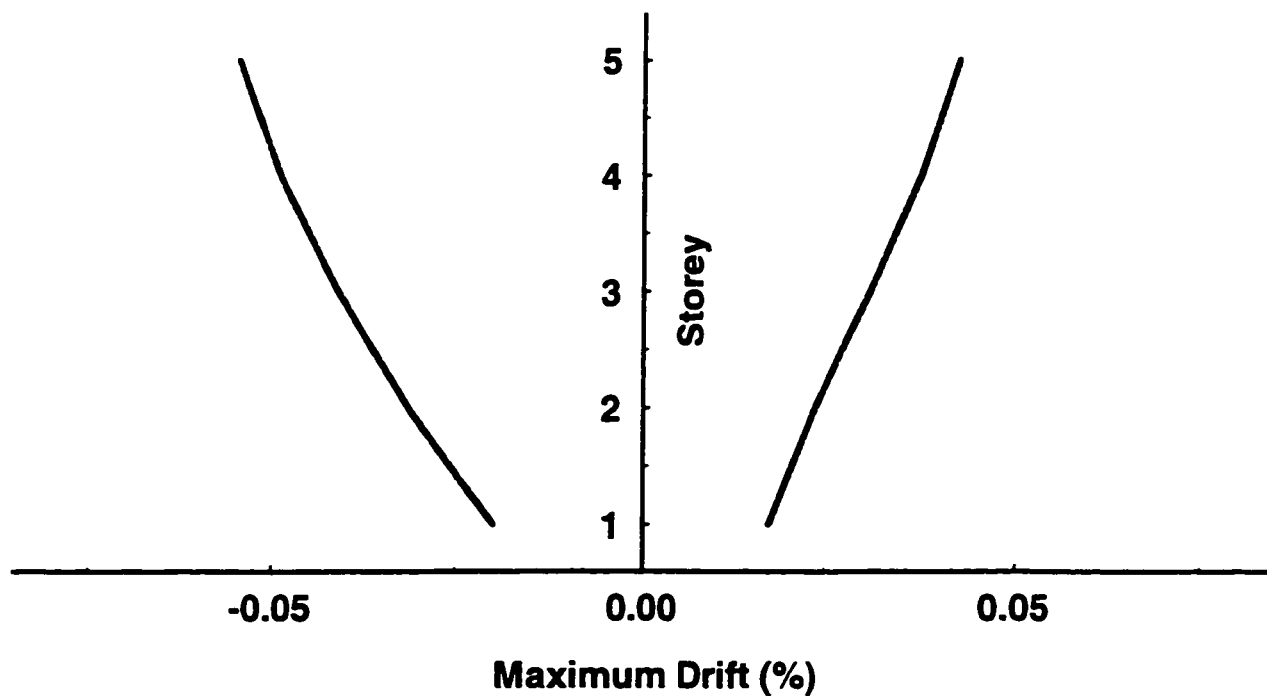
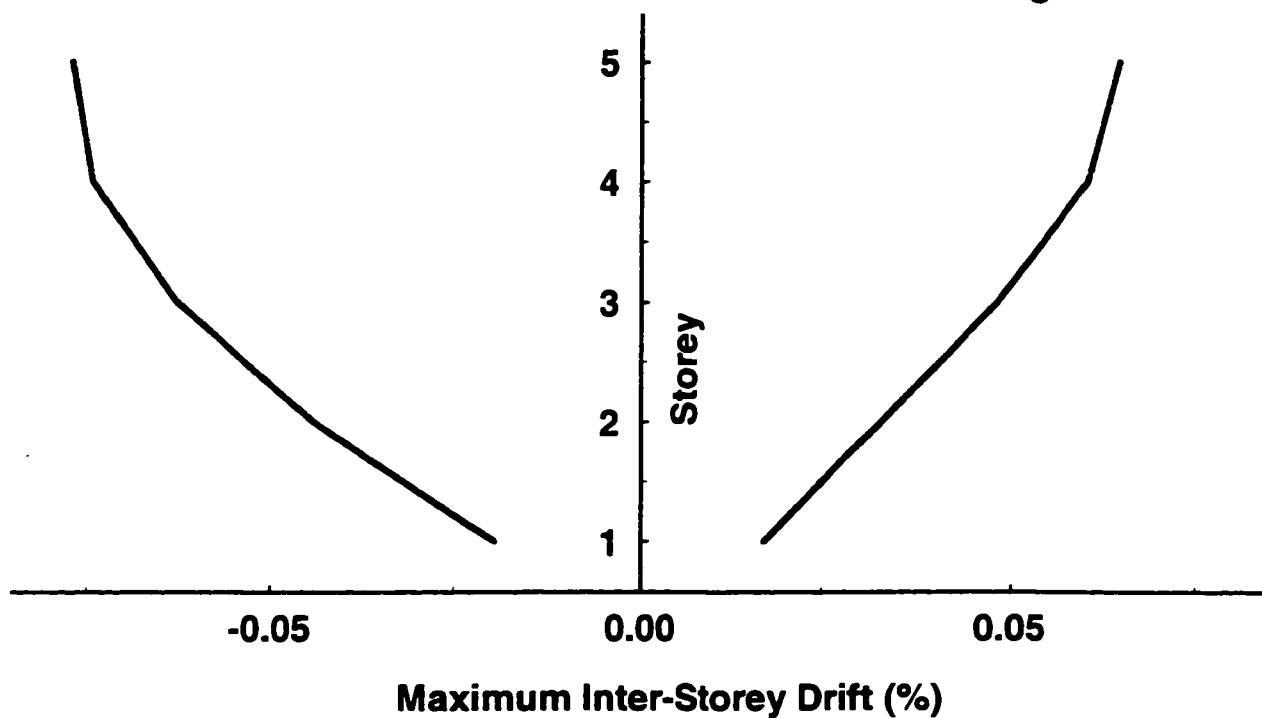


Fig. 5.93 Drift and Ductility Response for 15-Storey MRF Subjected to 1988 Saguenay Record for Eastern Canada.



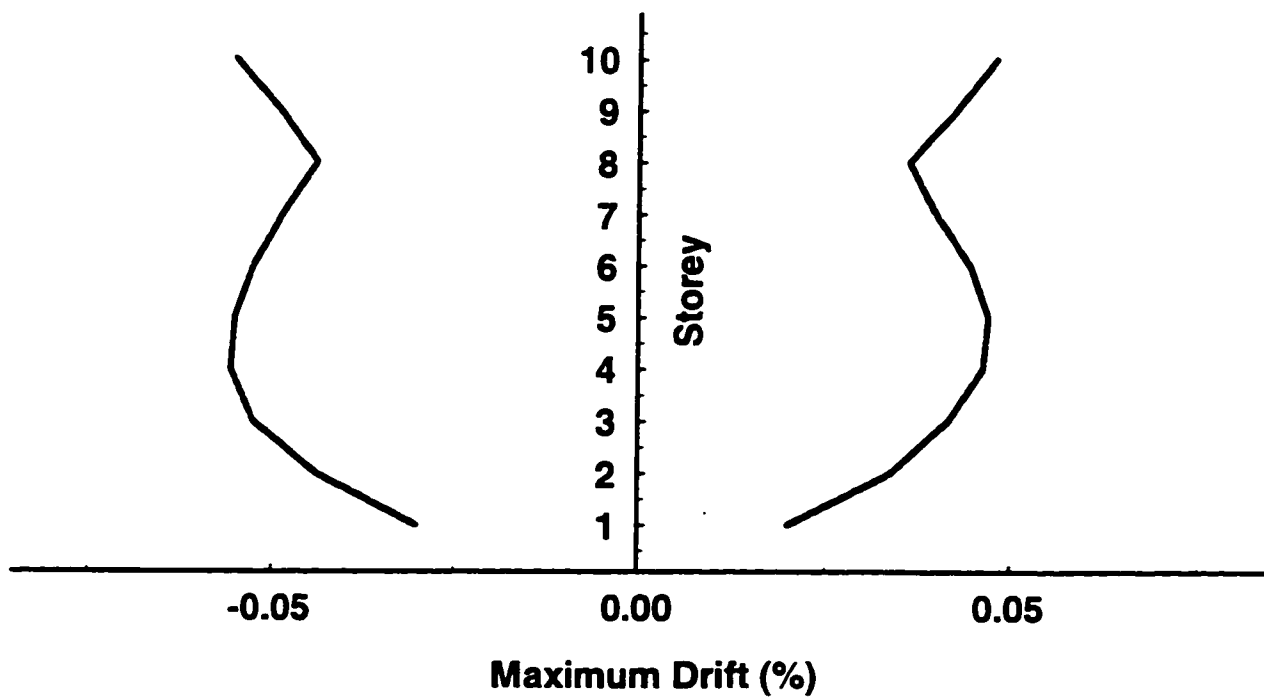
(a)

— Saguenay 1988
20% g



(b)

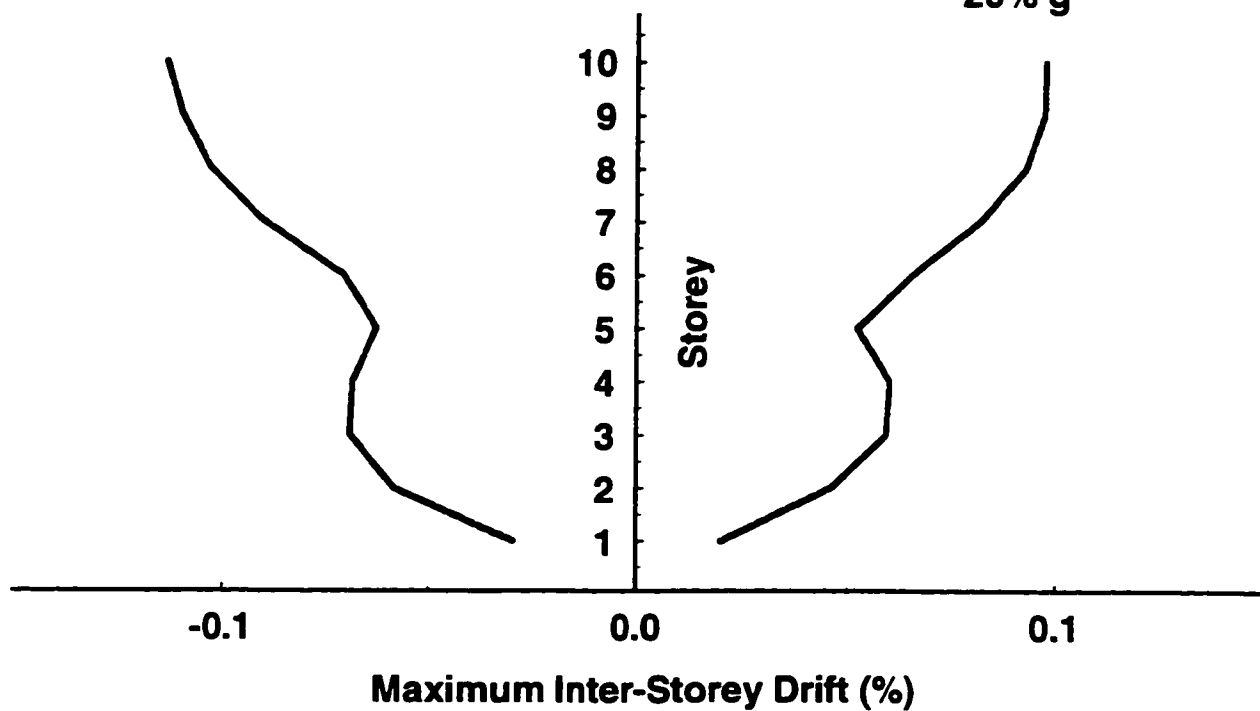
Fig. 5.94 Drift Response for 5-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.



(a)

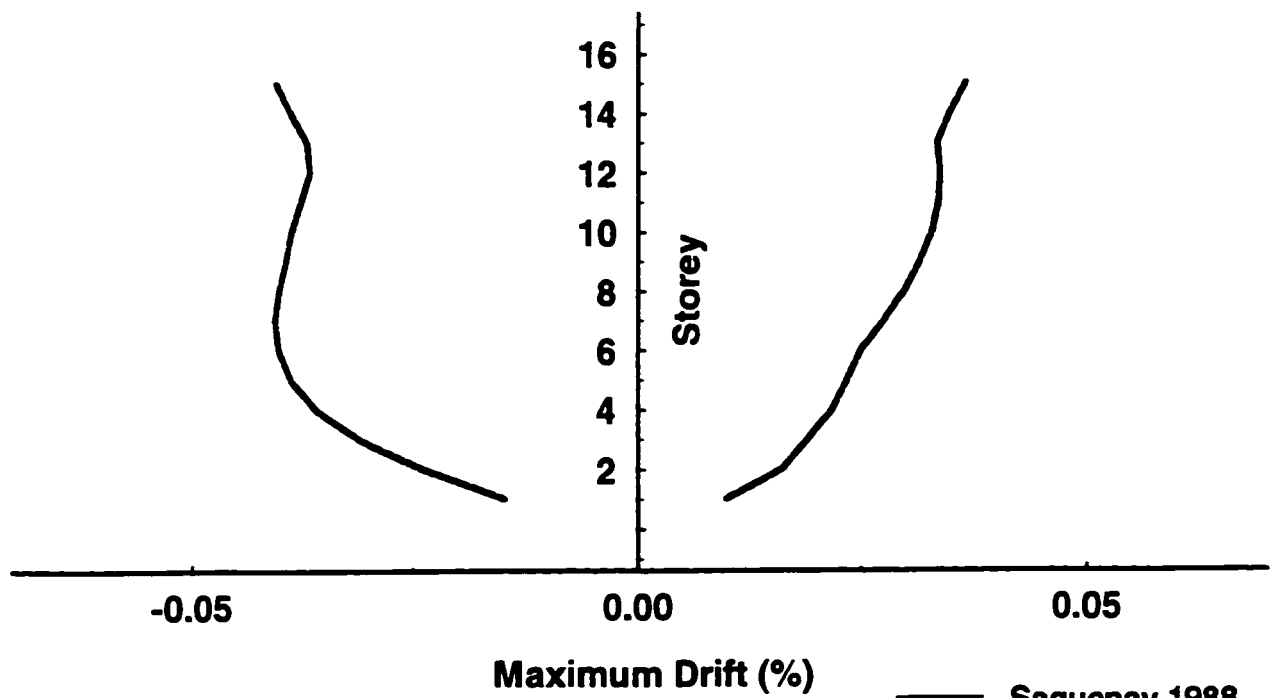
— Saguenay 1988

20% g

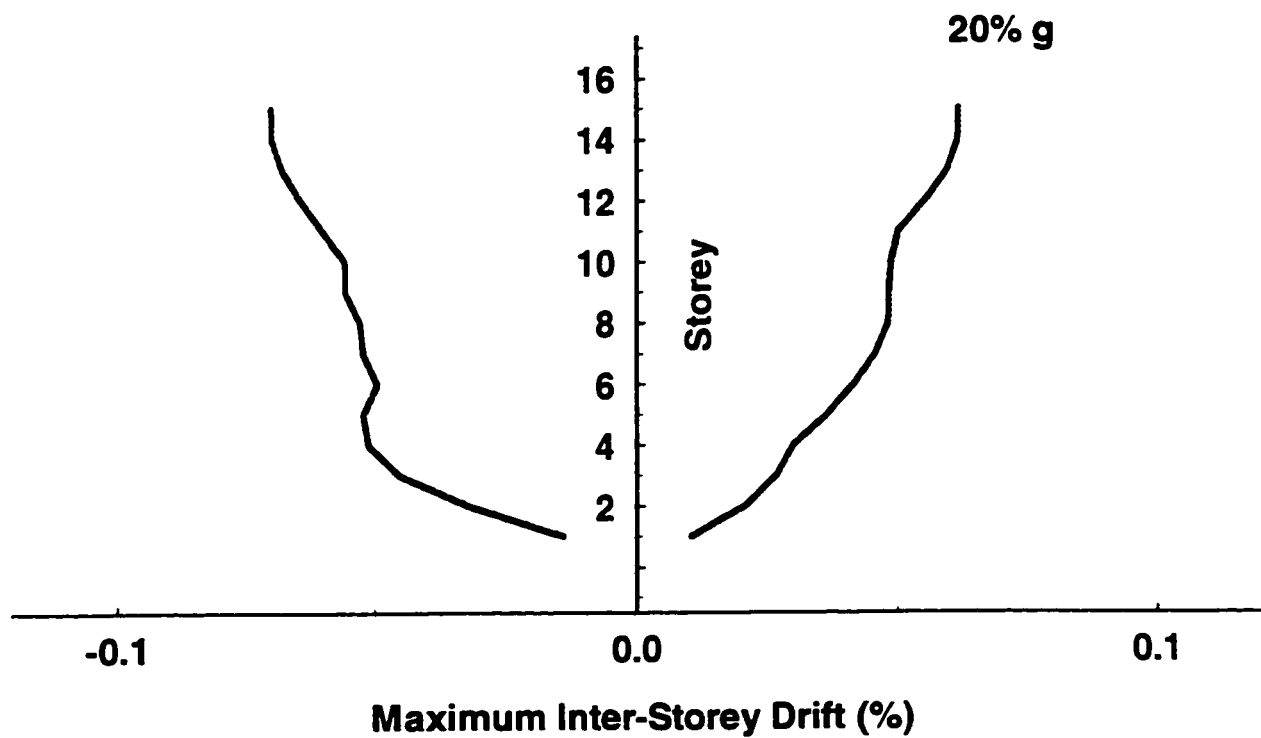


(b)

Fig. 5.95 Drift Response for 10-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.



(a)



(b)

Fig. 5.96 Drift Response for 15-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.

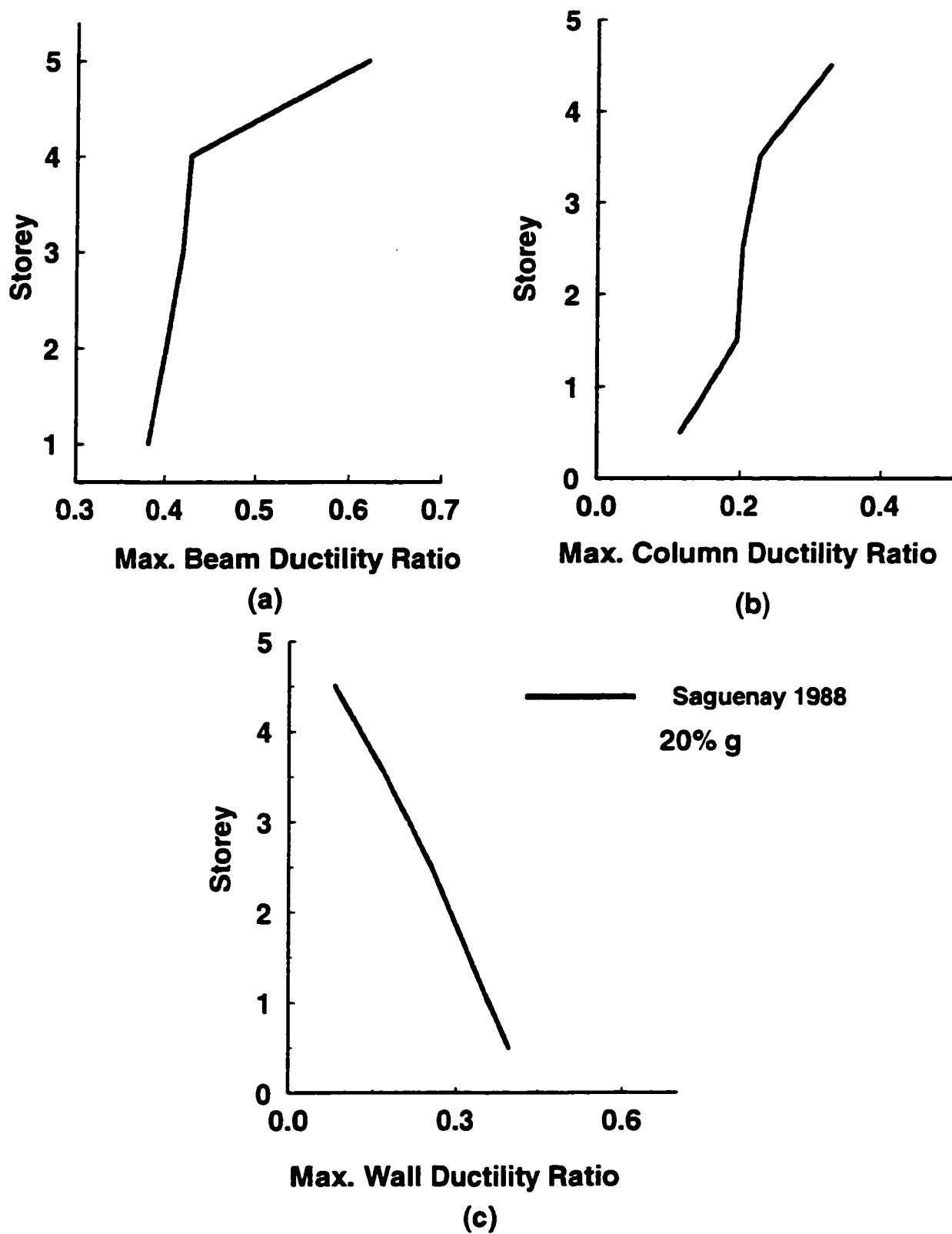
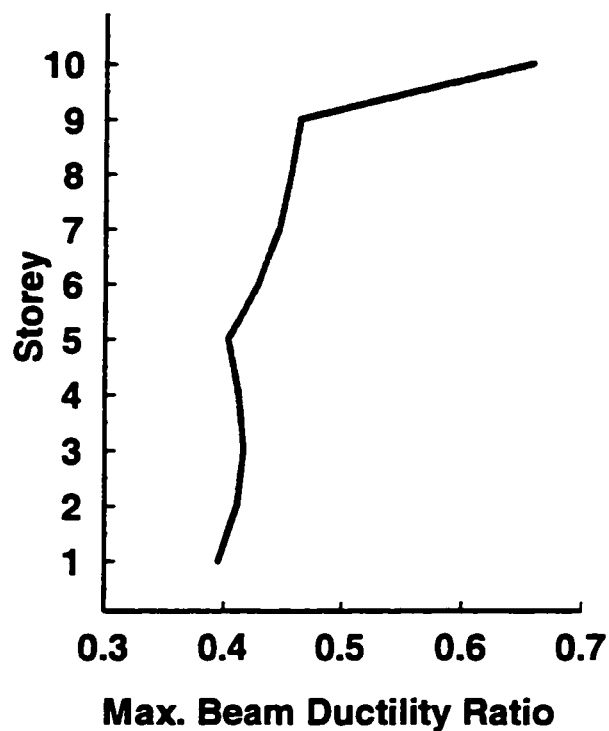
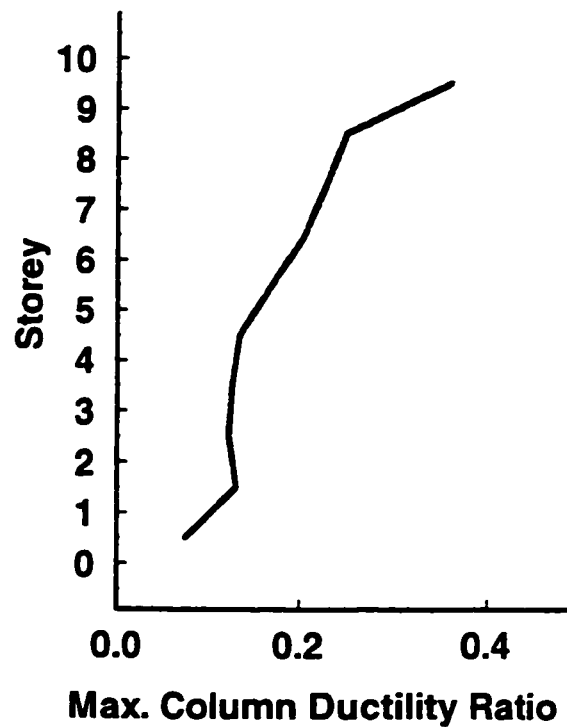


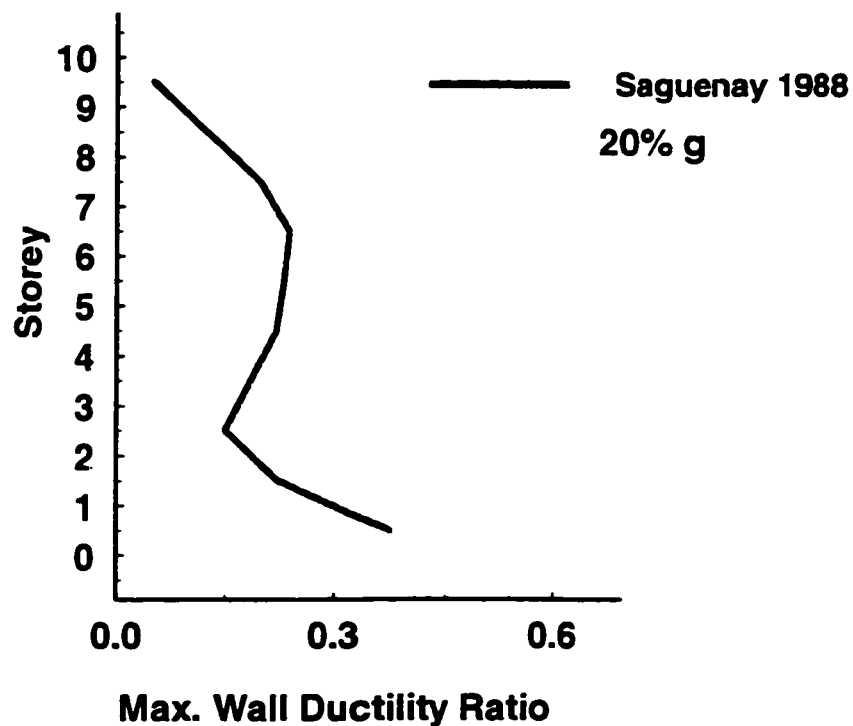
Fig. 5.97 Ductility Response for 5-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.



(a)



(b)



(c)

Fig. 5.98 Ductility Response for 10-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.

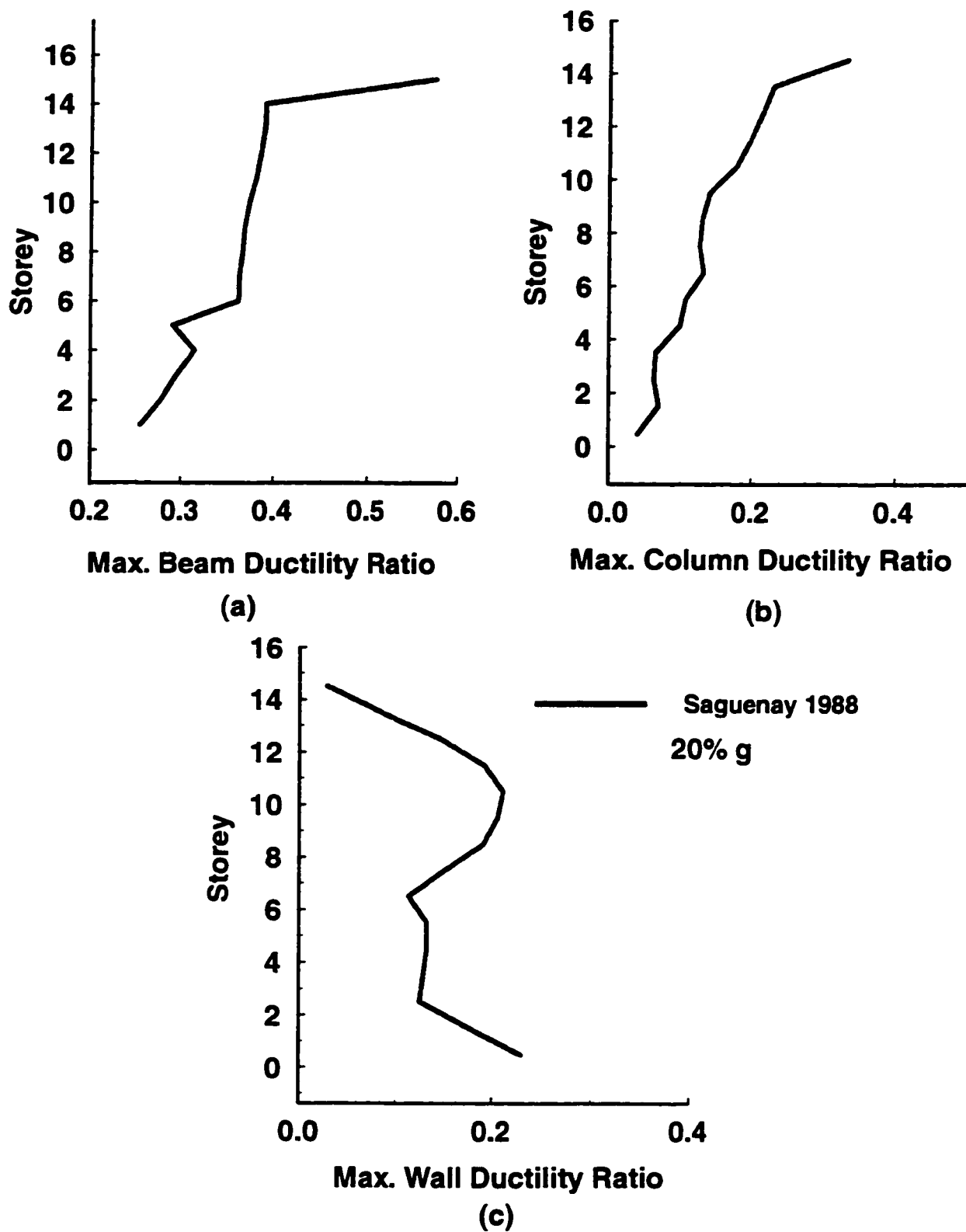


Fig. 5.99 Ductility Response for 15-Storey SHW Subjected to 1988 Saguenay Record for Eastern Canada.

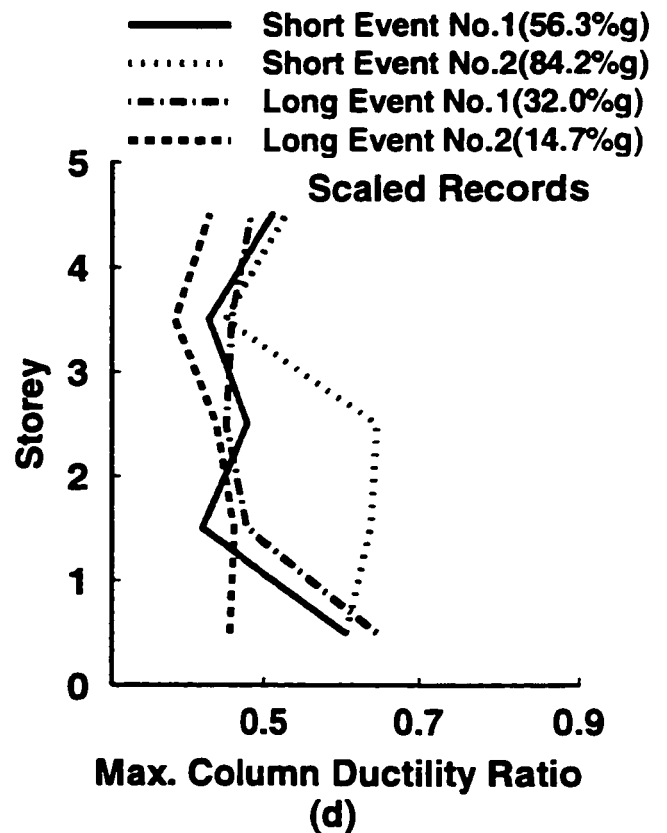
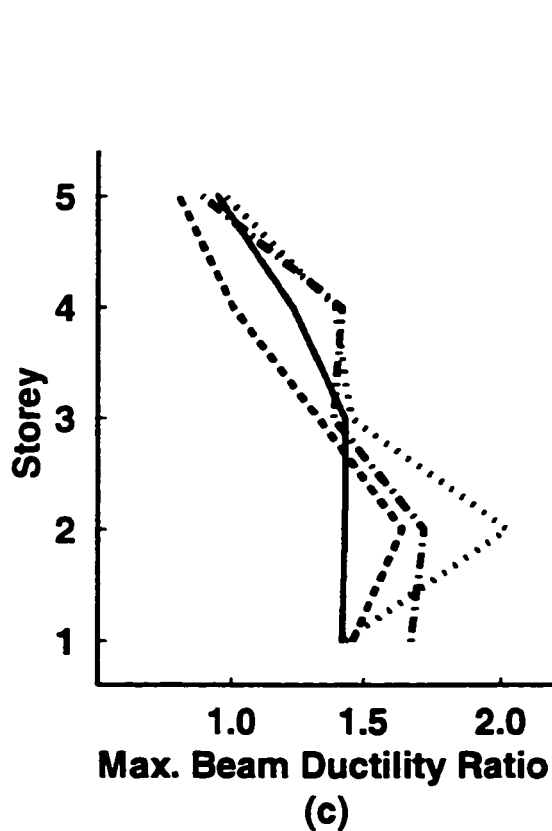
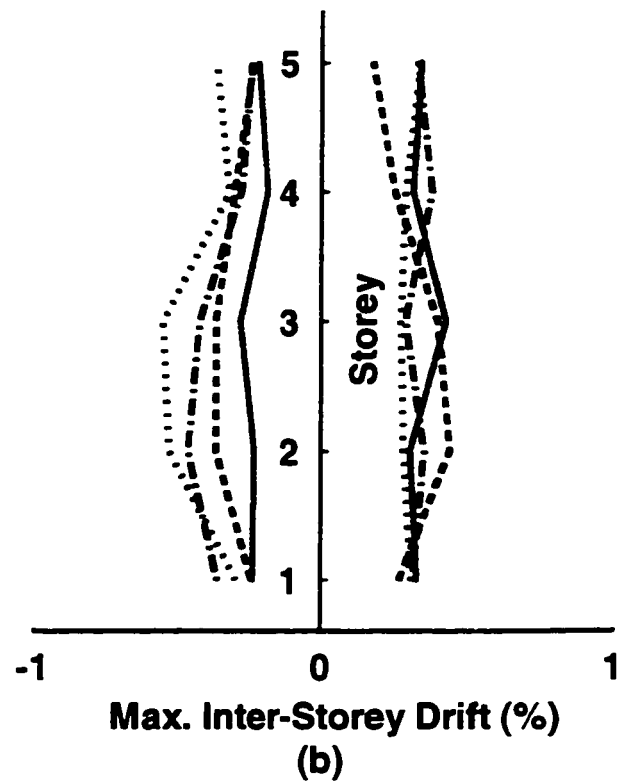
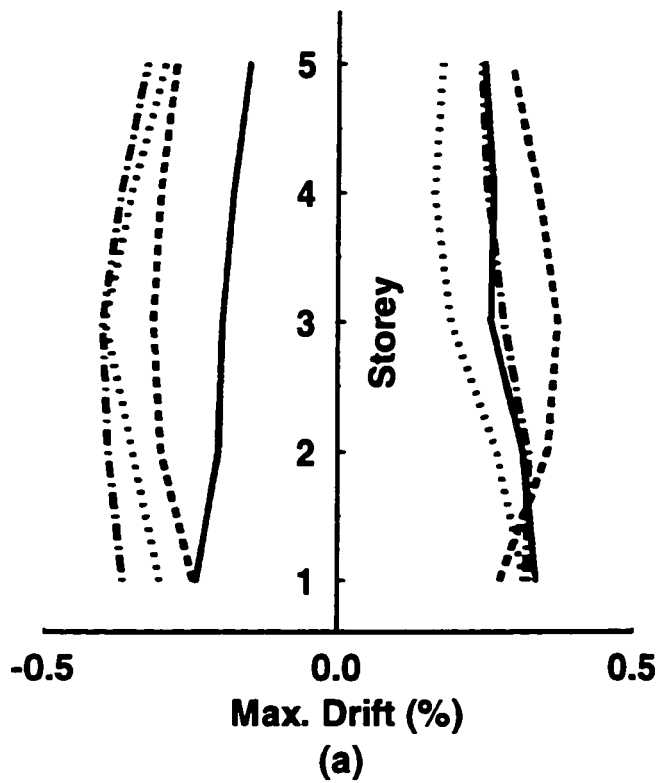


Fig. 5.100 Drift and Ductility Response for 5-Storey MRF Subjected to Scaled Artificial Records for Eastern Canada.

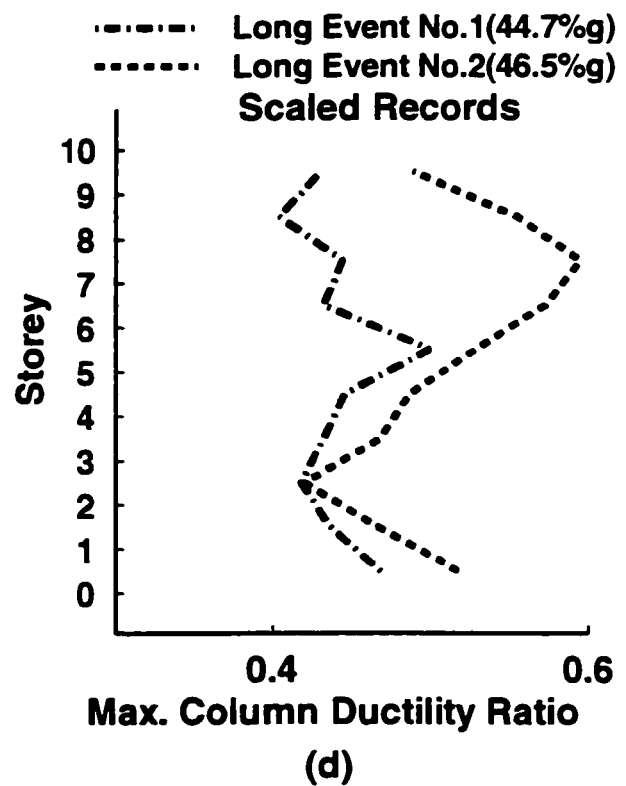
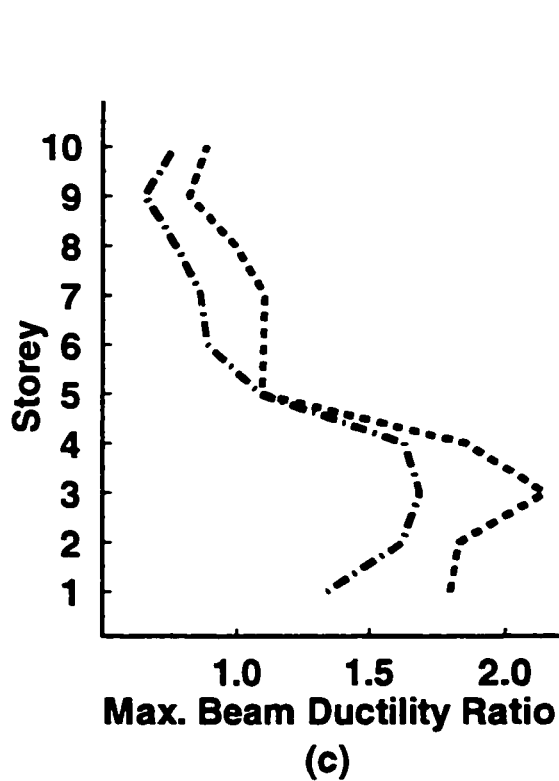
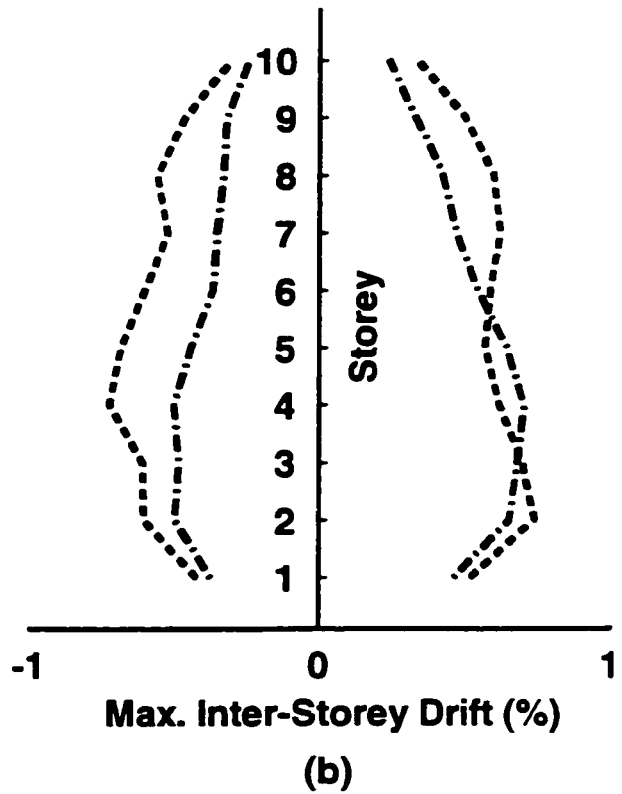
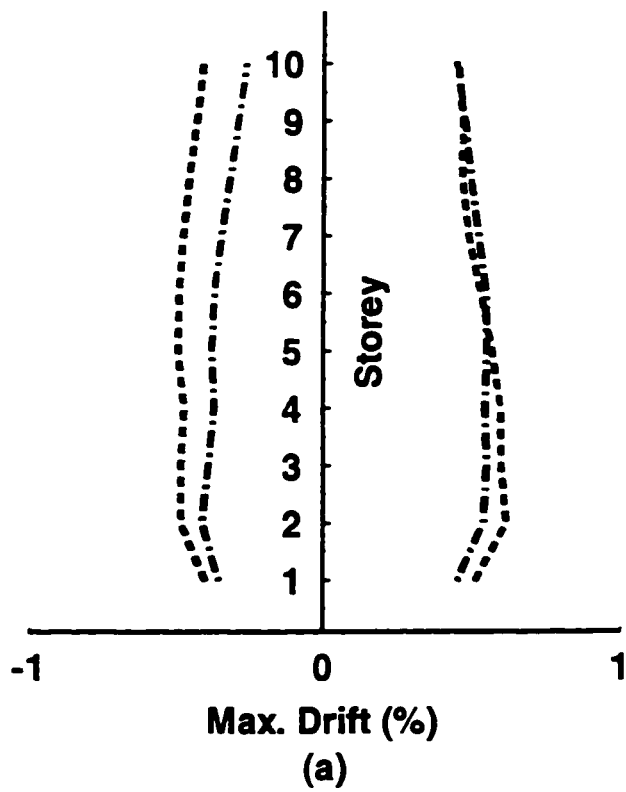


Fig. 5.101 Drift and Ductility Response for 10-Storey MRF Subjected to Scaled Artificial Records for Eastern Canada.

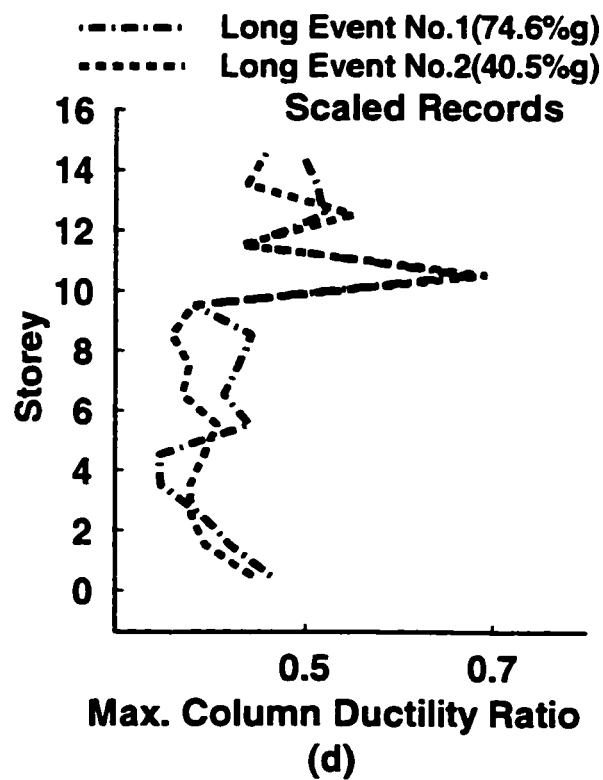
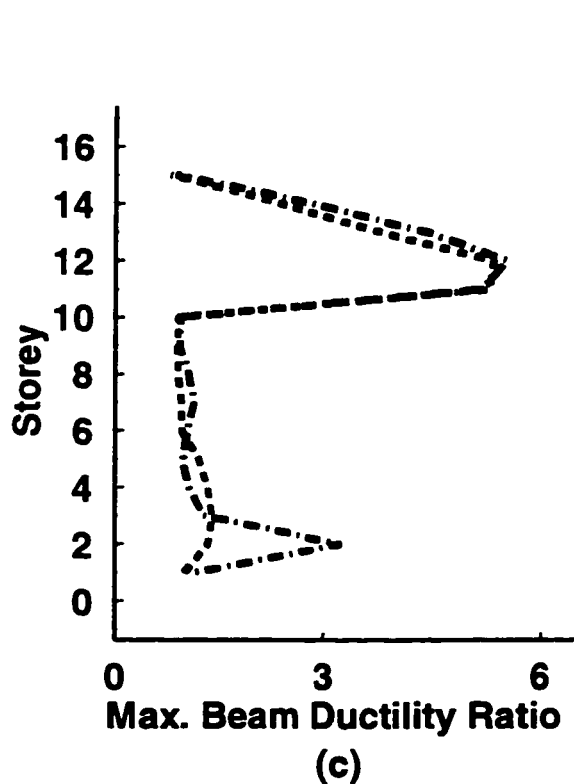
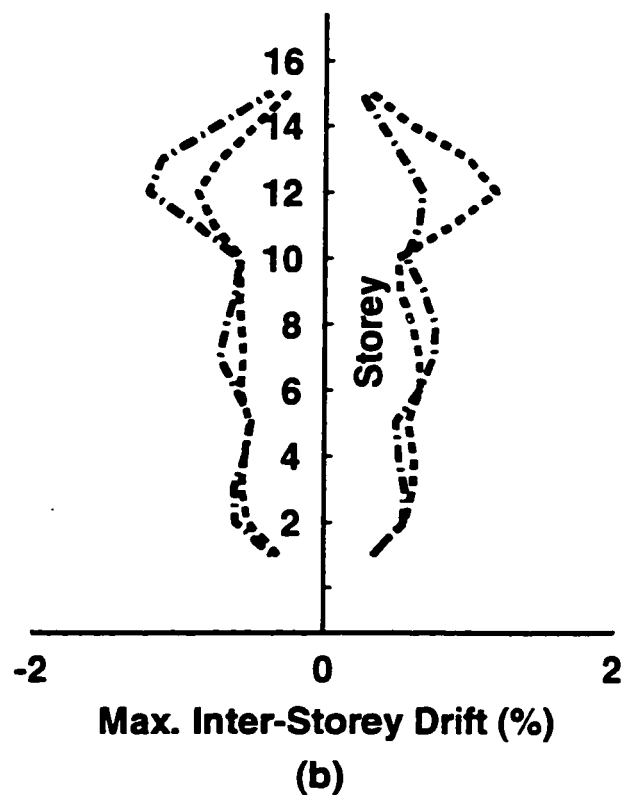
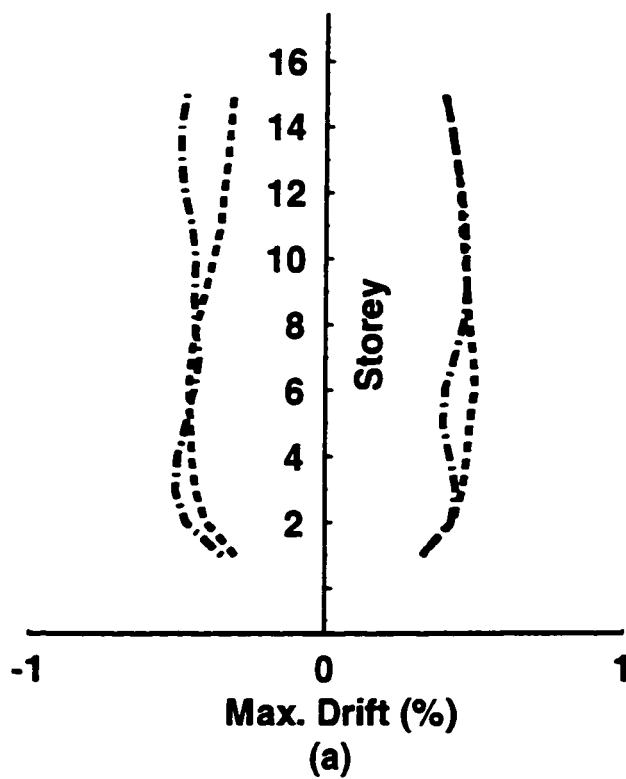
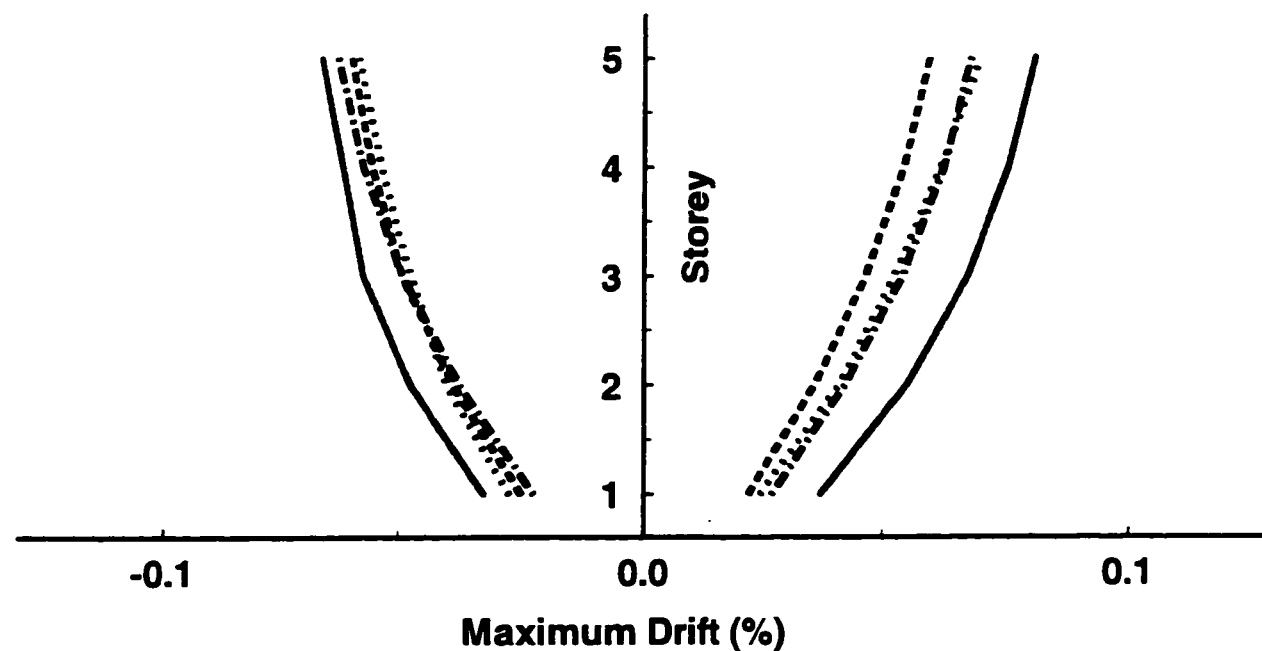


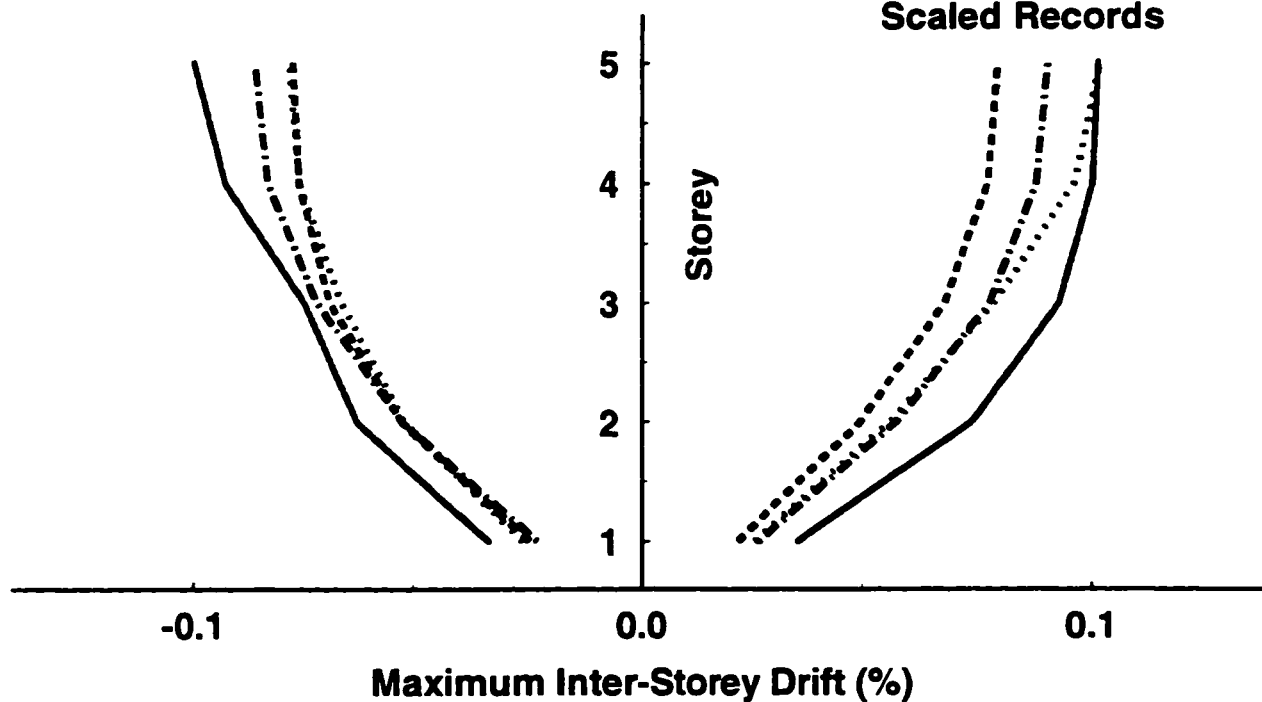
Fig. 5.102 Drift and Ductility Response for 15-Storey MRF Subjected to Scaled Artificial Records for Eastern Canada.



(a)

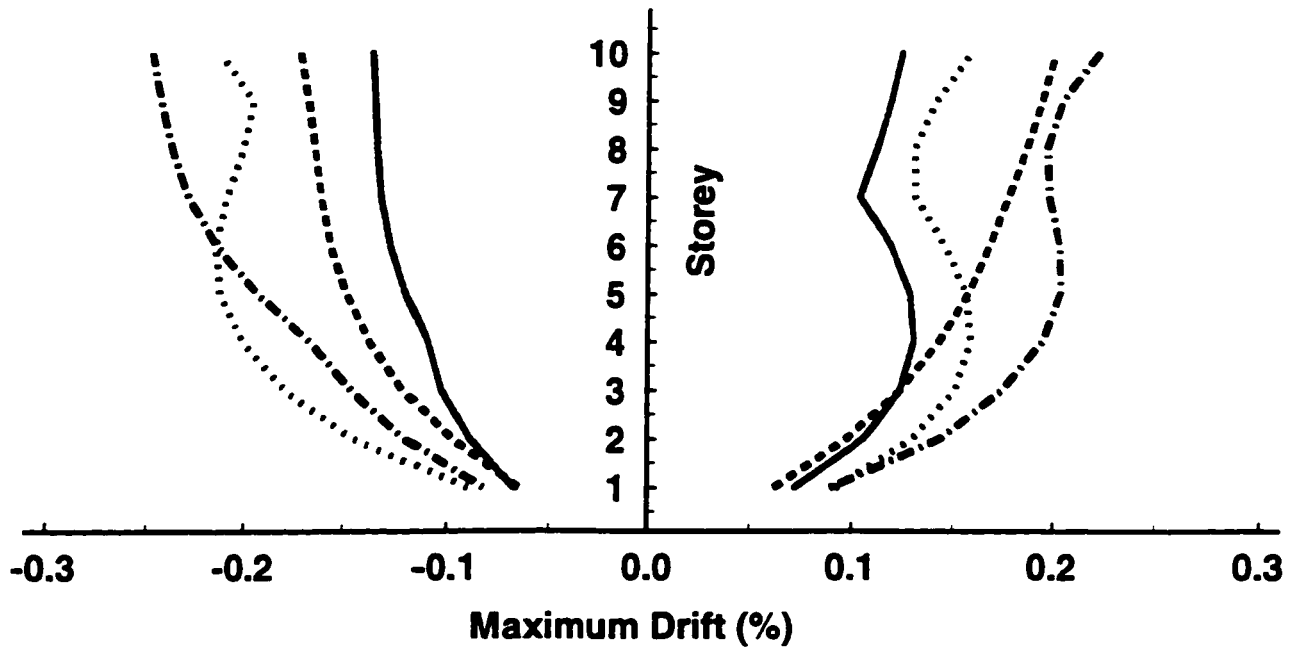
- Short Event No.1 (32.2%g)
- Short Event No.2 (24.0%g)
- .-.- Long Event No.1 (10.4%g)
- Long Event No.2 (7.8%g)

Scaled Records



(b)

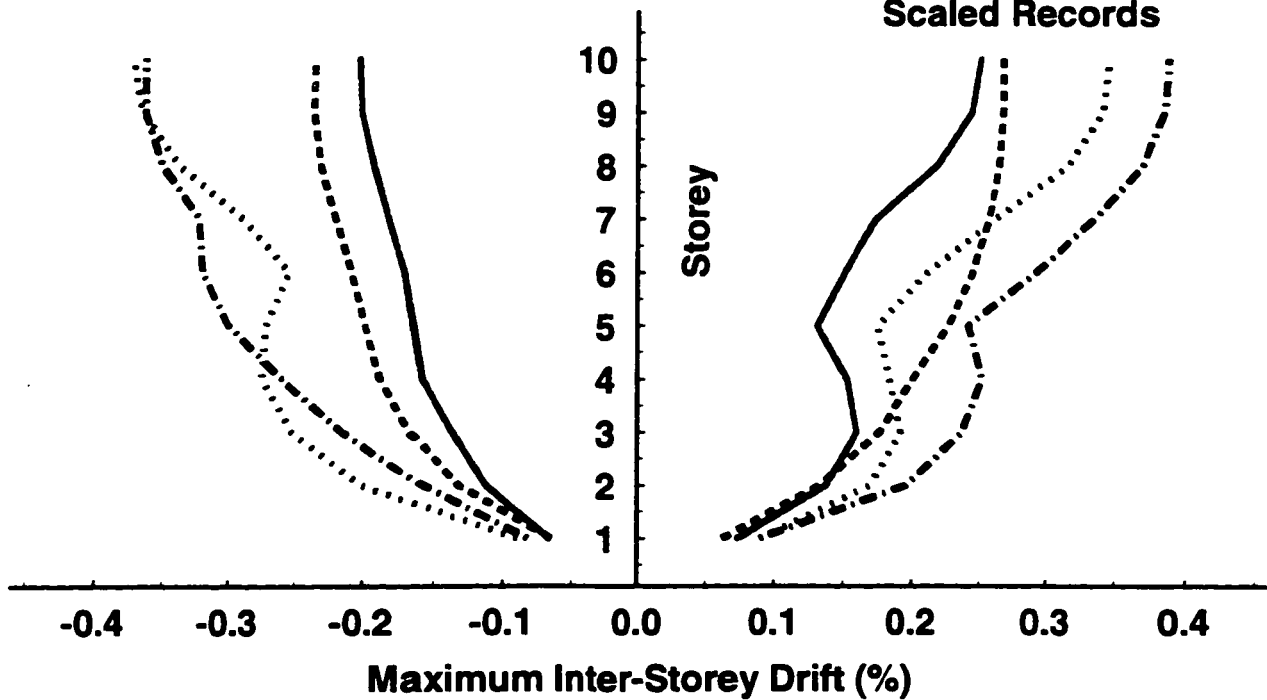
Fig. 5.103 Drift Response for 5-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.



(a)

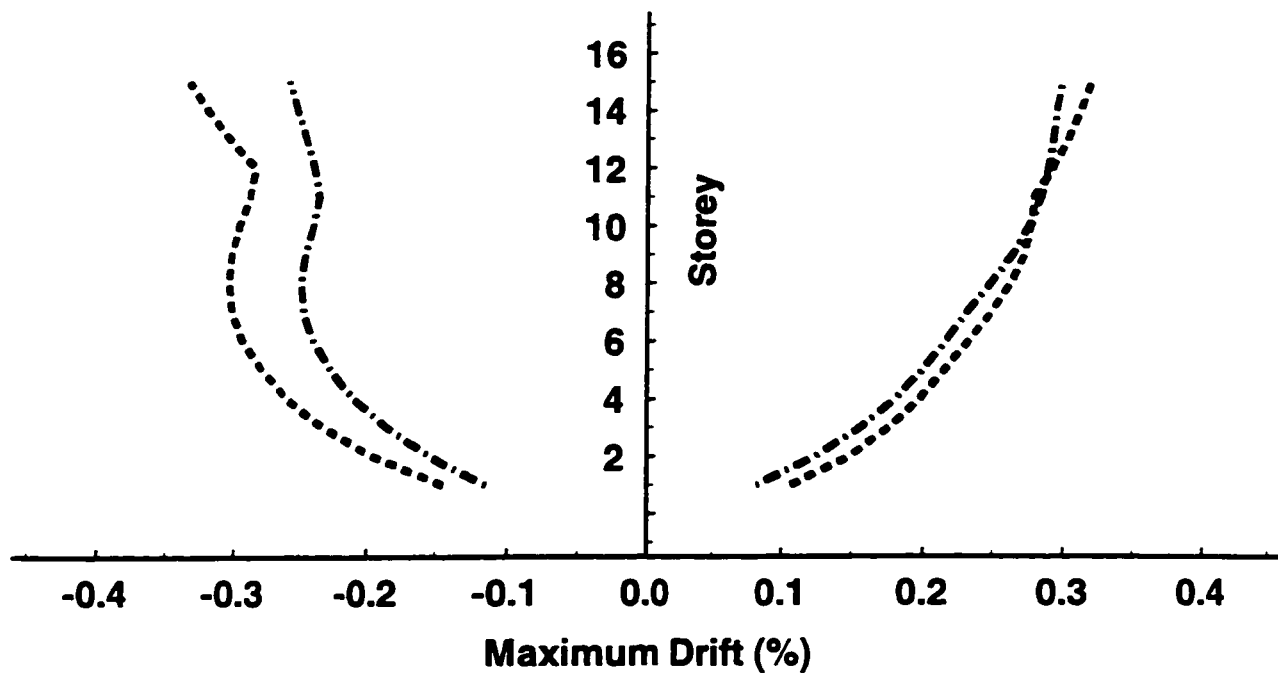
- Short Event No.1(52.8%g)
- Short Event No.2(79.8%g)
- .-.- Long Event No.1 (28.9%g)
- Long Event No.2 (10.1%g)

Scaled Records



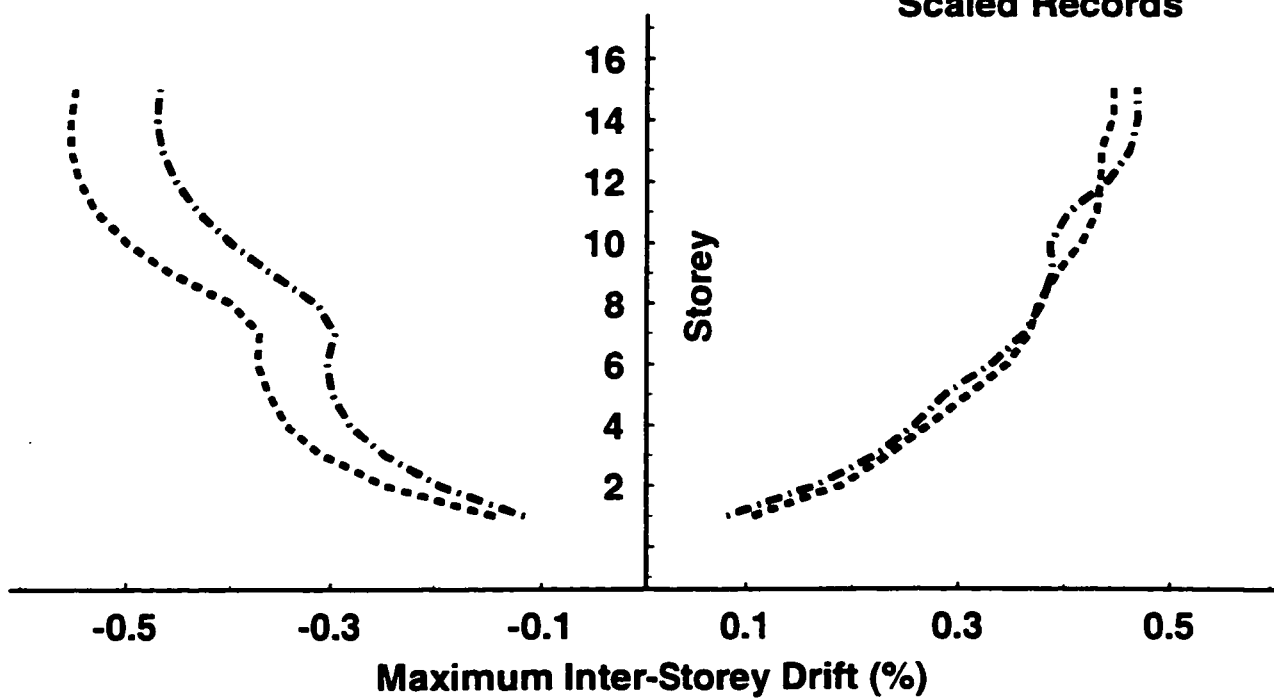
(b)

Fig. 5.104 Drift Response for 10-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.



(a)

--- Long Event No.1 (47.0%g)
 Long Event No.2 (45.8%g)
Scaled Records



(b)

Fig. 5.105 Drift Response for 15-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.

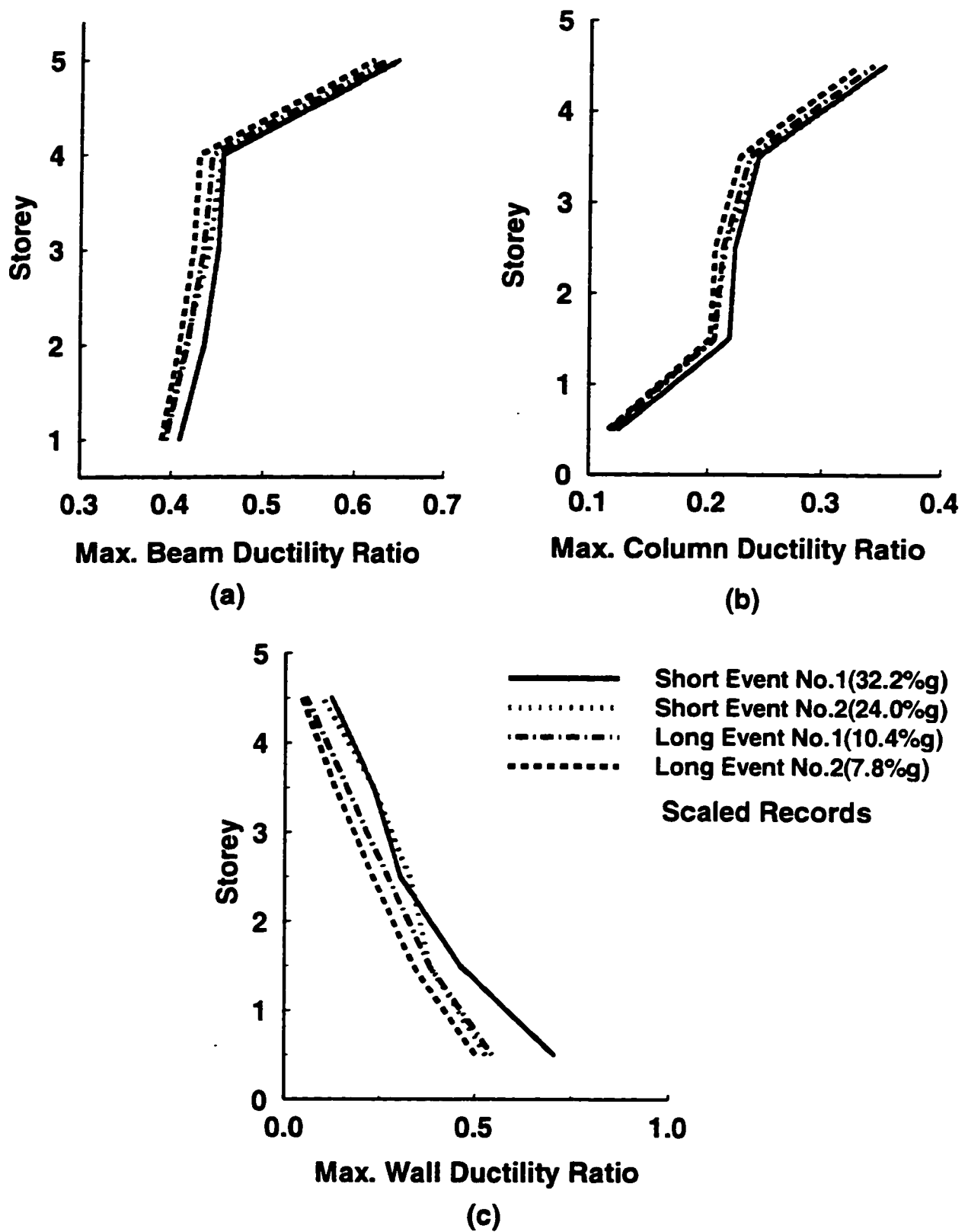


Fig. 5.106 Ductility Response for 5-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.

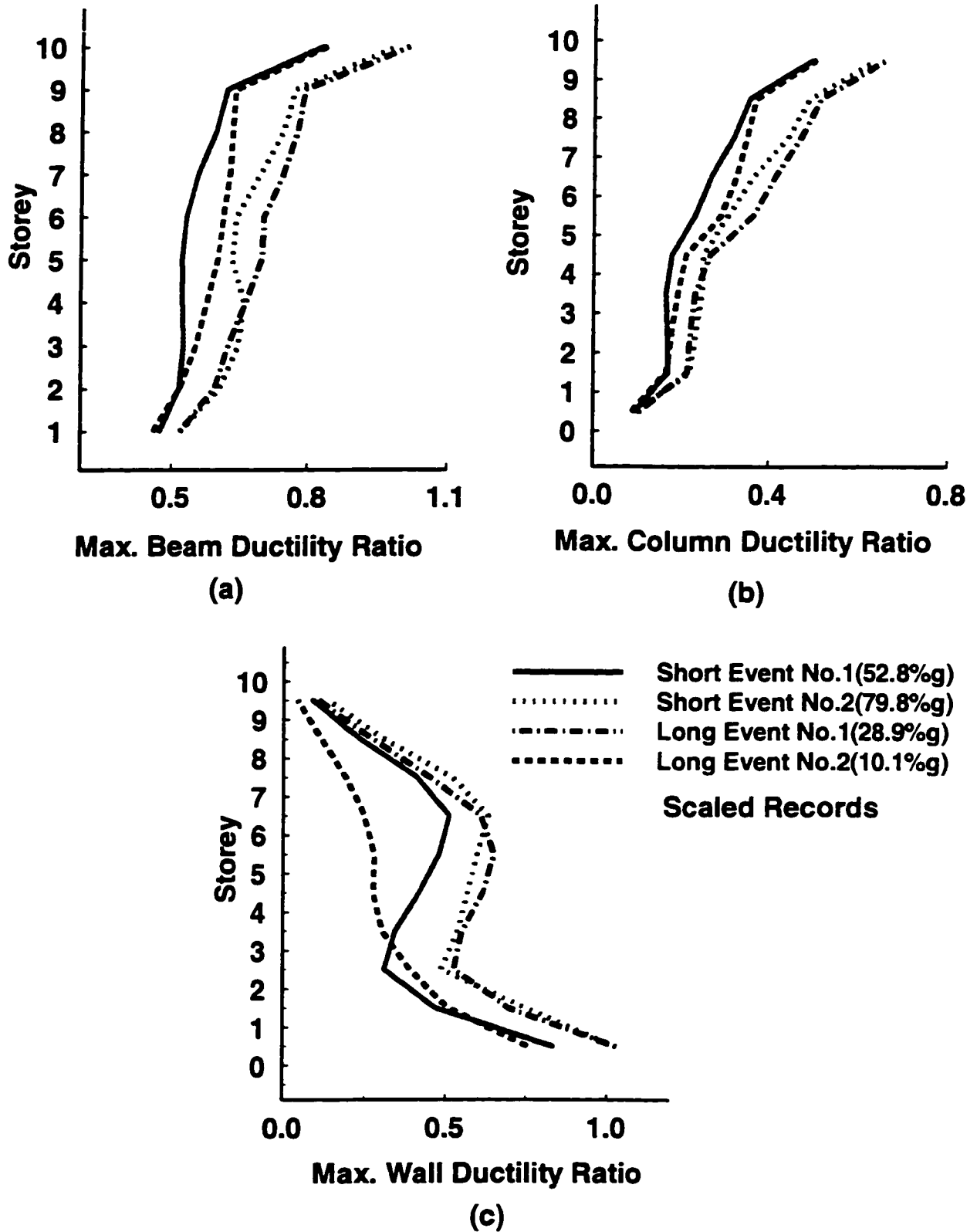


Fig. 5.107 Ductility Response for 10-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.

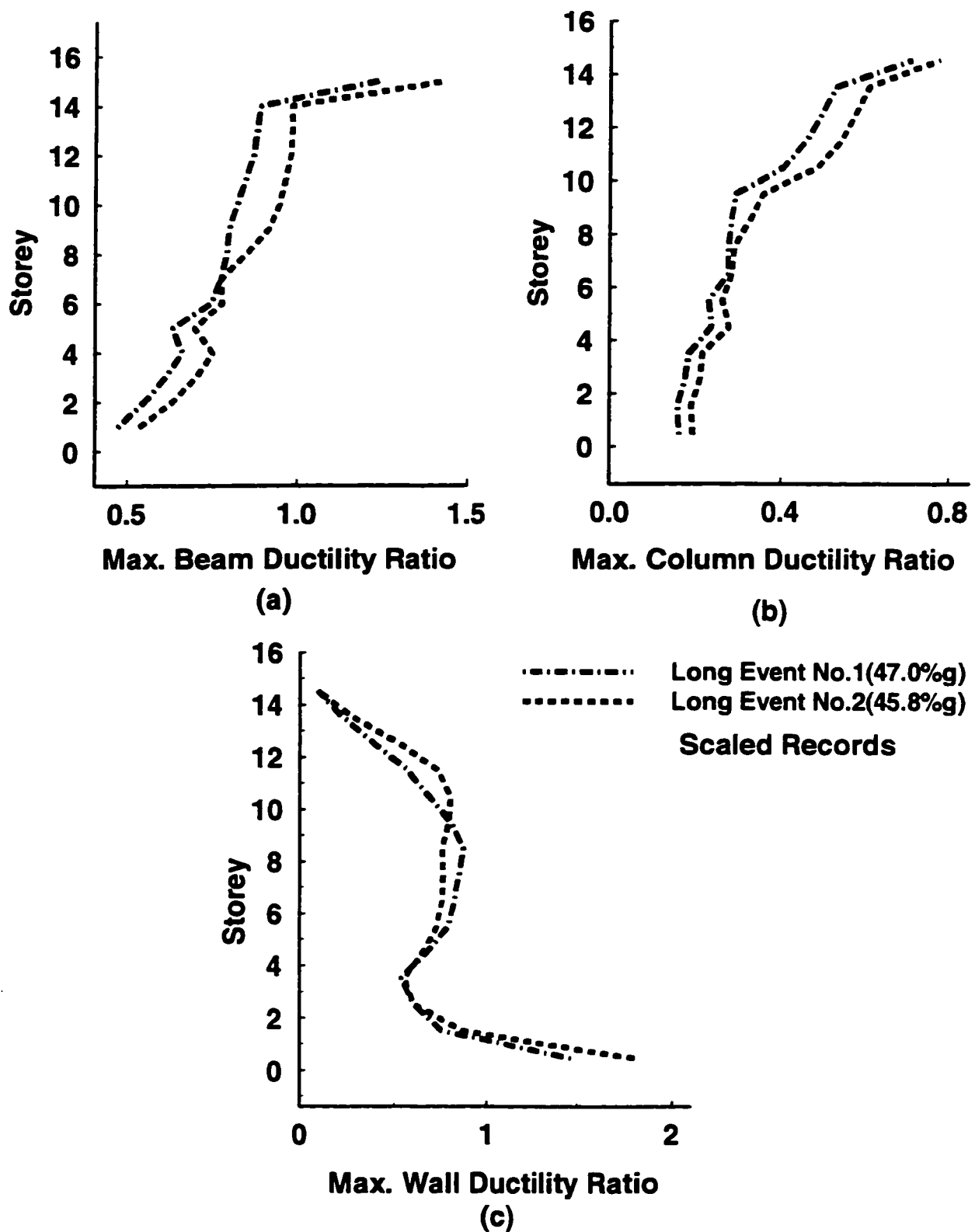


Fig. 5.108 Ductility Response for 15-Storey SHW Subjected to Scaled Artificial Records for Eastern Canada.

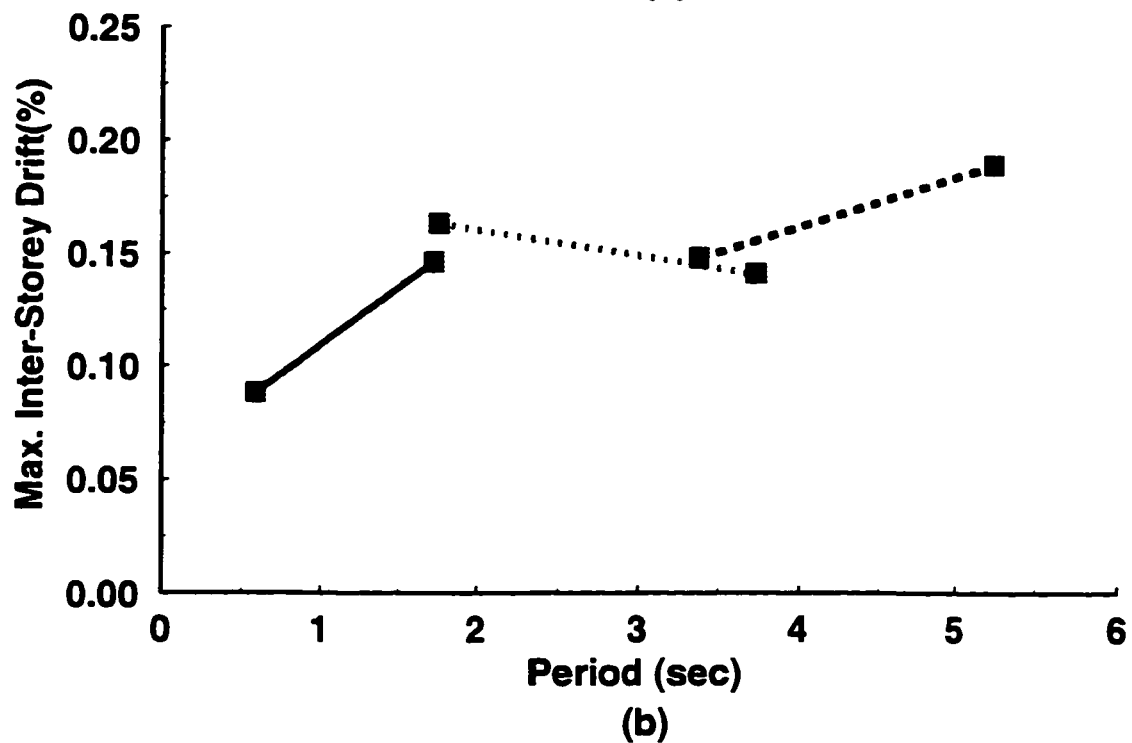
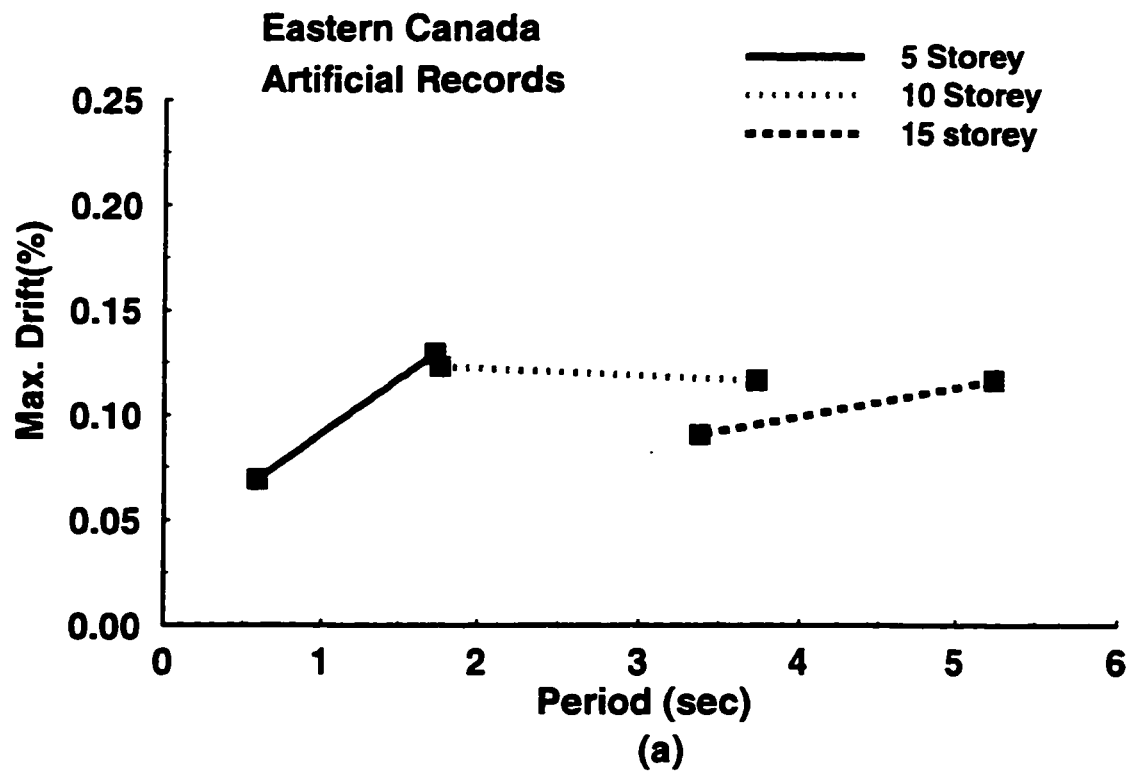


Fig. 5.109 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Artificial Records for Eastern Canada.

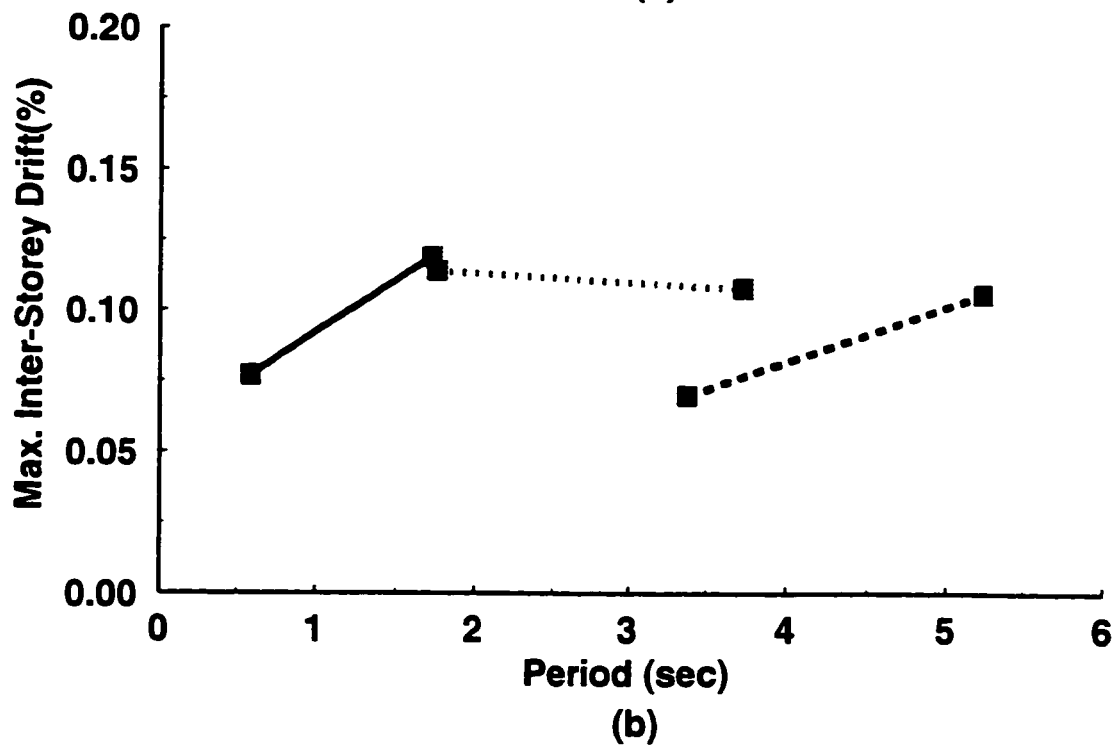
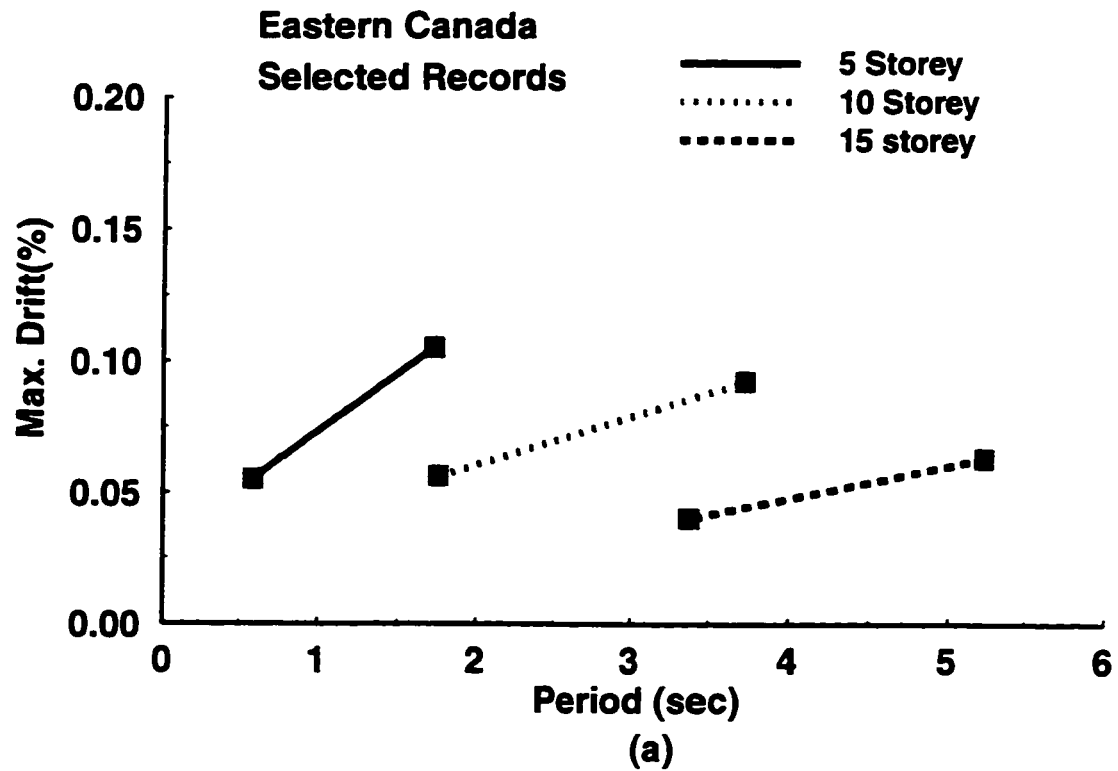


Fig. 5.110 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Selected Record for Eastern Canada.

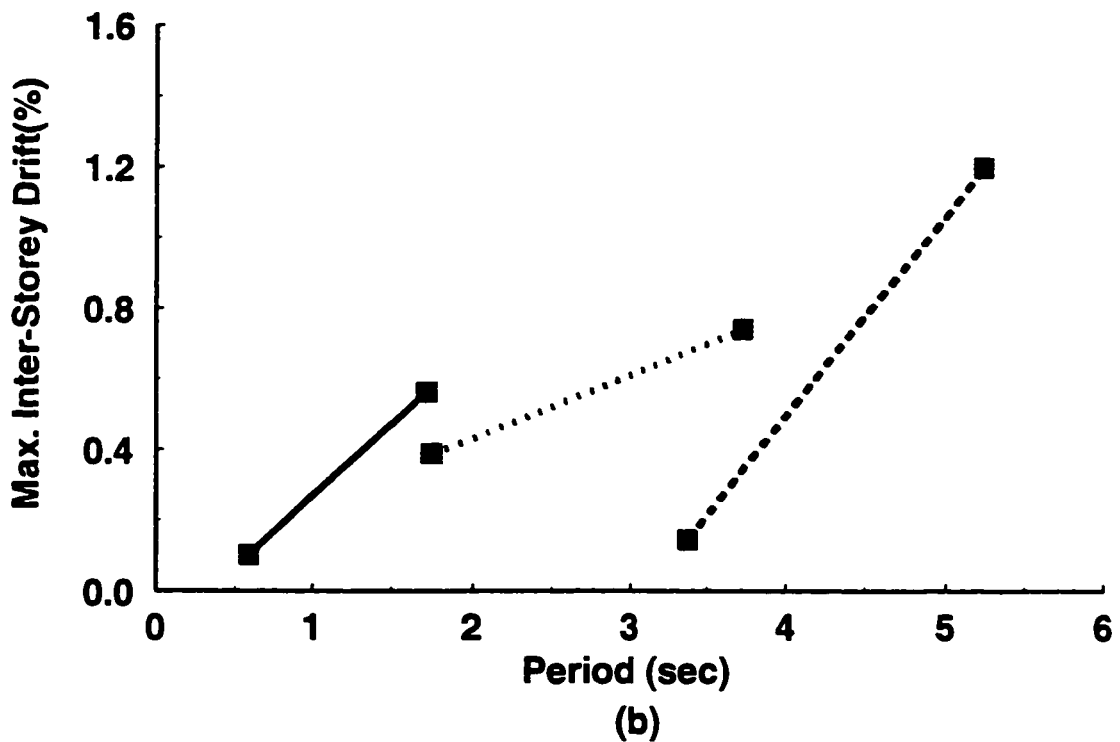
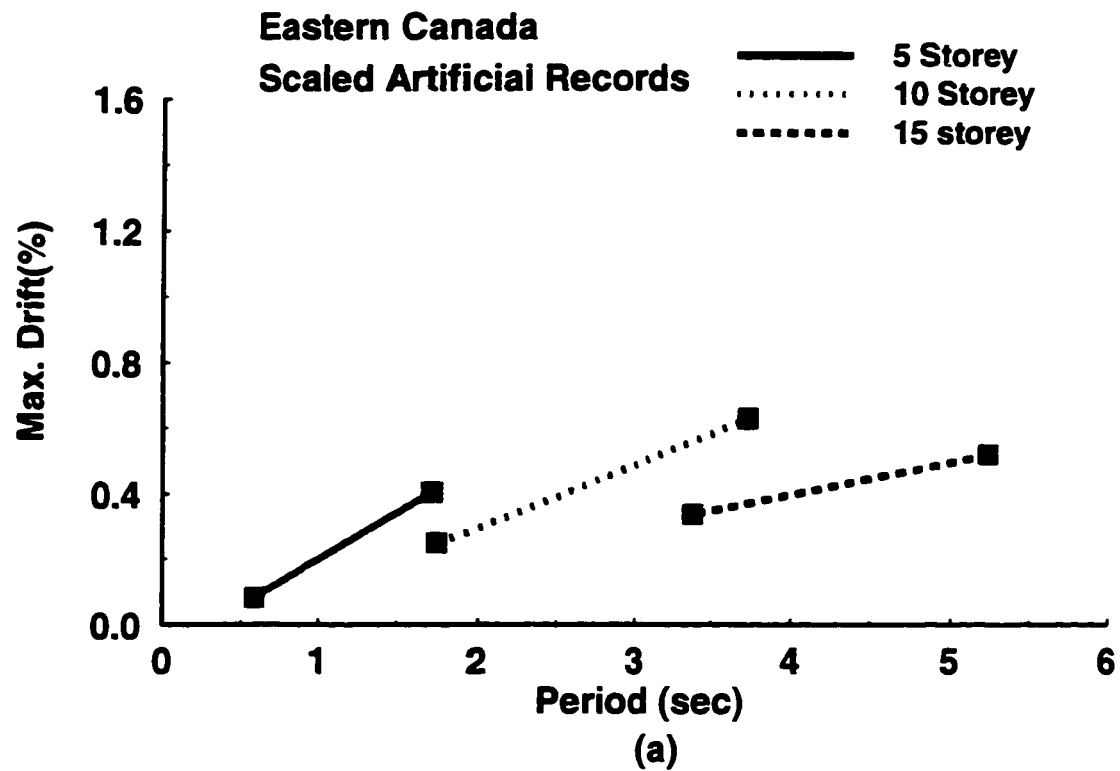


Fig. 5.111 The Relationship Between Maximum Drift and The Fundamental Period of Buildings Subjected to Scaled Artificial Records for Eastern Canada.

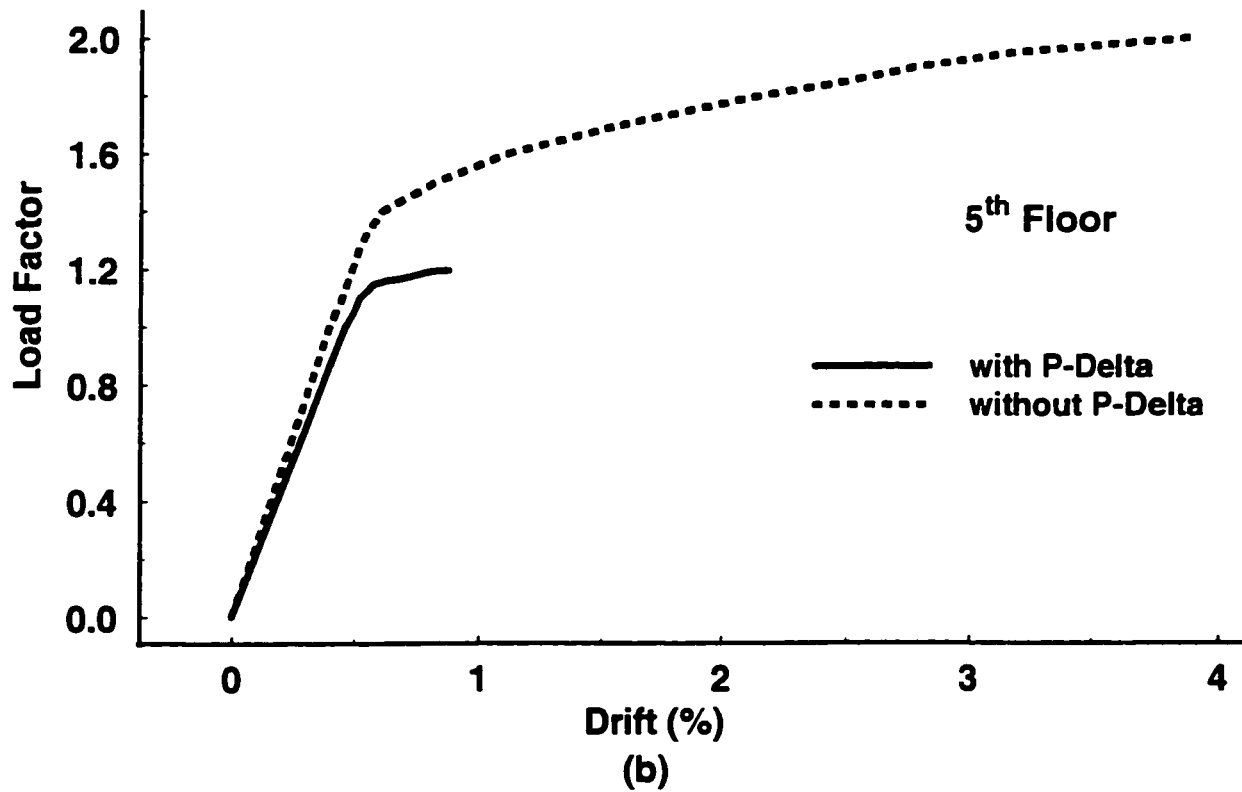
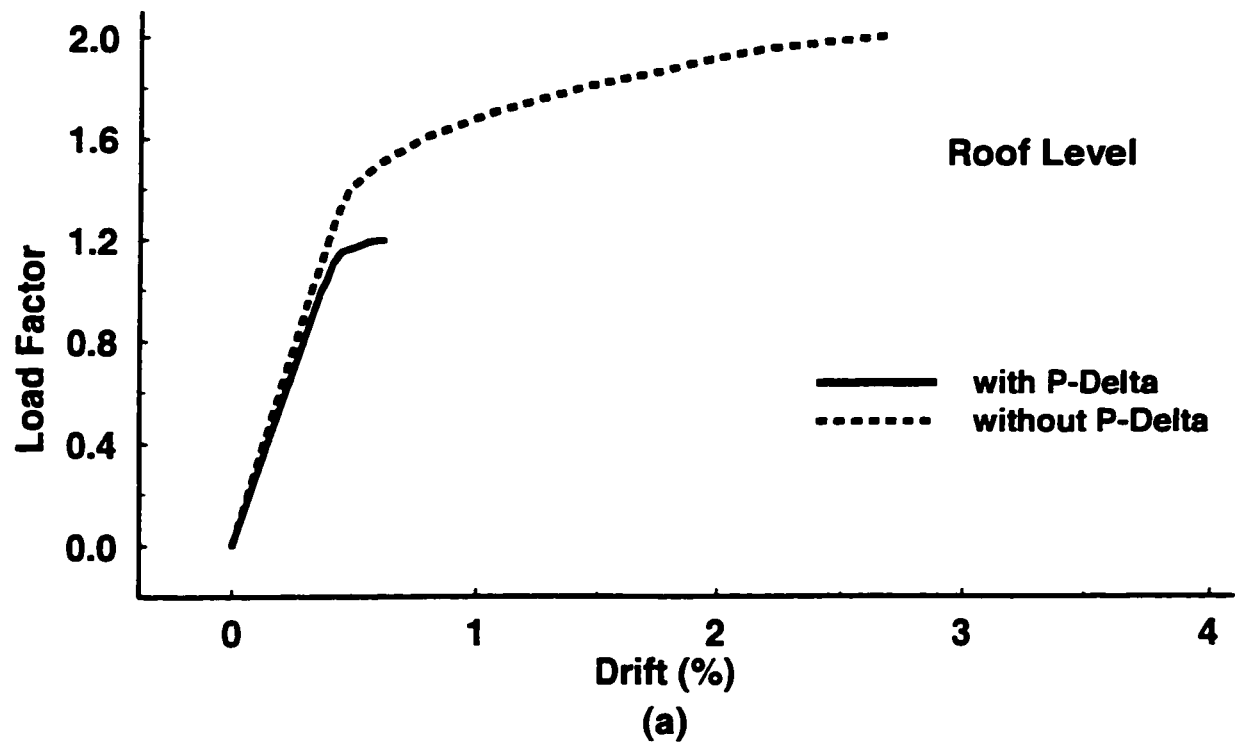


Fig. 6.1 Force-Lateral Drift Relationship for 10-Storey MRF building in Vancouver.

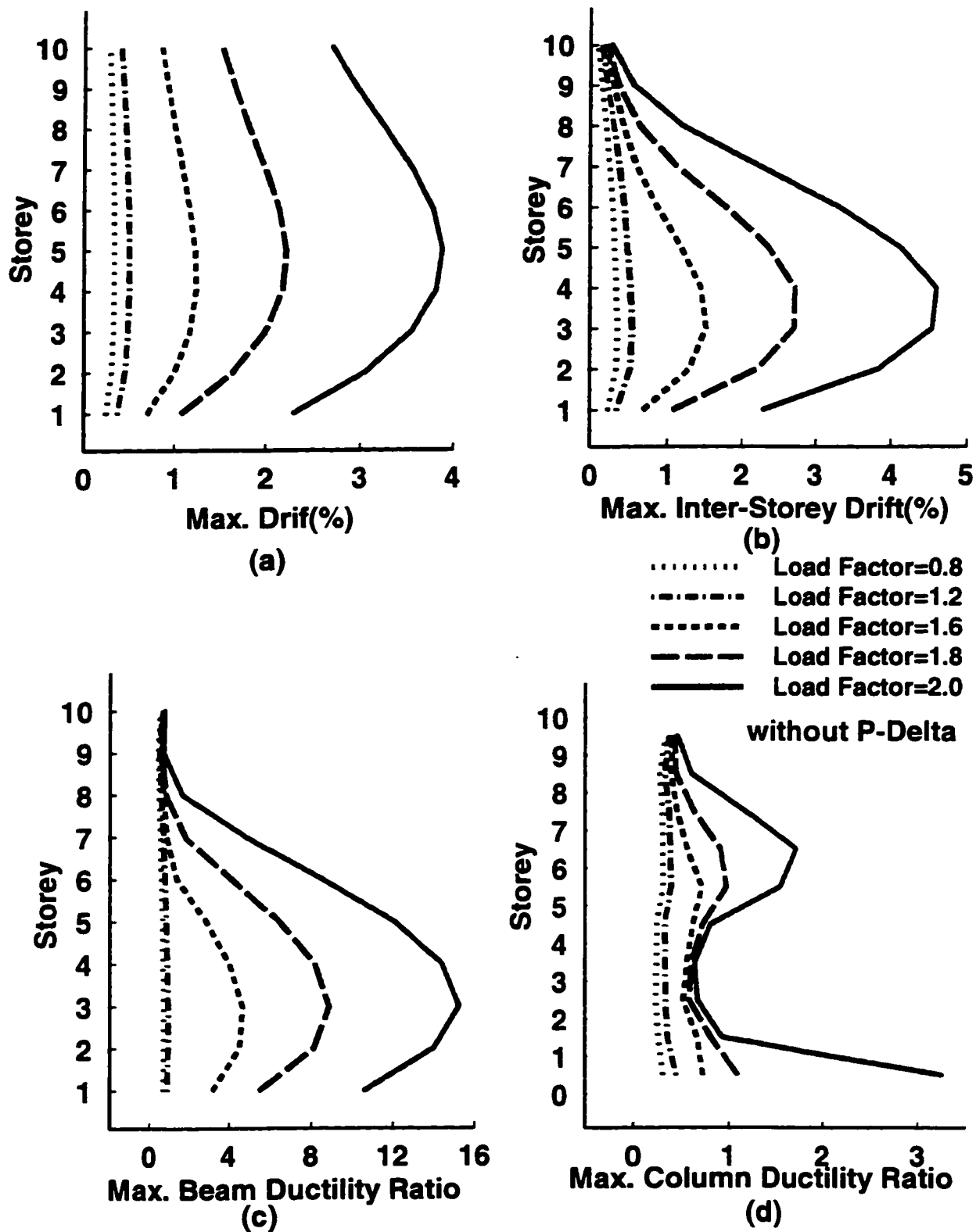
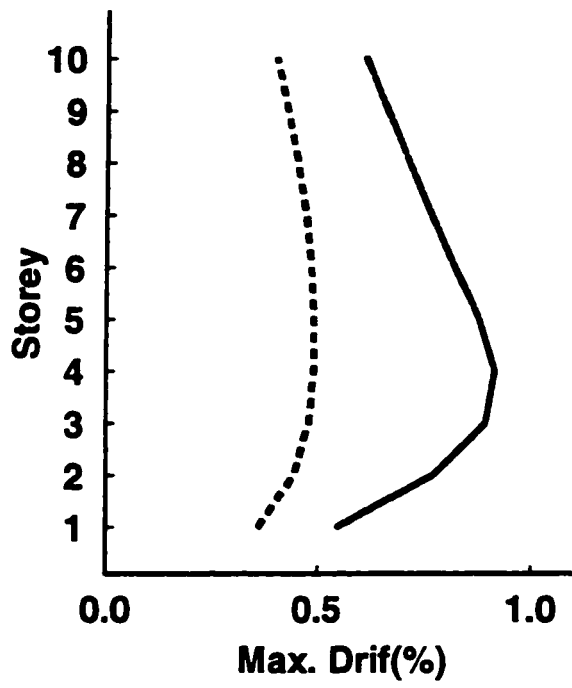
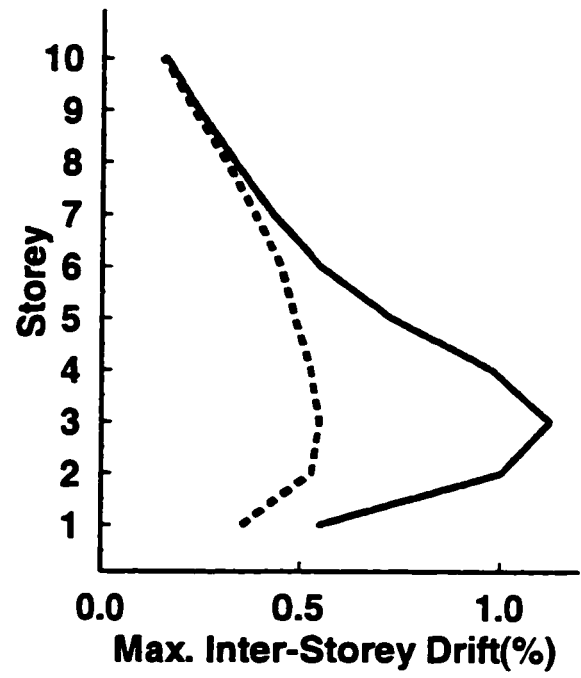


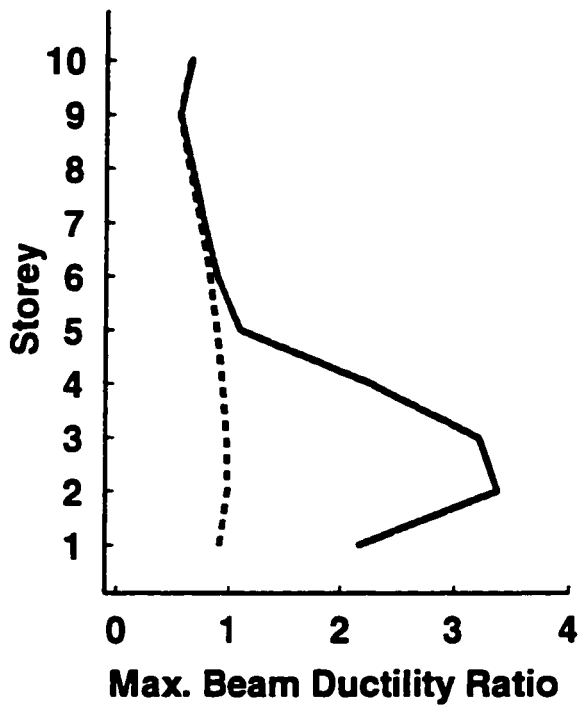
Fig. 6.2 Drift and Ductility Response at Selected Stages of Static Loading for 10-Storey MRF building in Vancouver.



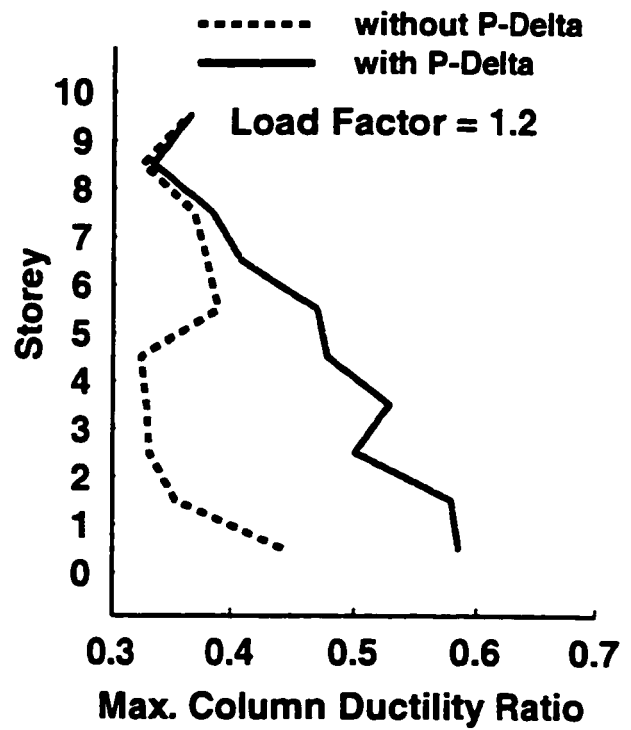
(a)



(b)



(c)



(d)

Fig. 6.3 The effect of P- Δ in Push-Over Analysis for 10-Storey MRF building in Vancouver.

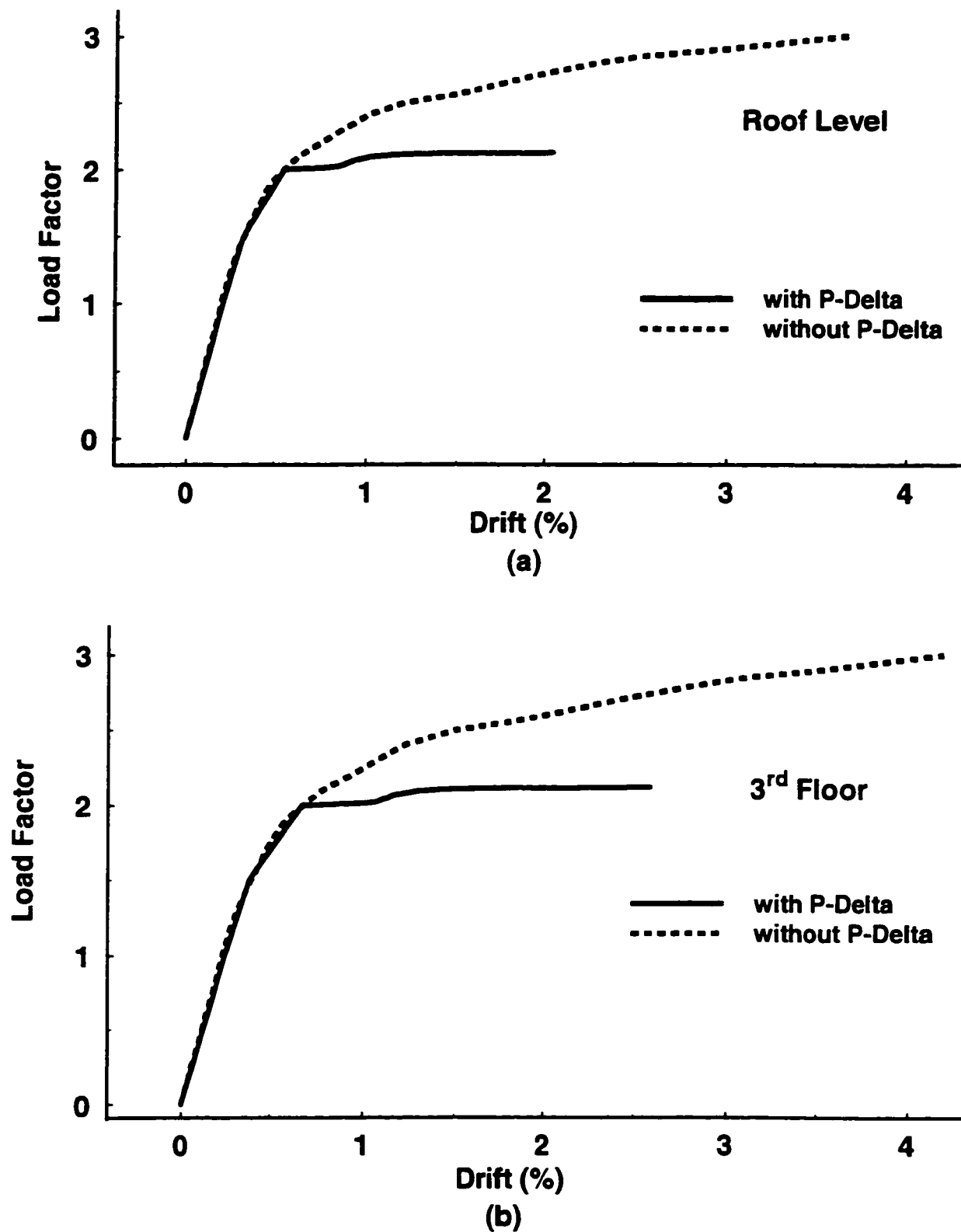
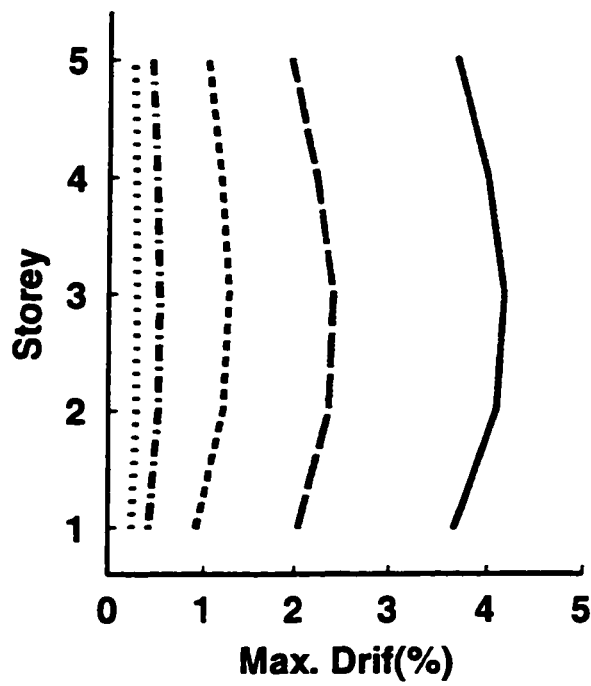
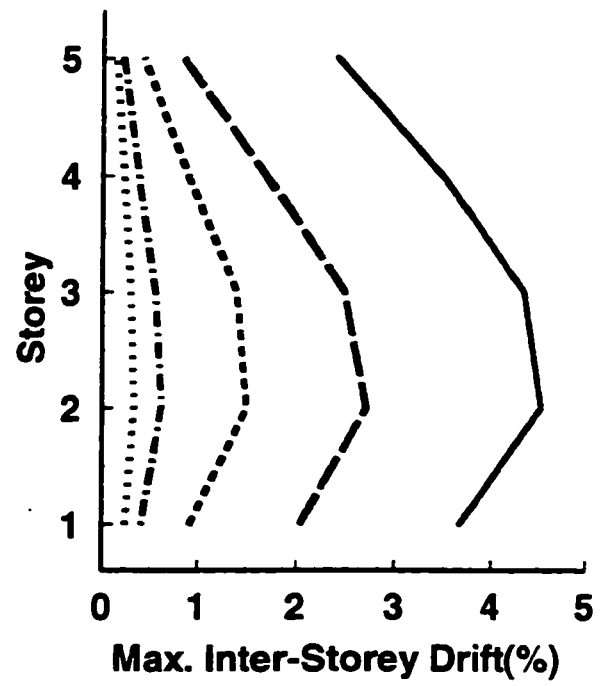


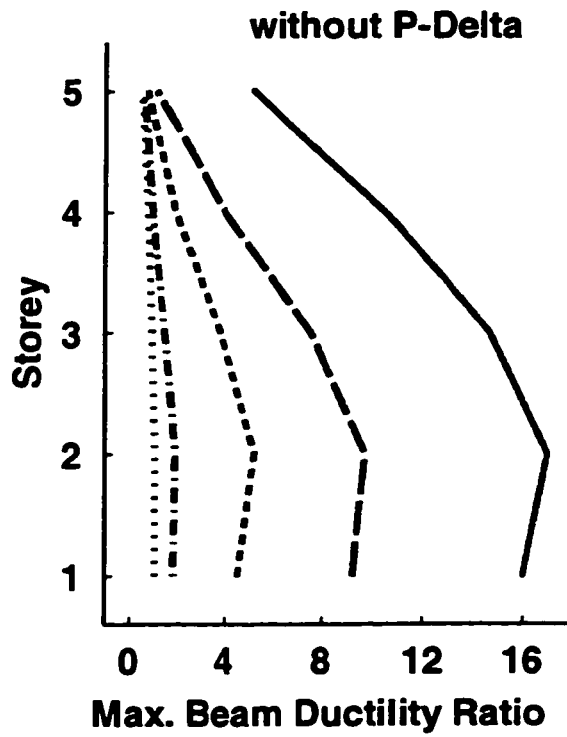
Fig. 6.4 Force-Lateral Drift Relationship for 5-Storey MRF building in Vancouver.



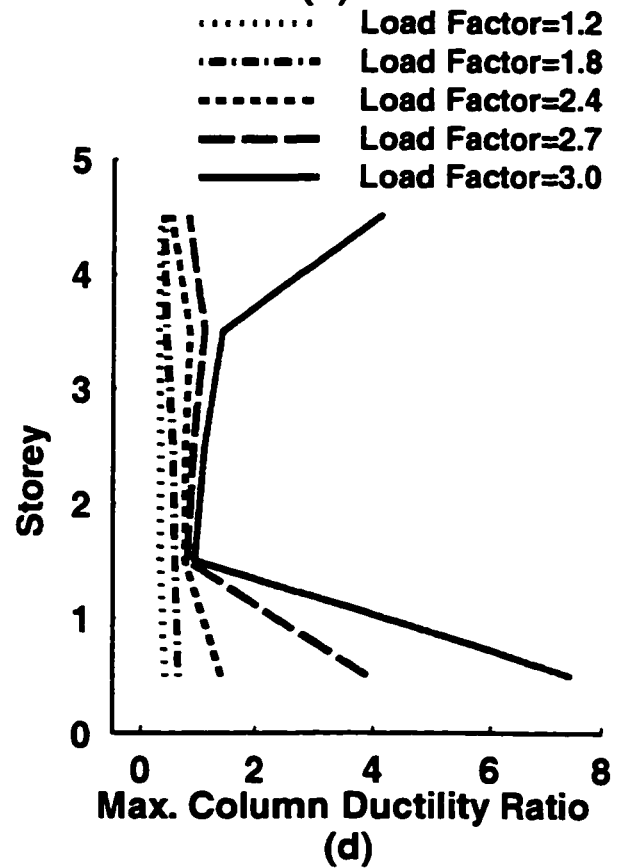
(a)



(b)

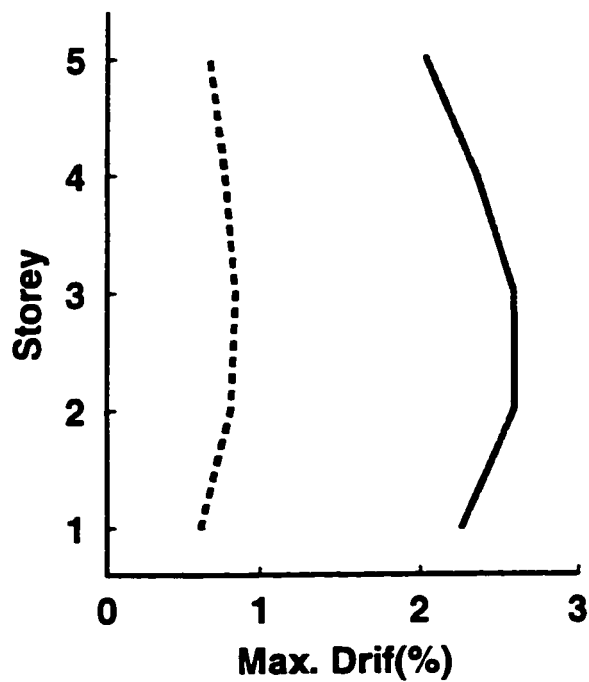


(c)

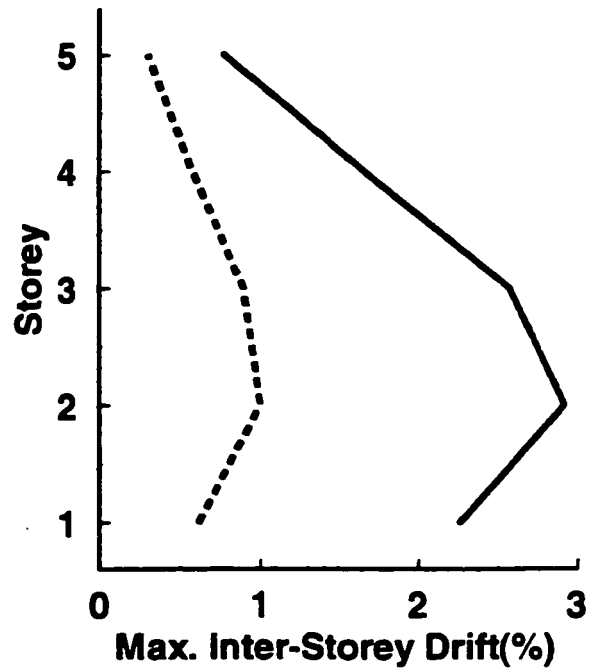


(d)

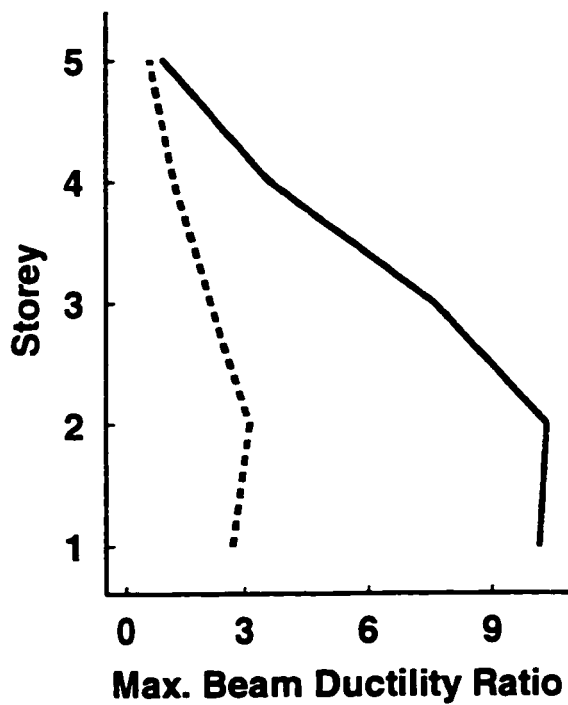
Fig. 6.5 Drift and Ductility Response at Selected Stages of Static Loading for 5-Storey MRF building in Vancouver.



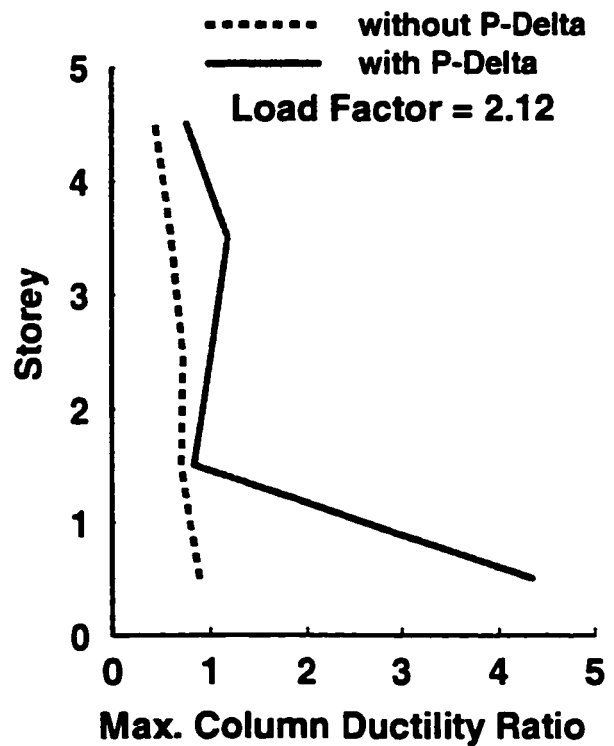
(a)



(b)



(c)



(d)

Fig. 6.6 The effect of P- Δ in Push-Over Analysis for 5-Storey MRF building in Vancouver.

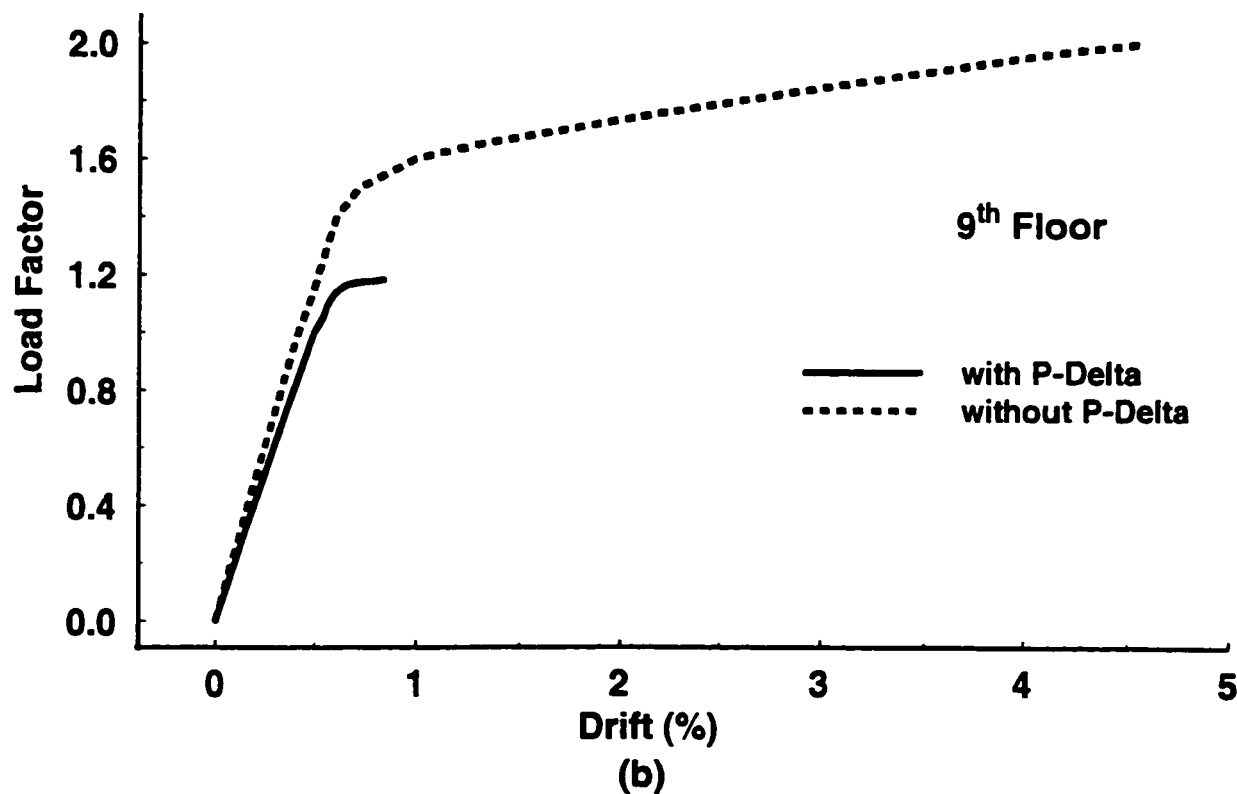
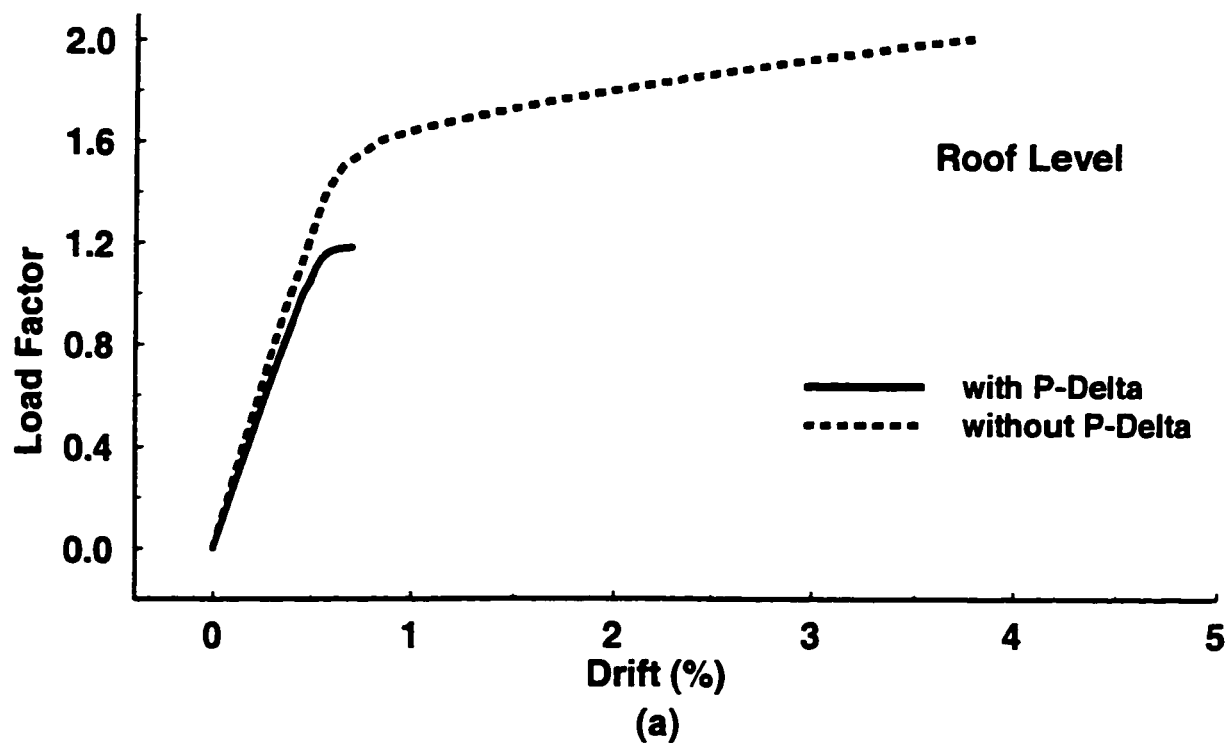
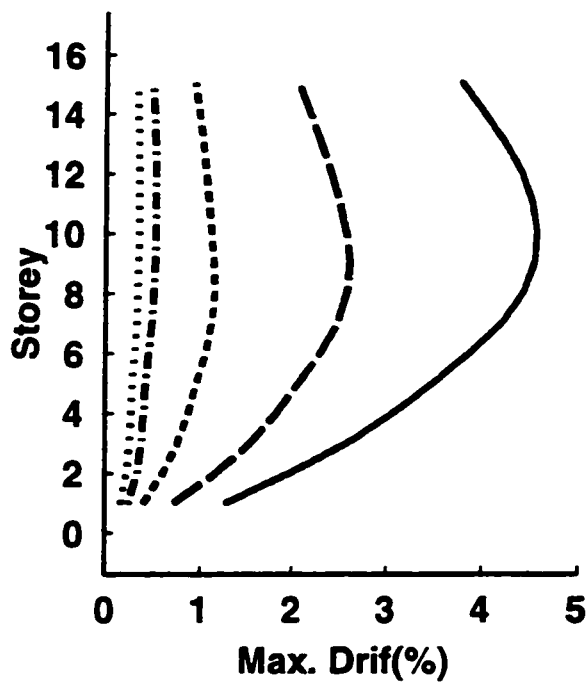
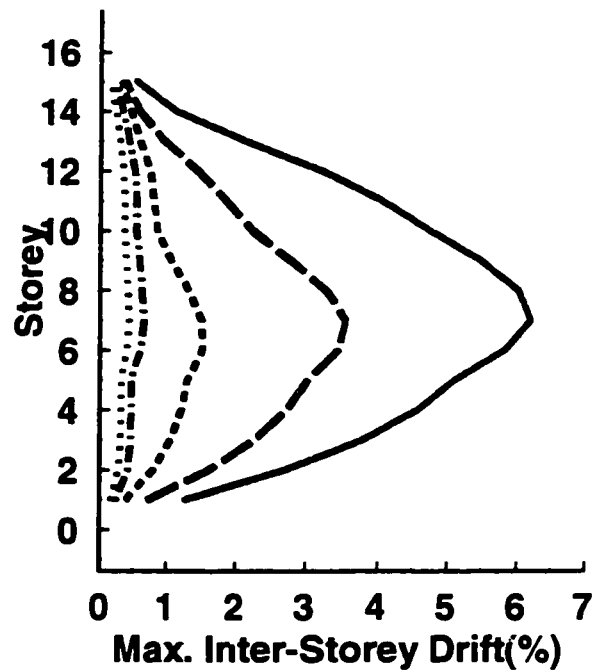


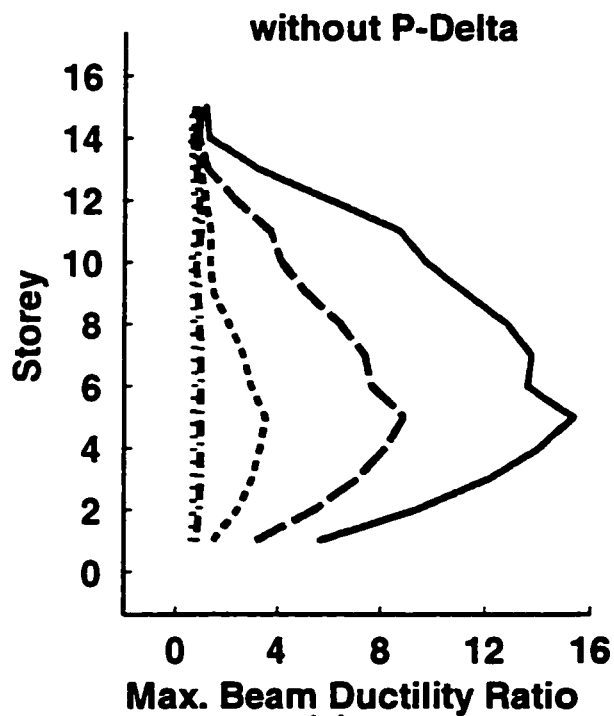
Fig. 6.7 Force-Lateral Drift Relationship for 15-Storey MRF building in Vancouver.



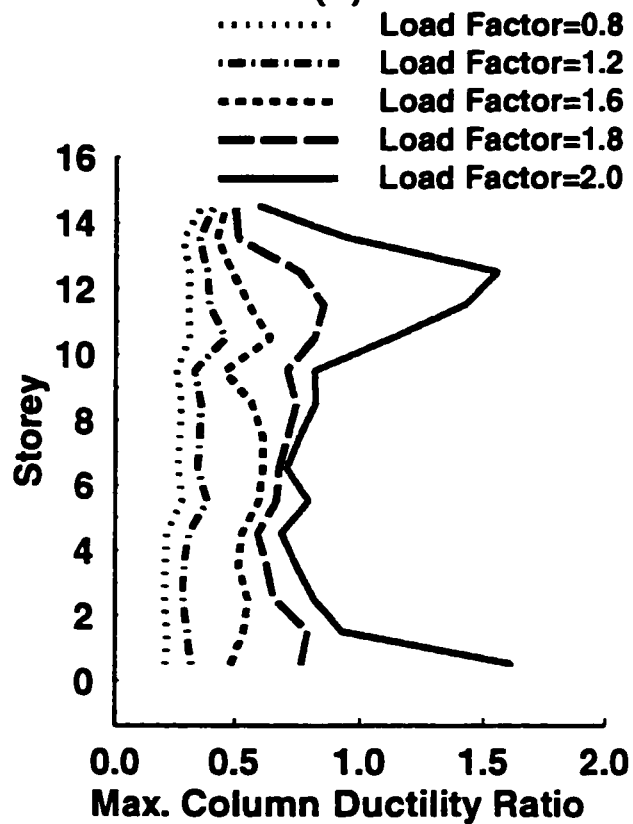
(a)



(b)

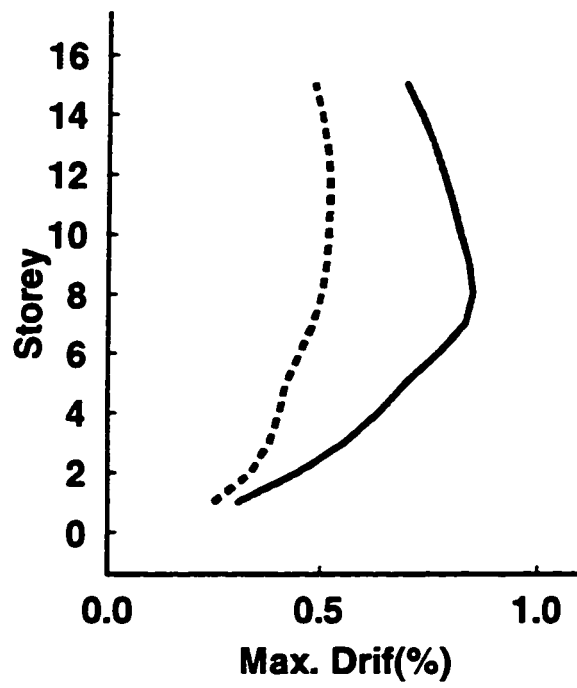


(c)

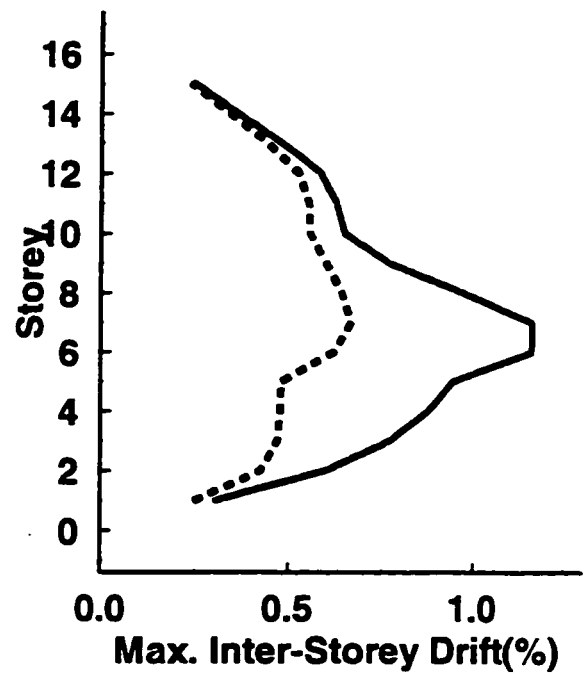


(d)

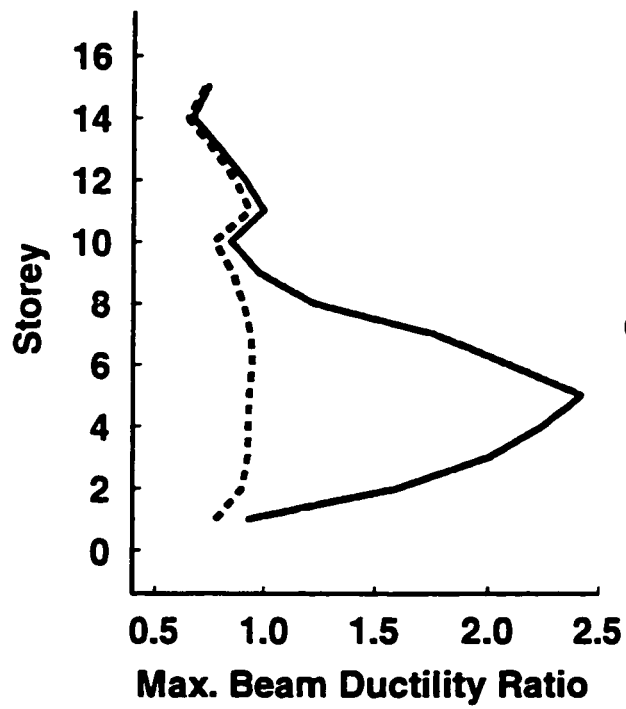
Fig. 6.8 Drift and Ductility Response at Selected Stages of Static Loading for 15-Storey MRF building in Vancouver.



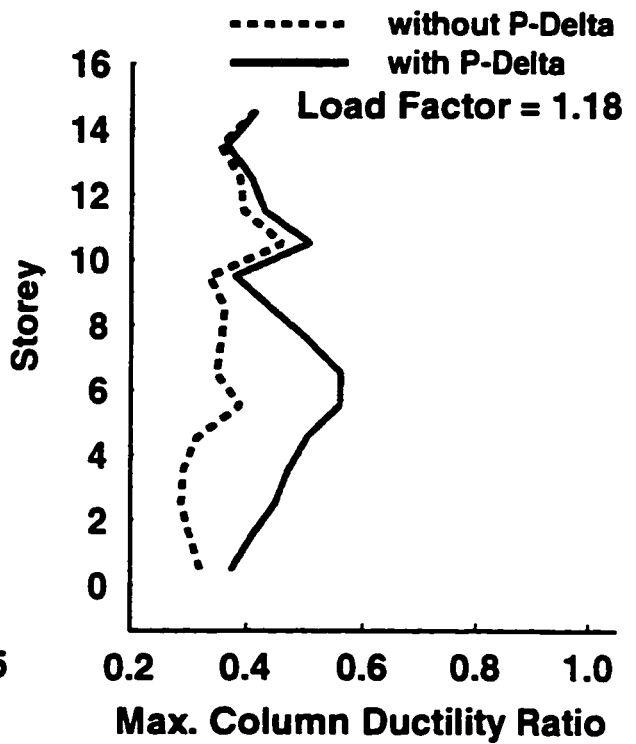
(a)



(b)

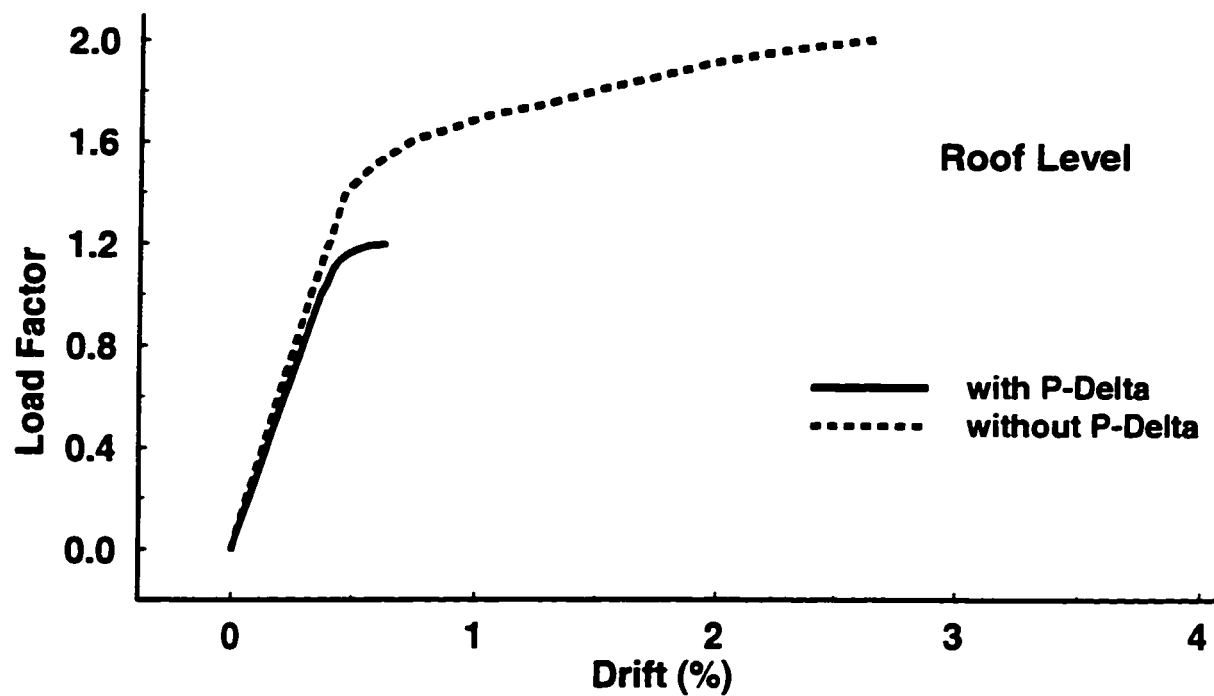


(c)

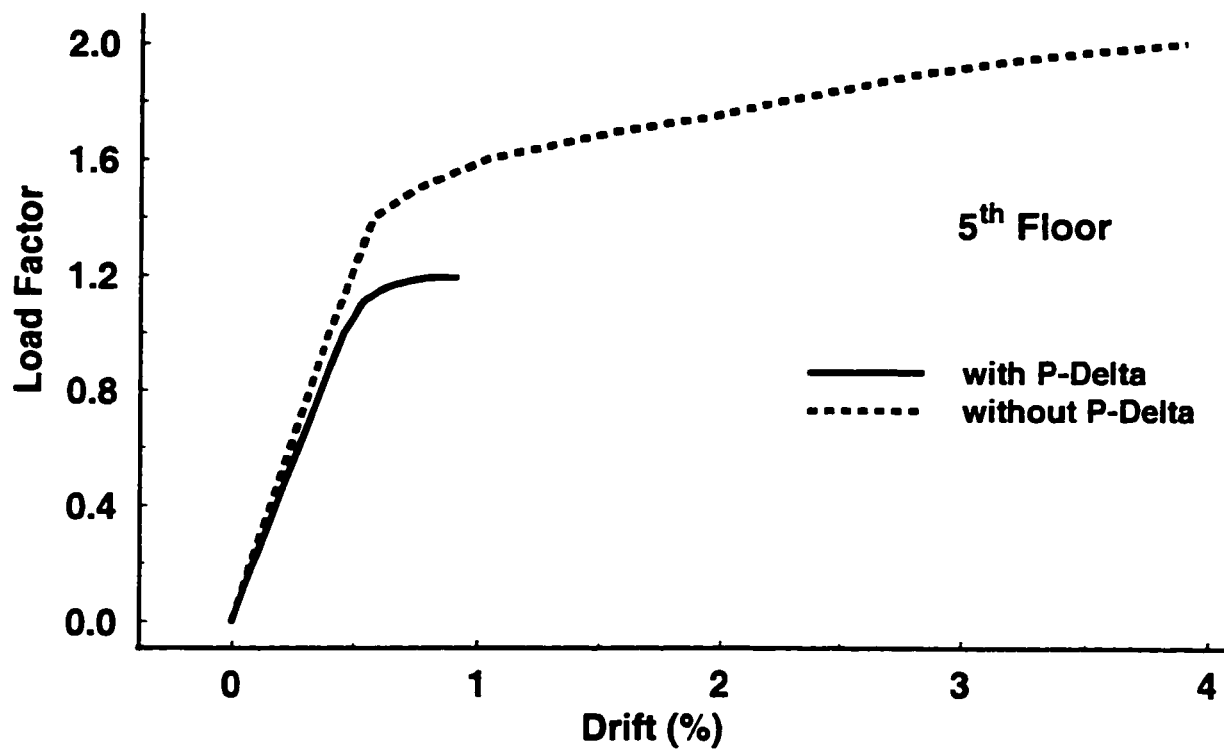


(d)

Fig. 6.9 The effect of P- Δ in Push-Over Analysis for 15-Storey MRF building in Vancouver.



(a)



(b)

Fig. 6.10 Force-Lateral Drift Relationship for 10-Storey MRF building in Ottawa.

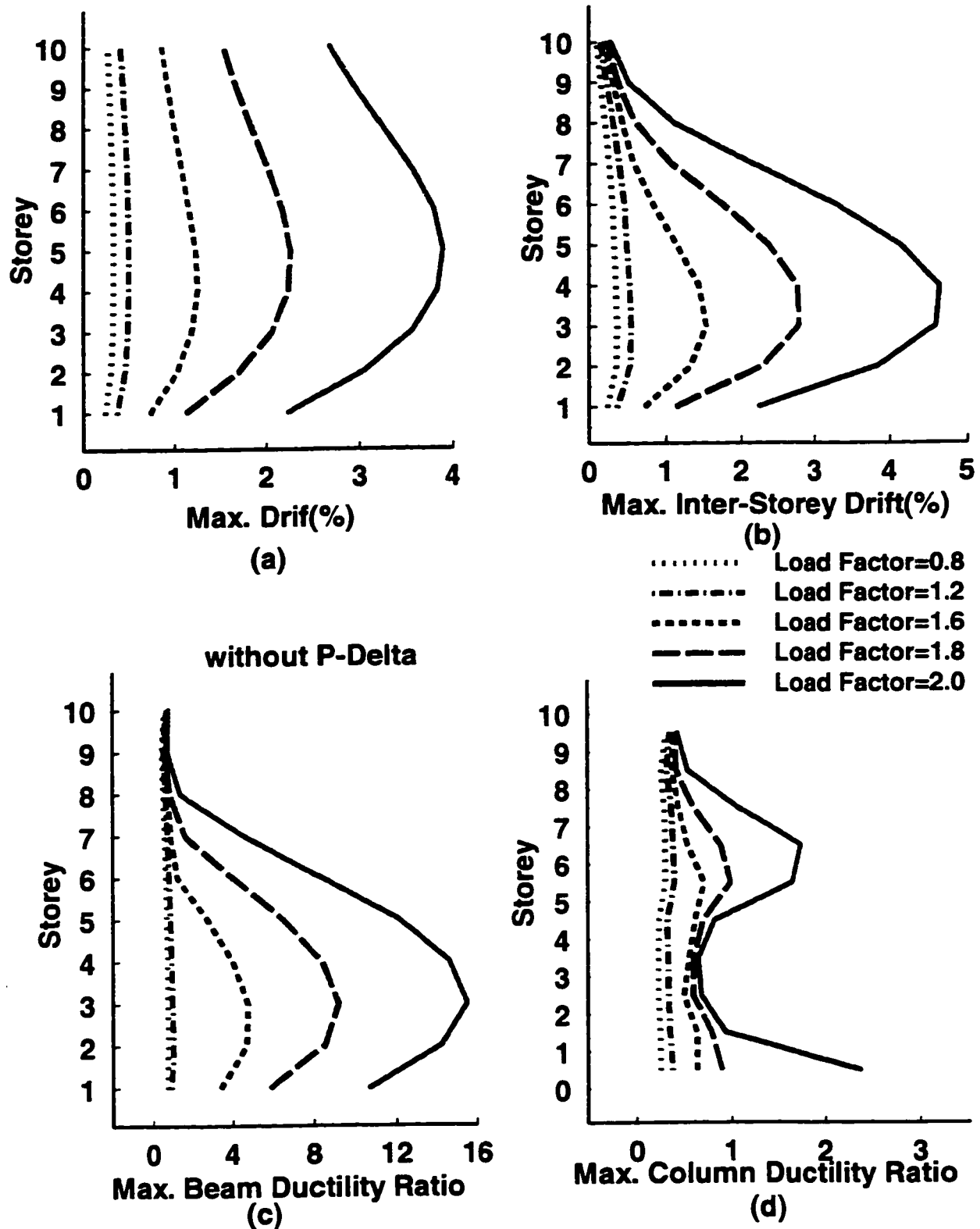
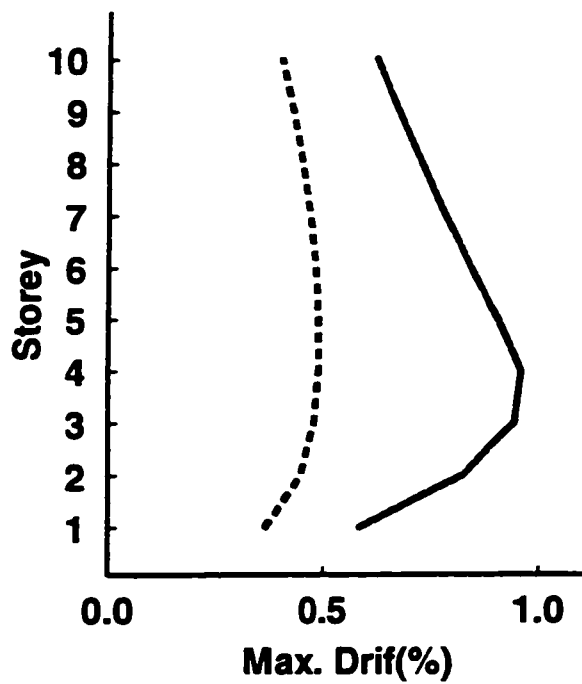
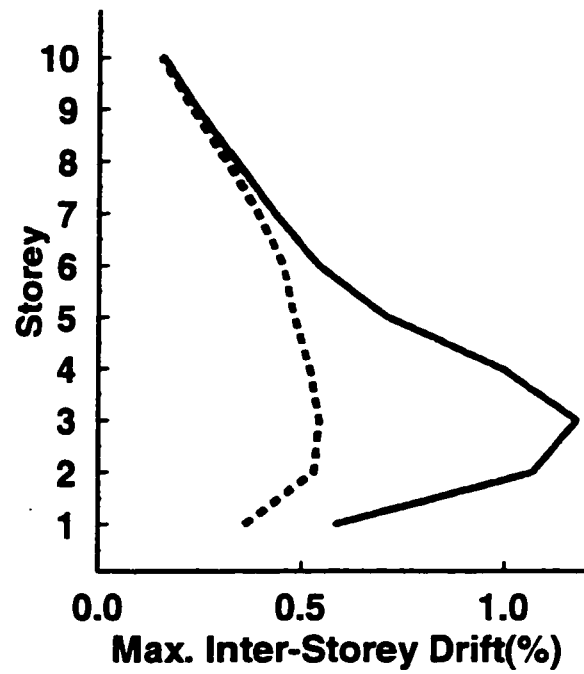


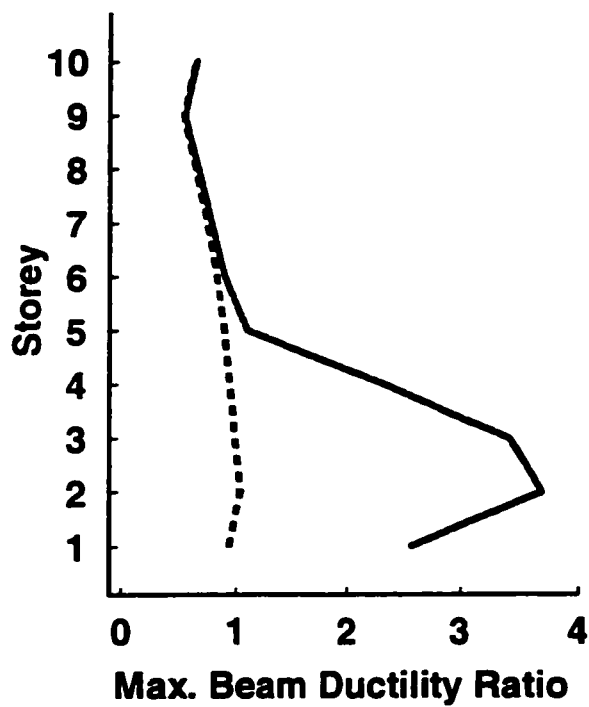
Fig. 6.11 Drift and Ductility Response at Selected Stages of Static Loading for 10-Storey MRF building in Ottawa.



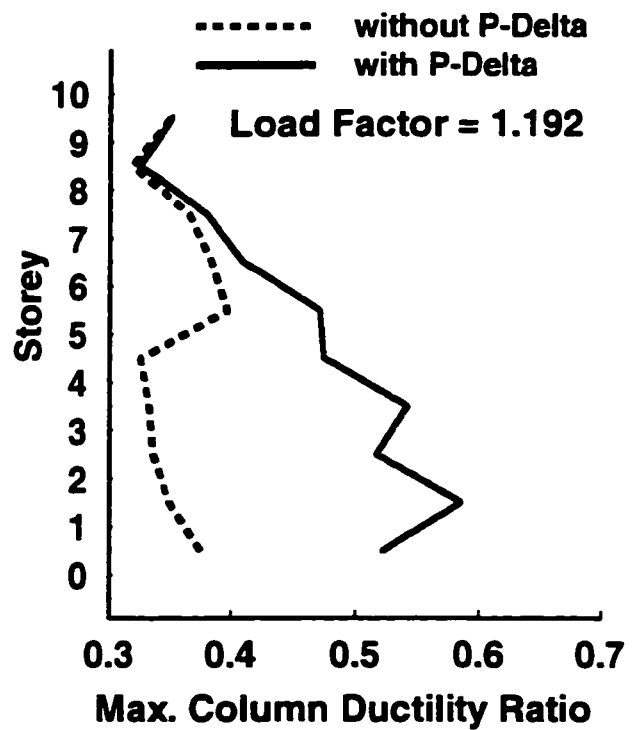
(a)



(b)



(c)



(d)

Fig. 6.12 The Effect of P- Δ in Push-Over Analysis for 10-Storey MRF building in Ottawa.

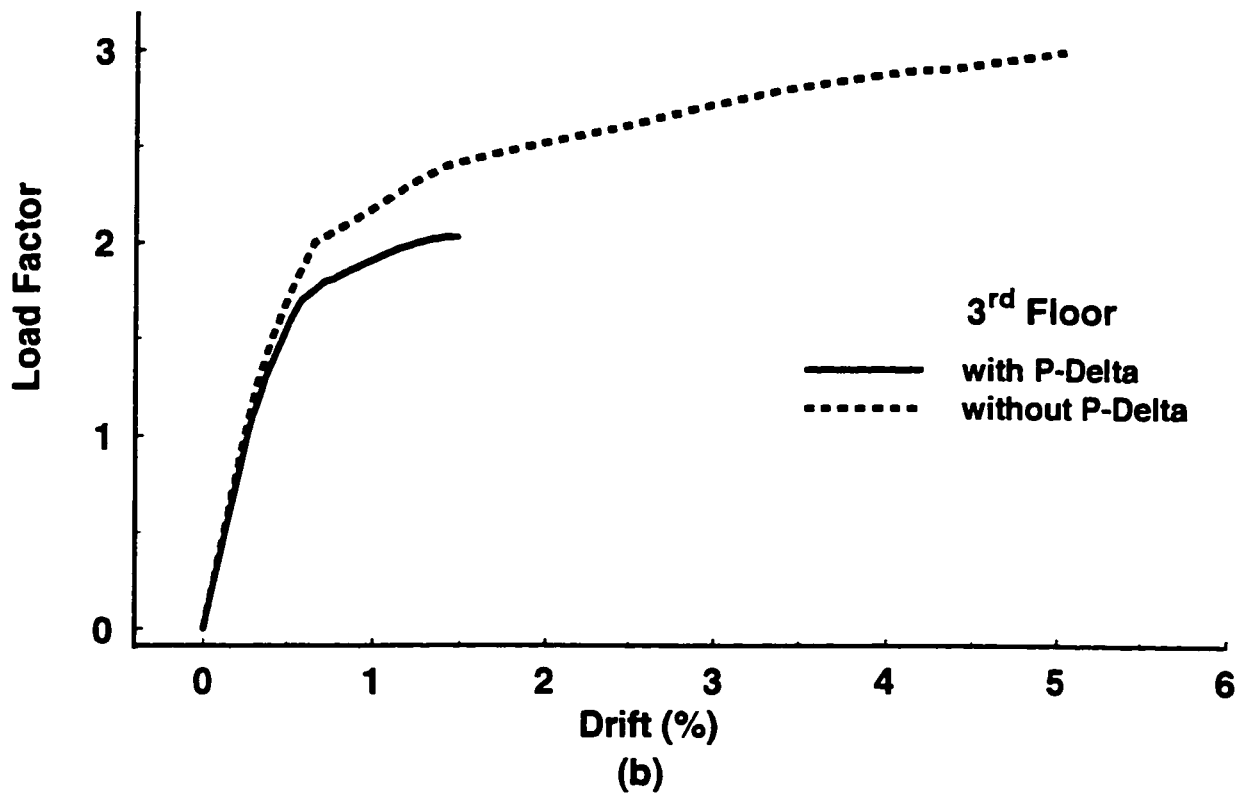
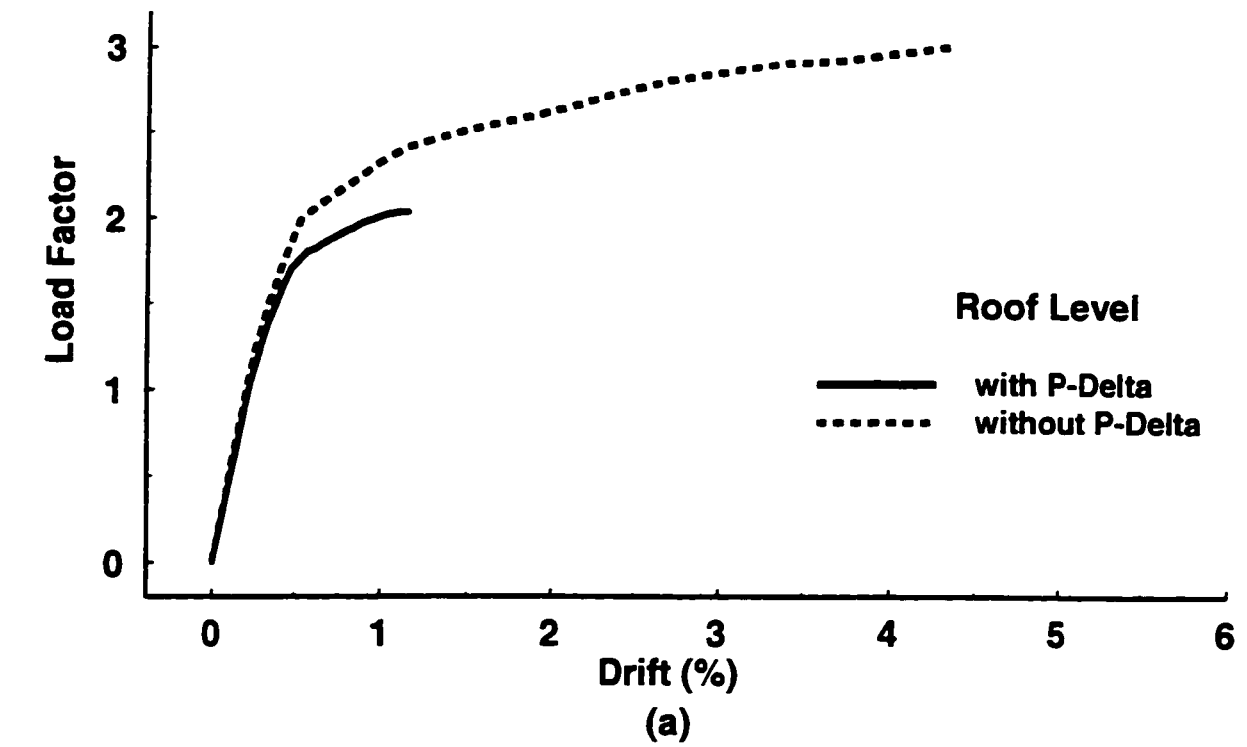


Fig. 6.13 Force-Lateral Drift Relationship for 5-Storey MRF building in Ottawa.

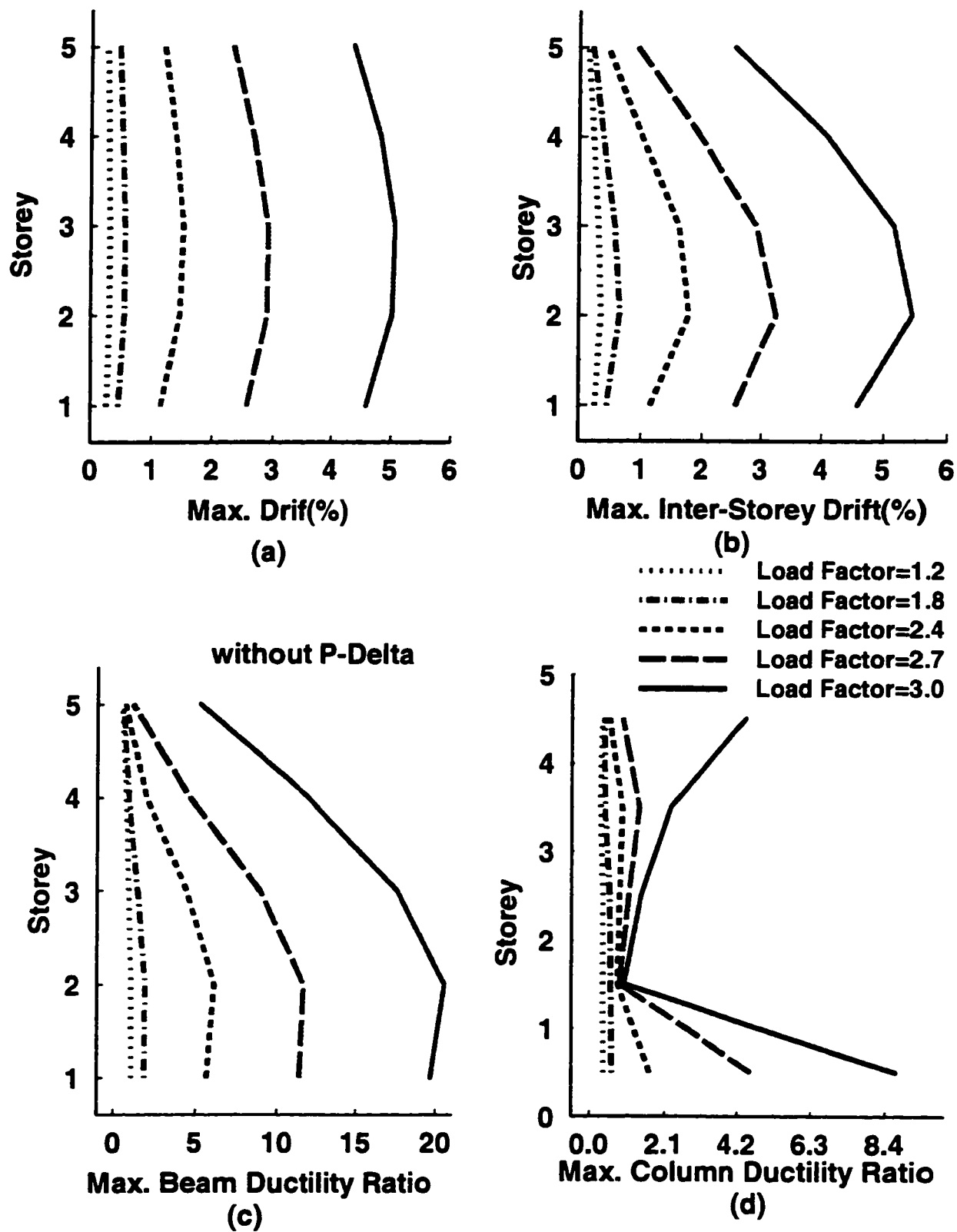
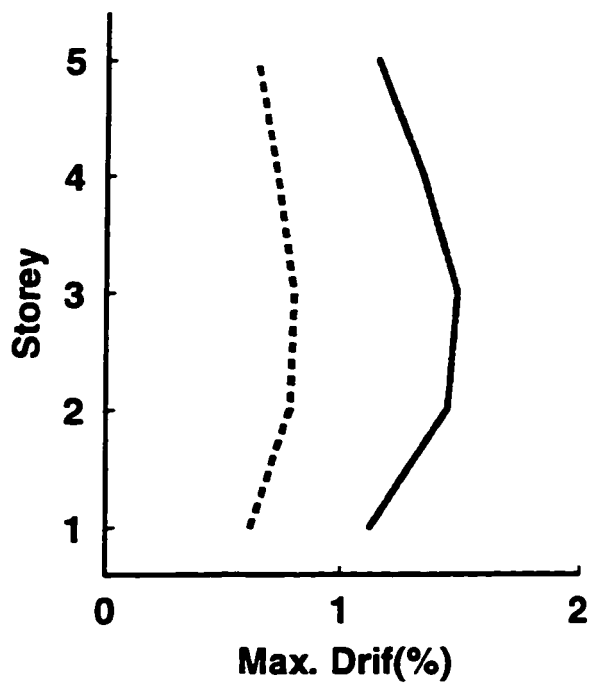
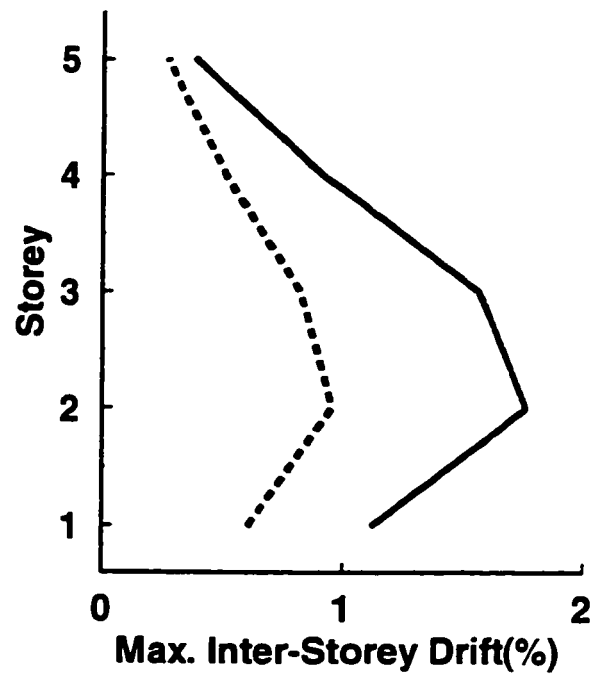


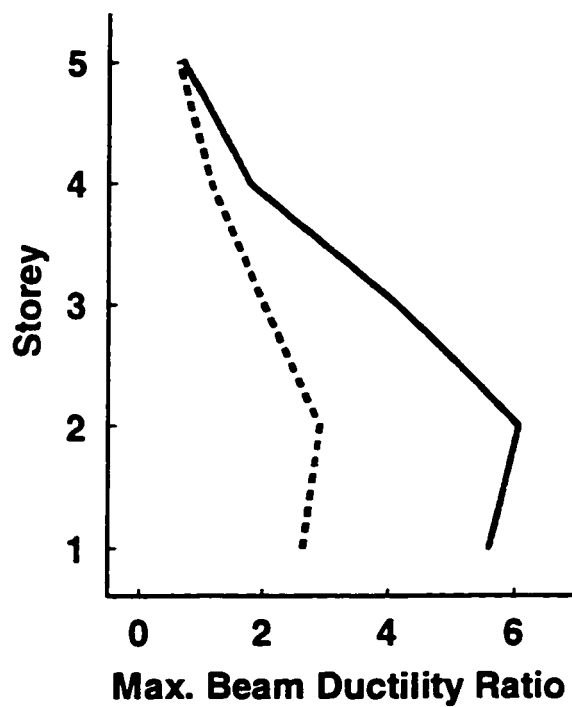
Fig. 6.14 Drift and Ductility Response at Selected Stages of Static Loading for 5-Storey MRF building in Ottawa.



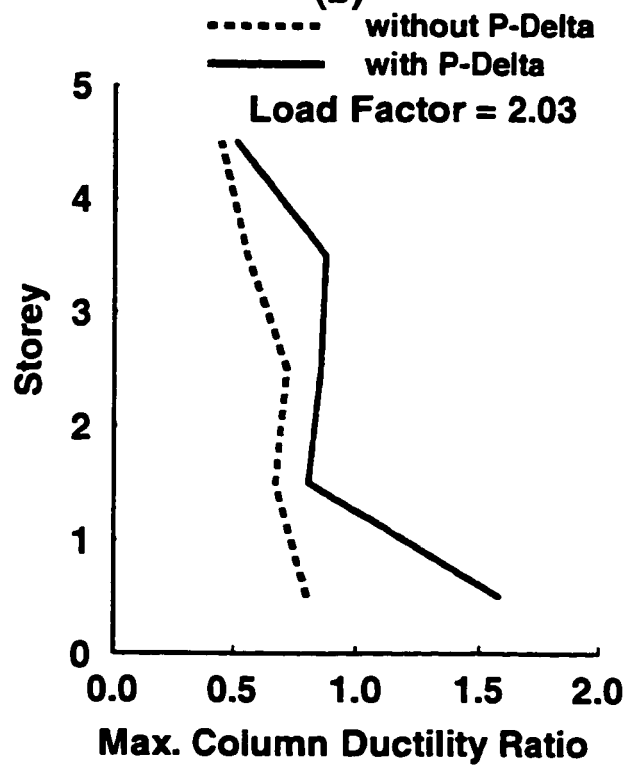
(a)



(b)



(c)



(d)

Fig. 6.15 The Effect of P- Δ in Push-Over Analysis for 5-Storey MRF building in Ottawa.

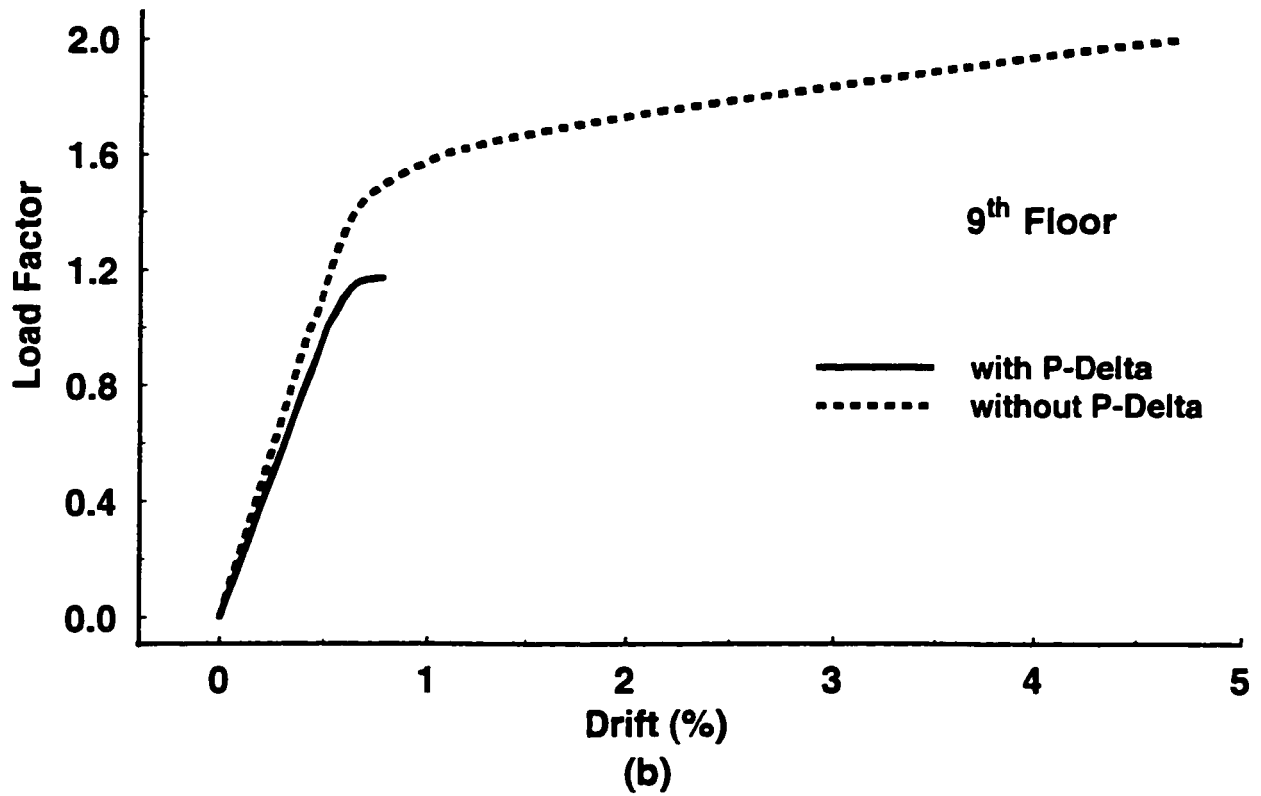
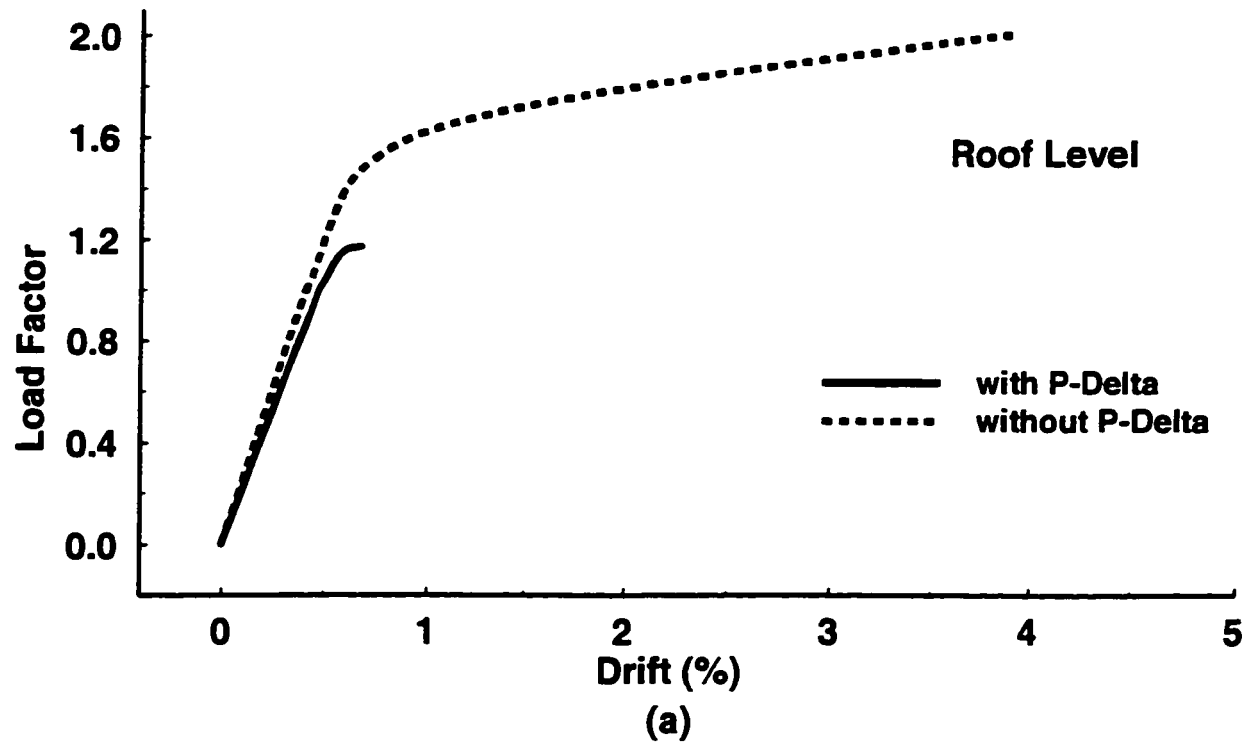
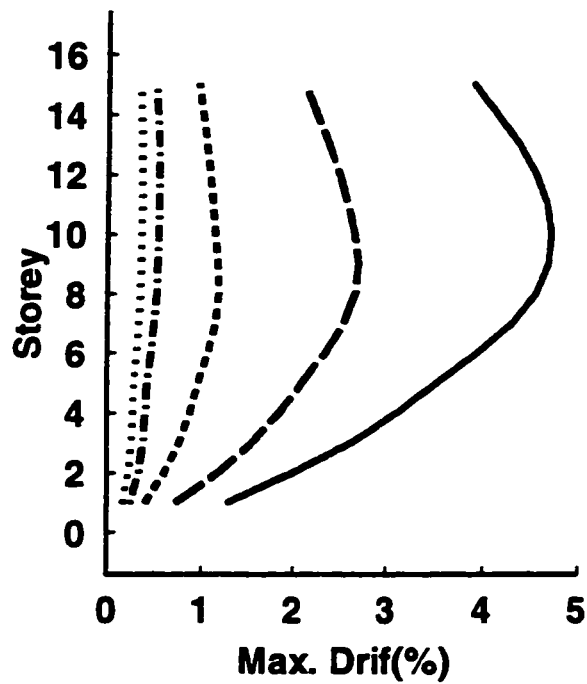
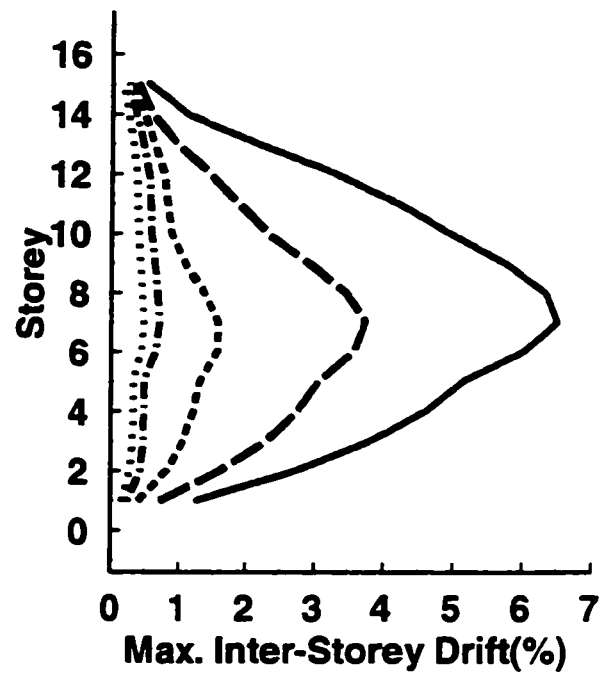


Fig. 6.16 Force-Lateral Drift Relationship for 15-Storey MRF building in Ottawa.

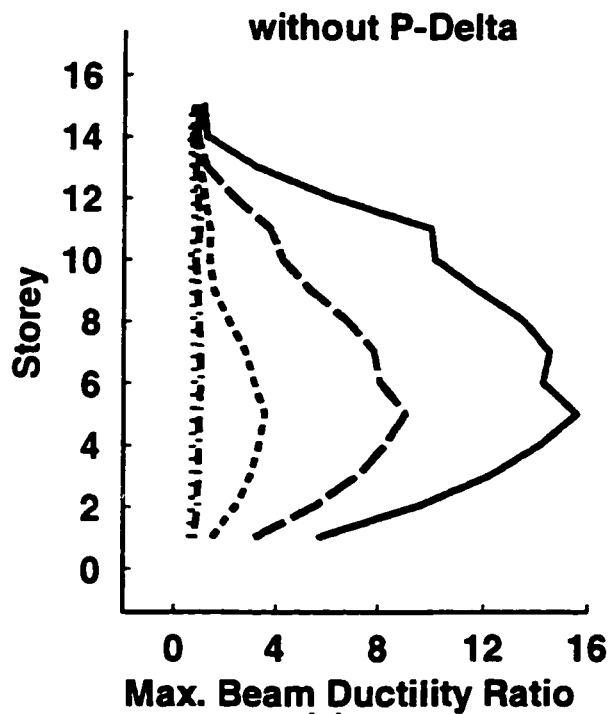


(a)

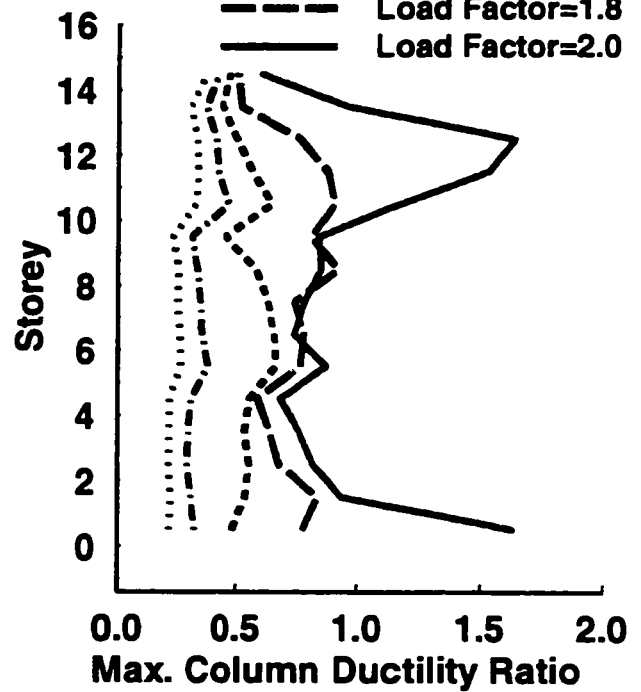


(b)

- Load Factor=0.8
- .-.-.- Load Factor=1.2
- Load Factor=1.6
- Load Factor=1.8
- Load Factor=2.0

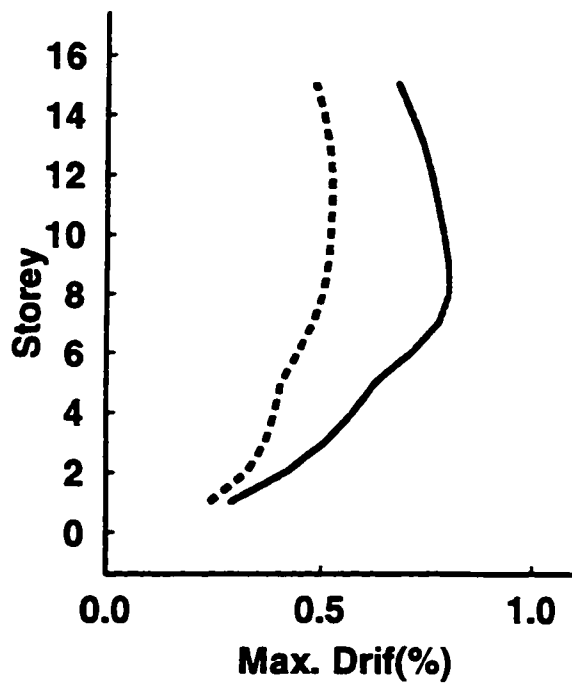


(c)

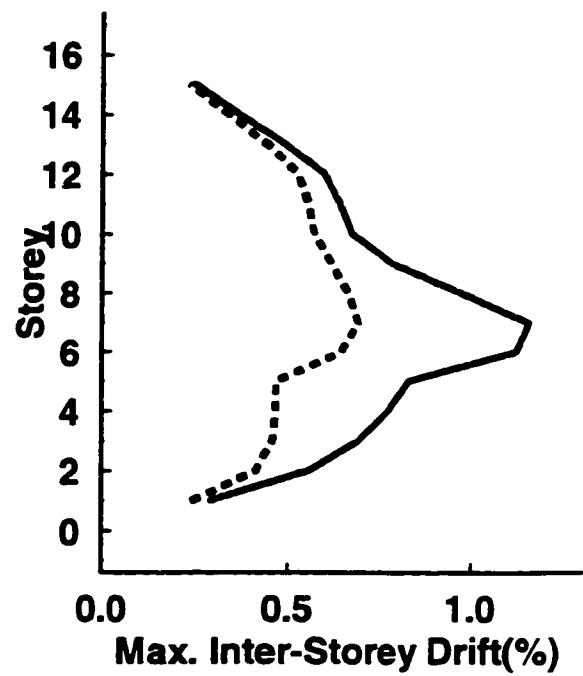


(d)

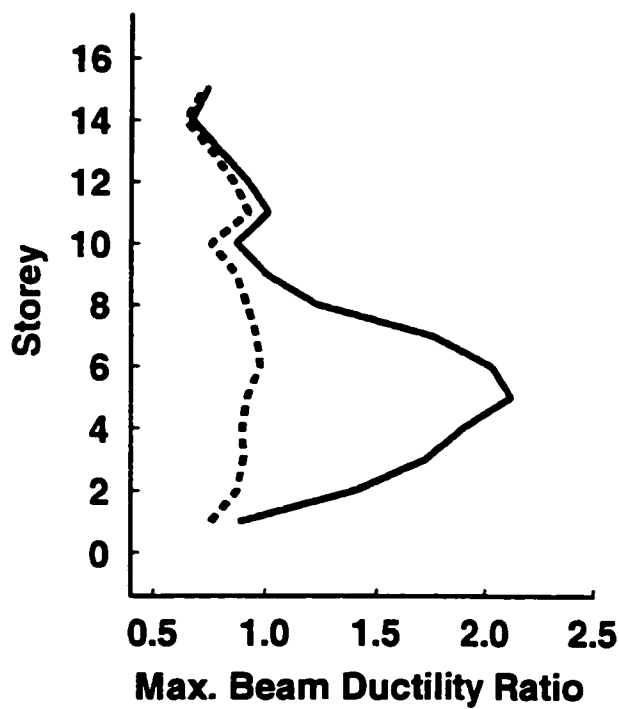
Fig. 6.17 Drift and Ductility Response at Selected Stages of Static Loading for 15-Storey MRF building in Ottawa.



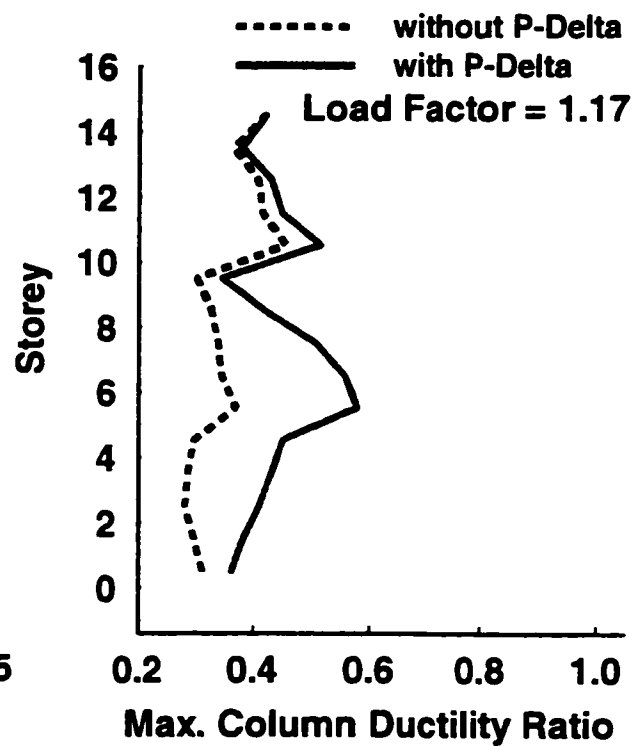
(a)



(b)



(c)



(d)

Fig. 6.18 The Effect of P- Δ in Push-Over Analysis for 15-Storey MRF building in Ottawa.

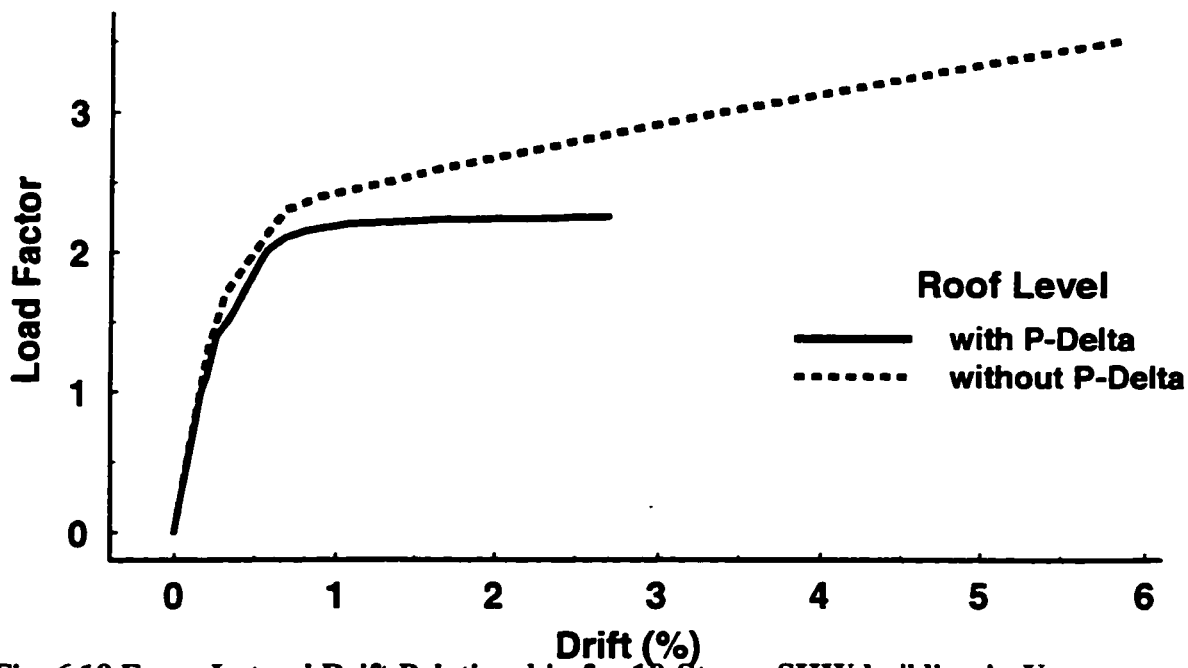


Fig. 6.19 Force-Lateral Drift Relationship for 10-Storey SHW building in Vancouver.

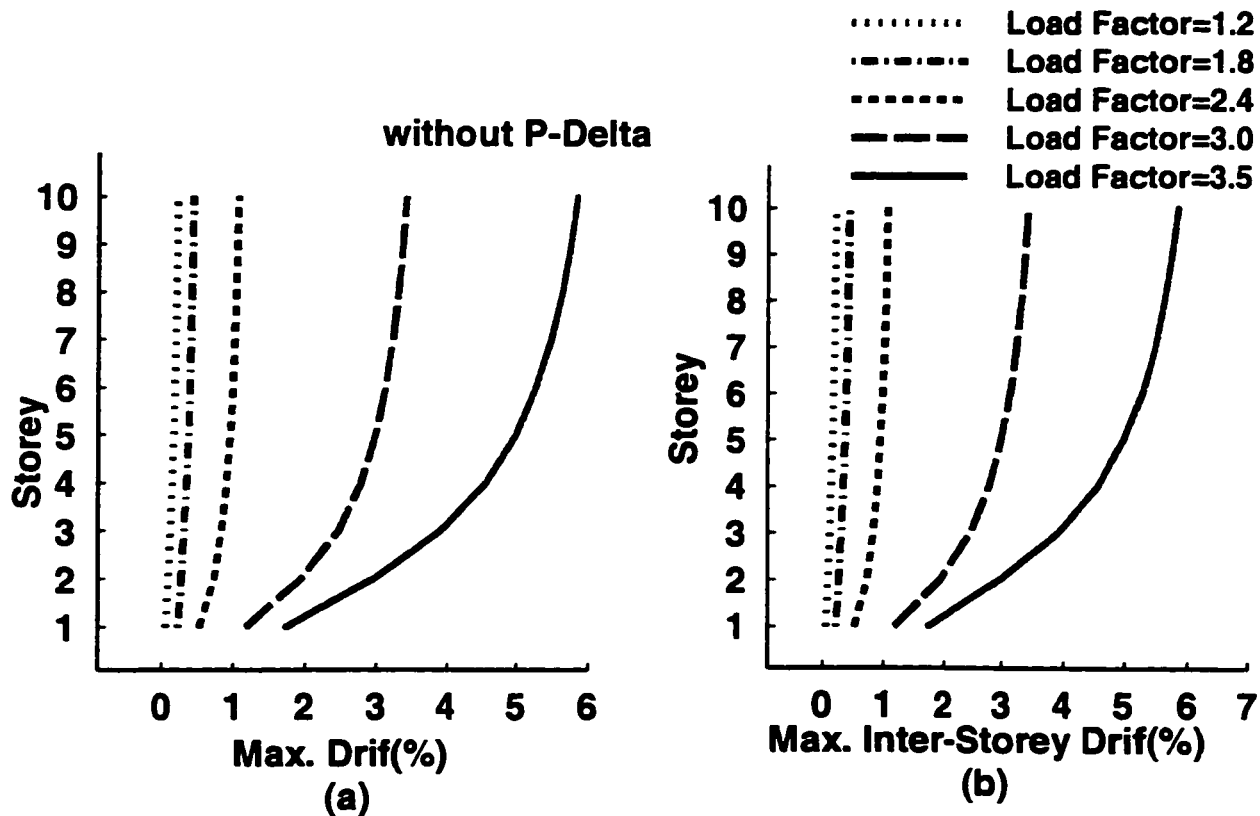
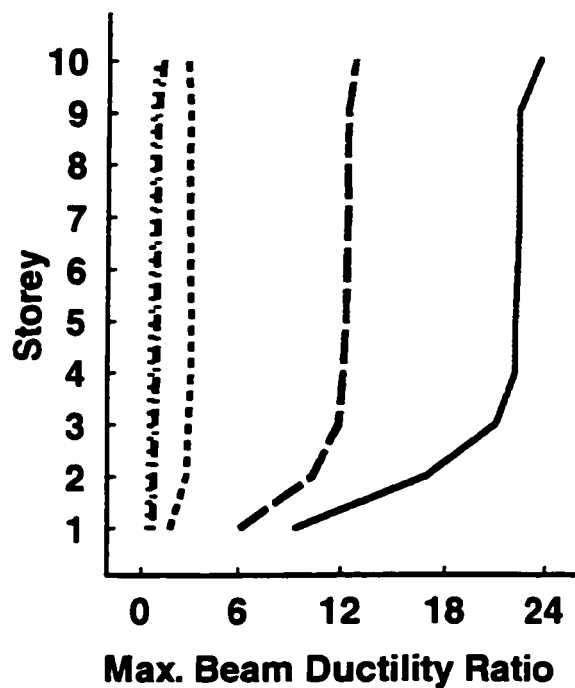
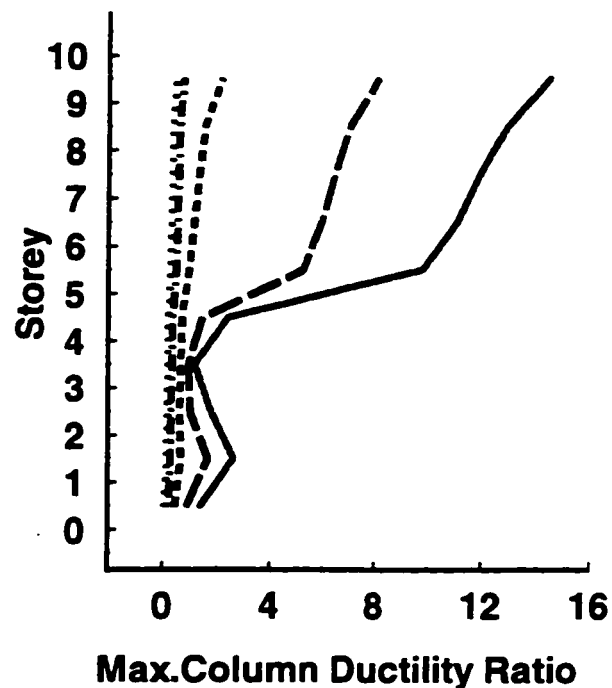


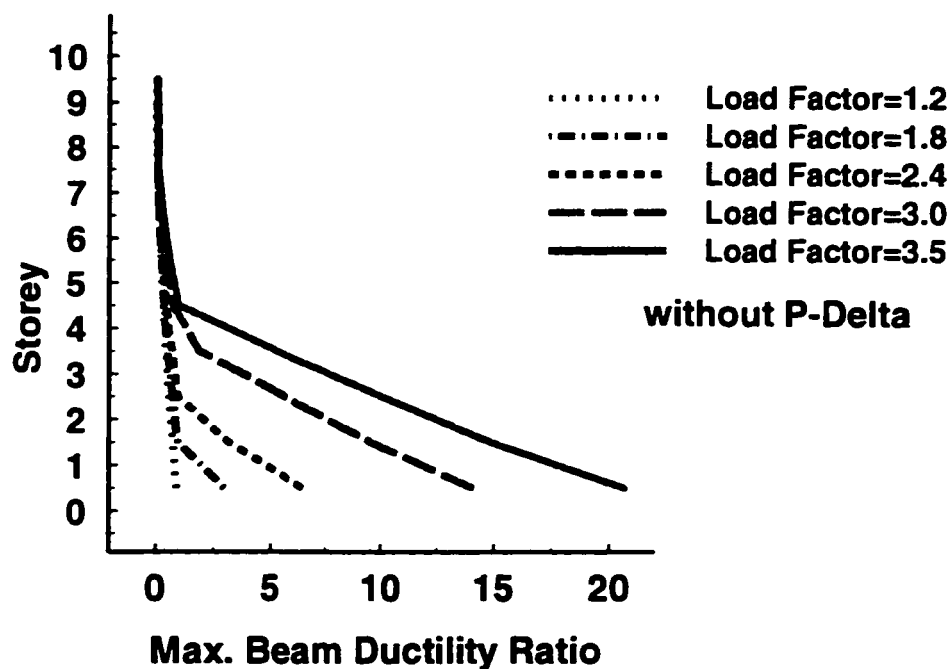
Fig. 6.20 Drift Response at Selected Stages of Static Loading for 10-Storey SHW building in Vancouver.



(a)

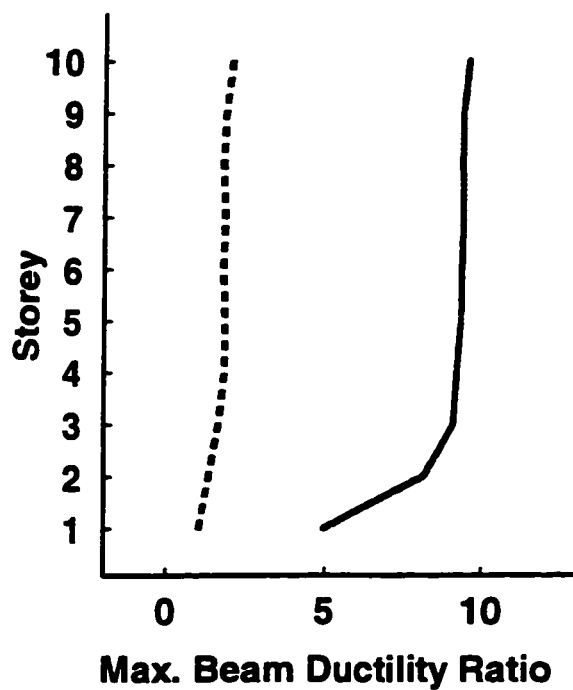


(b)

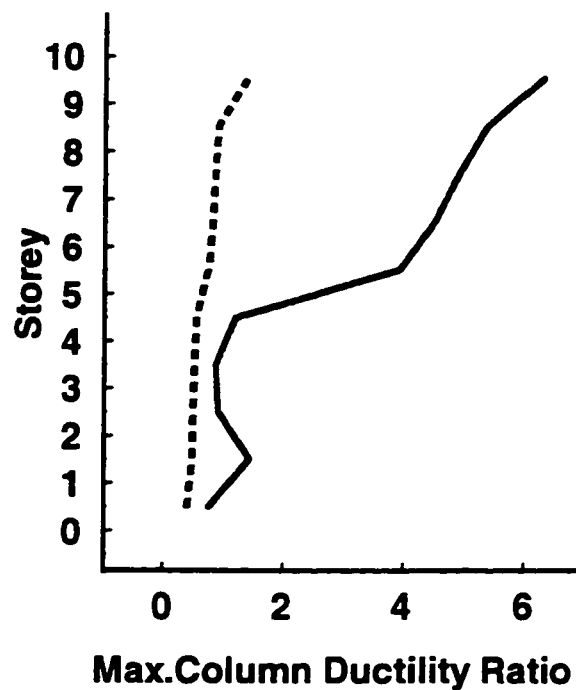


(c)

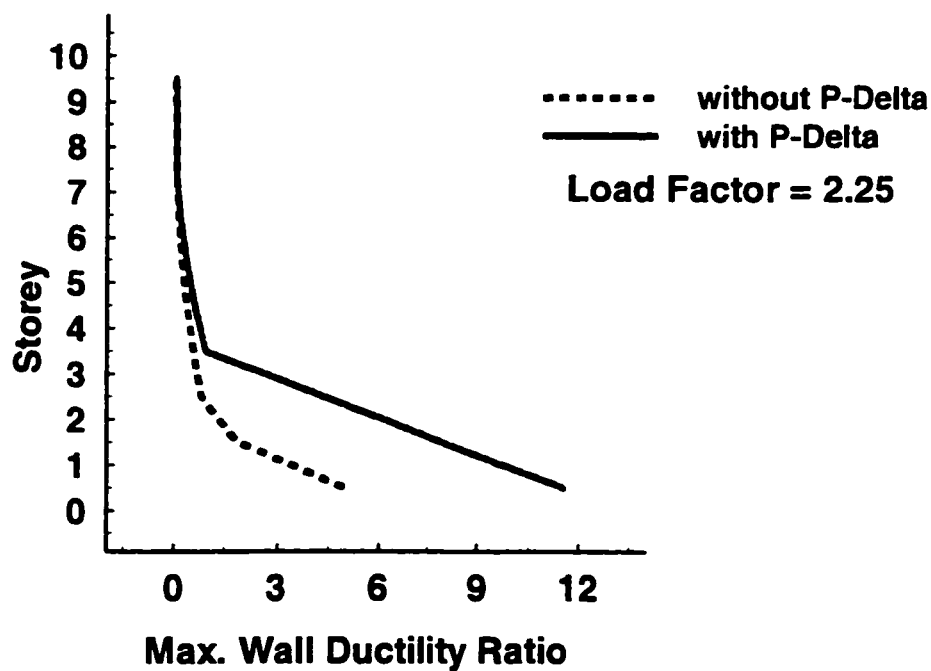
Fig. 6.21 Ductility Response at Selected Stages of Static Loading for 10-Storey SHW building in Vancouver.



(a)



(b)



(c)

Fig. 6.22 The Effect of P- Δ in Push-Over Analysis for 10-Storey SHW building in Vancouver.

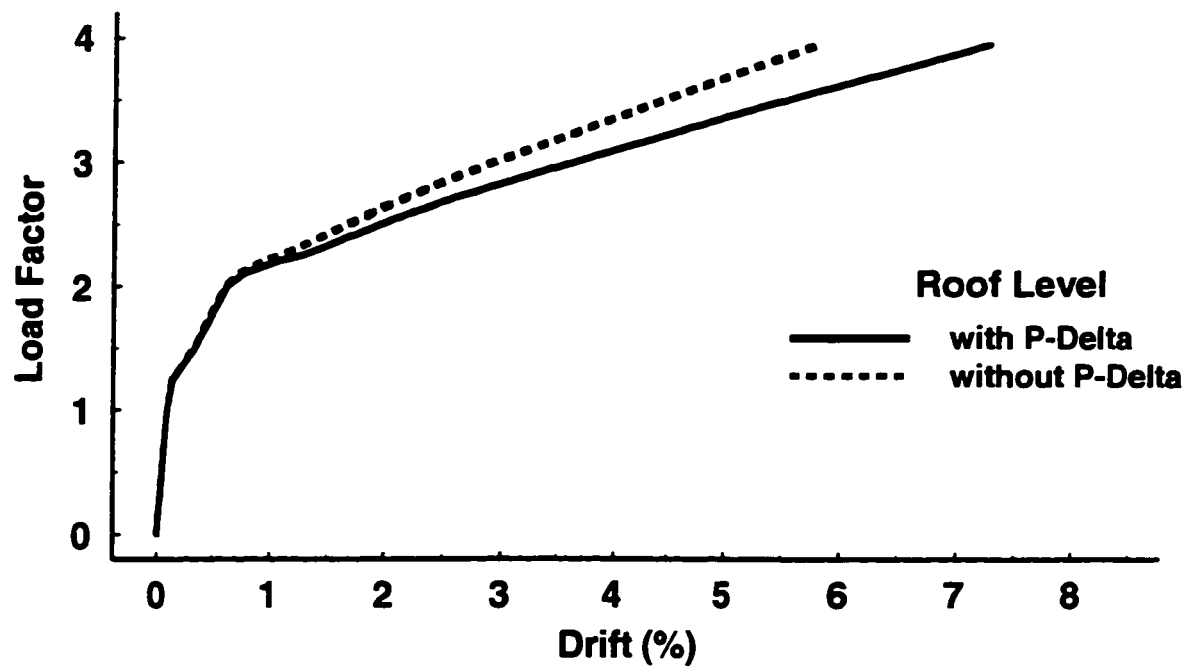
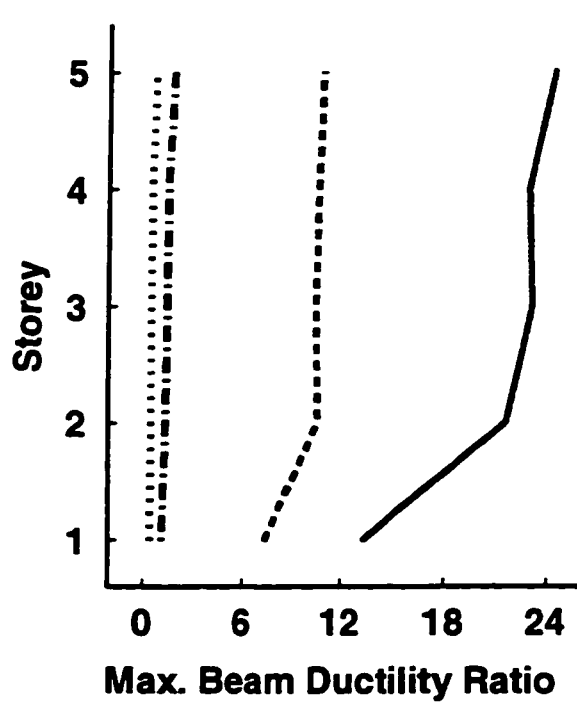
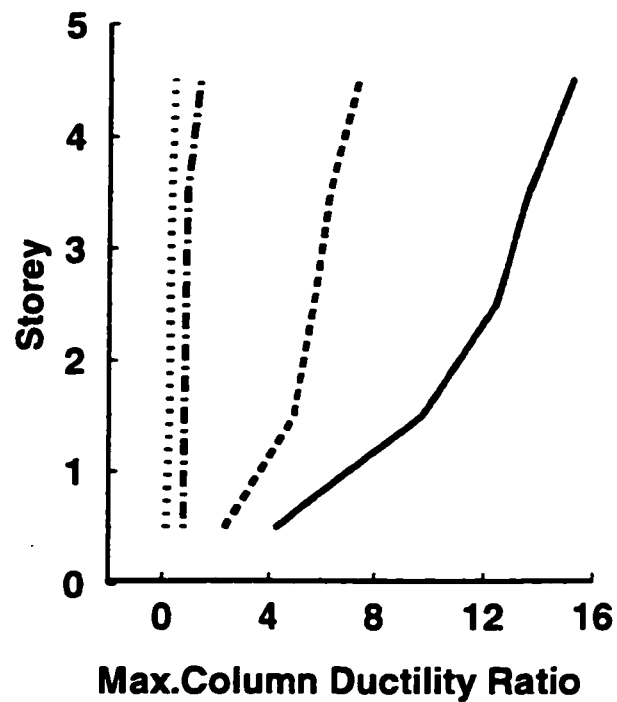


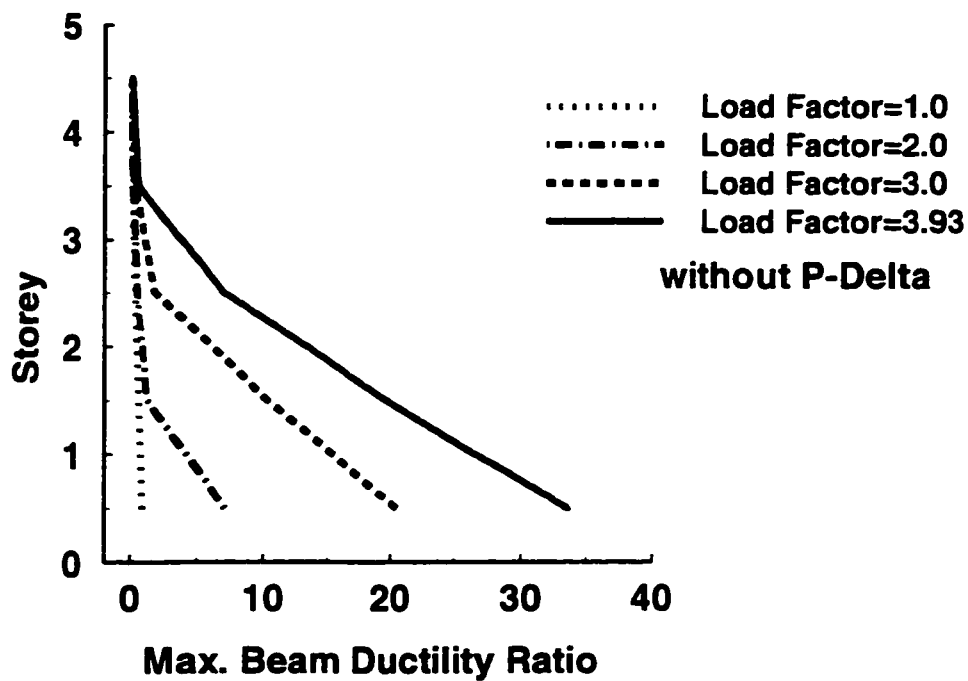
Fig. 6.23 Force-Lateral Drift Relationship for 5-Storey SHW building in Vancouver.



(a)



(b)



(c)

Fig. 6.24 Ductility Response at Selected Stages of Static Loading for 5-Storey SHW building in Vancouver.

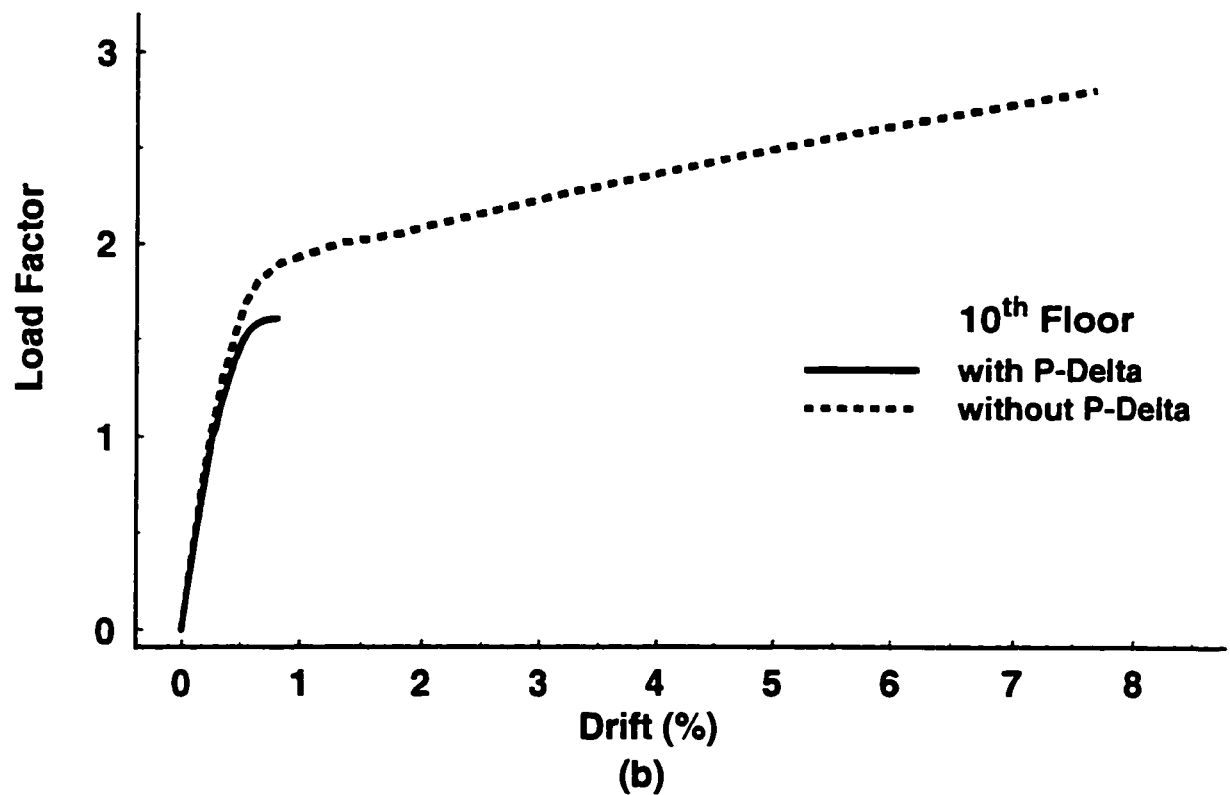
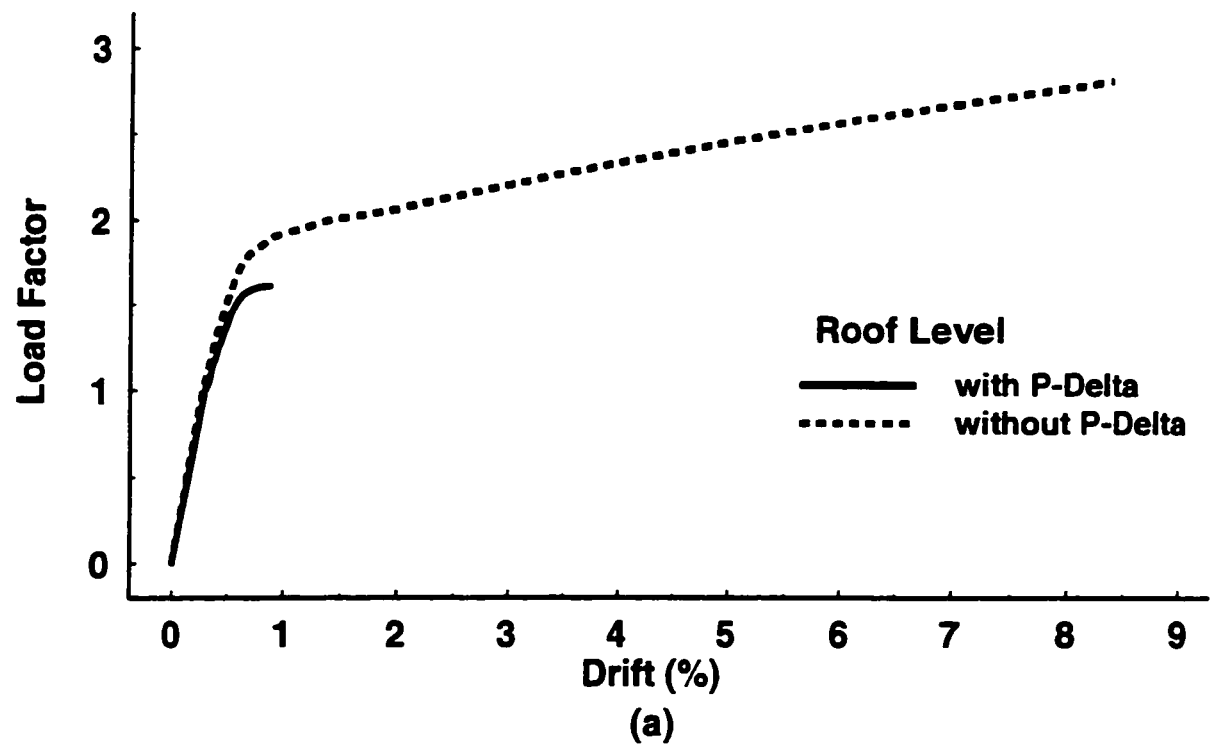
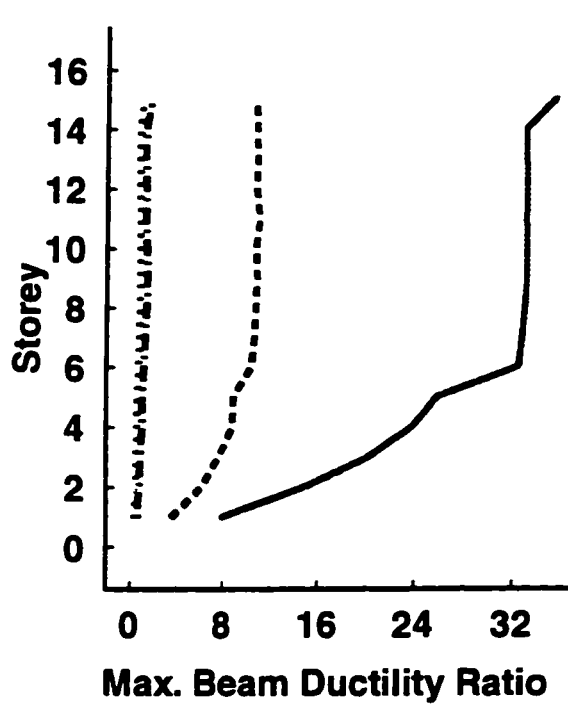
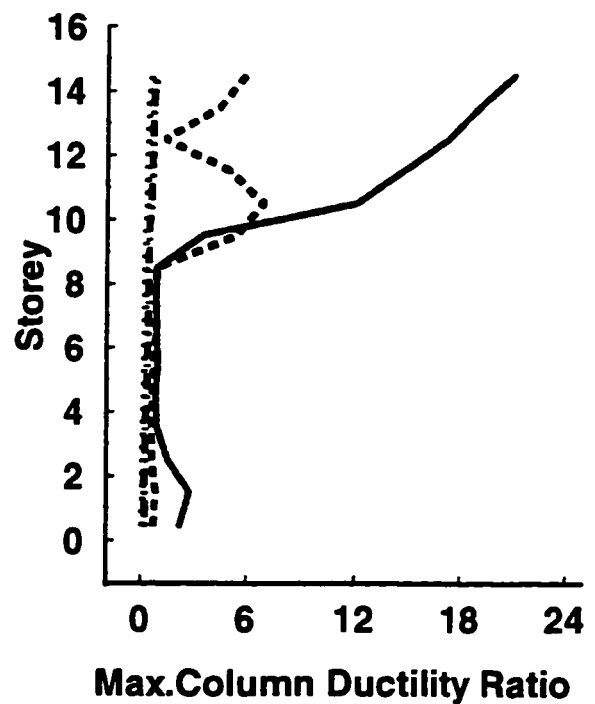


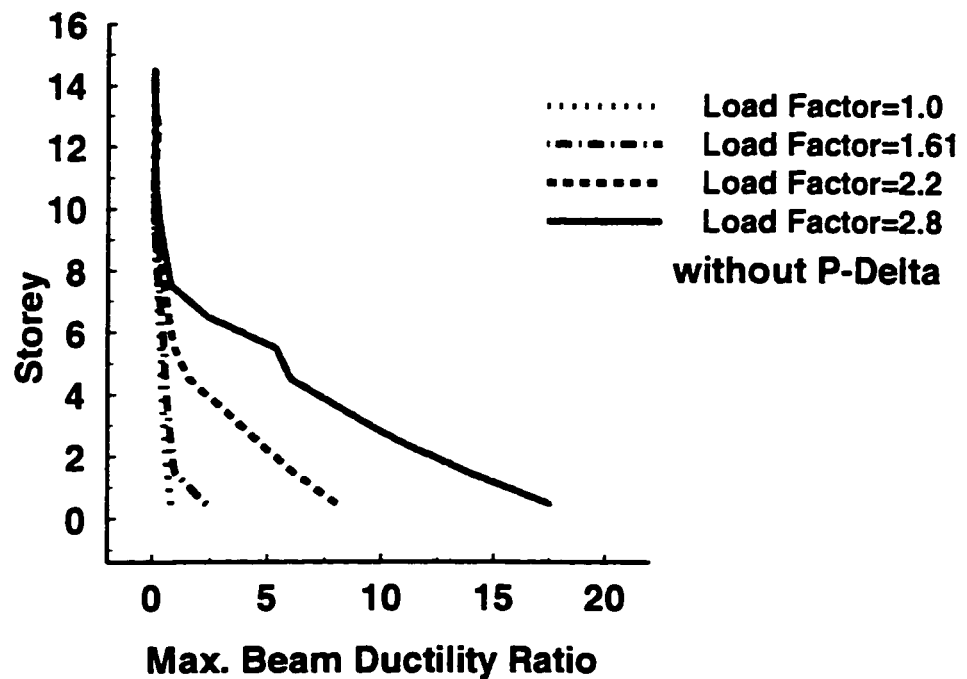
Fig. 6.25 Force-Lateral Drift Relationship for 15-Storey SHW building in Vancouver.



(a)



(b)



(c)

Fig. 6.26 Ductility Response at Selected Stages of Static Loading for 15-Storey SHW building in Vancouver.

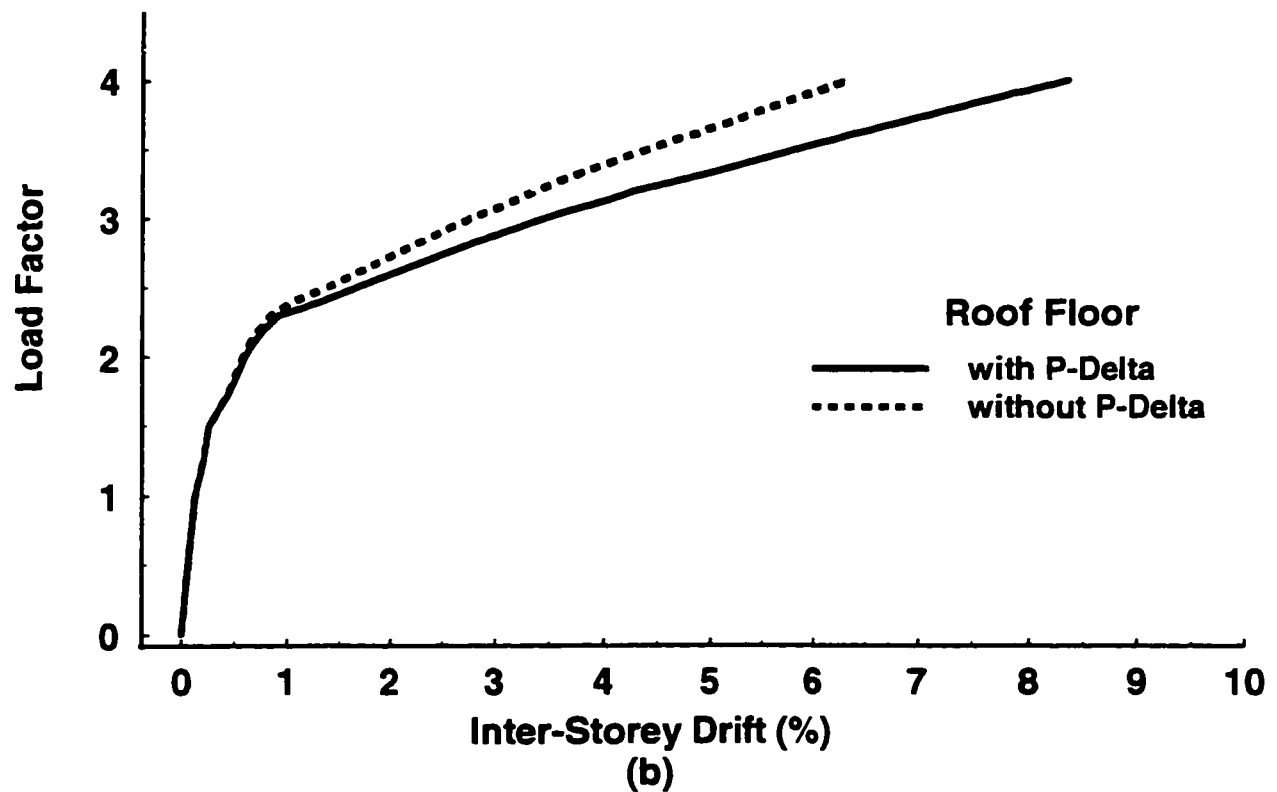
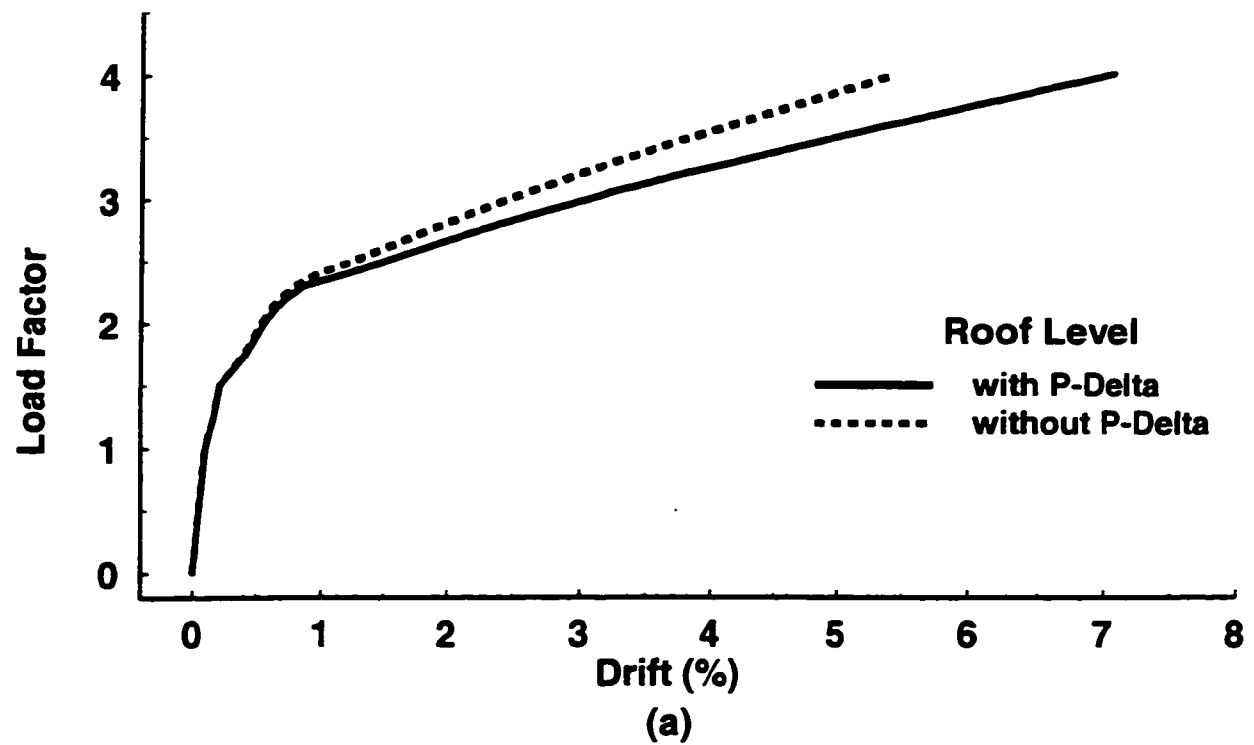


Fig. 6.27 Force-Lateral Drift Relationship for 5-Storey SHW building in Ottawa.

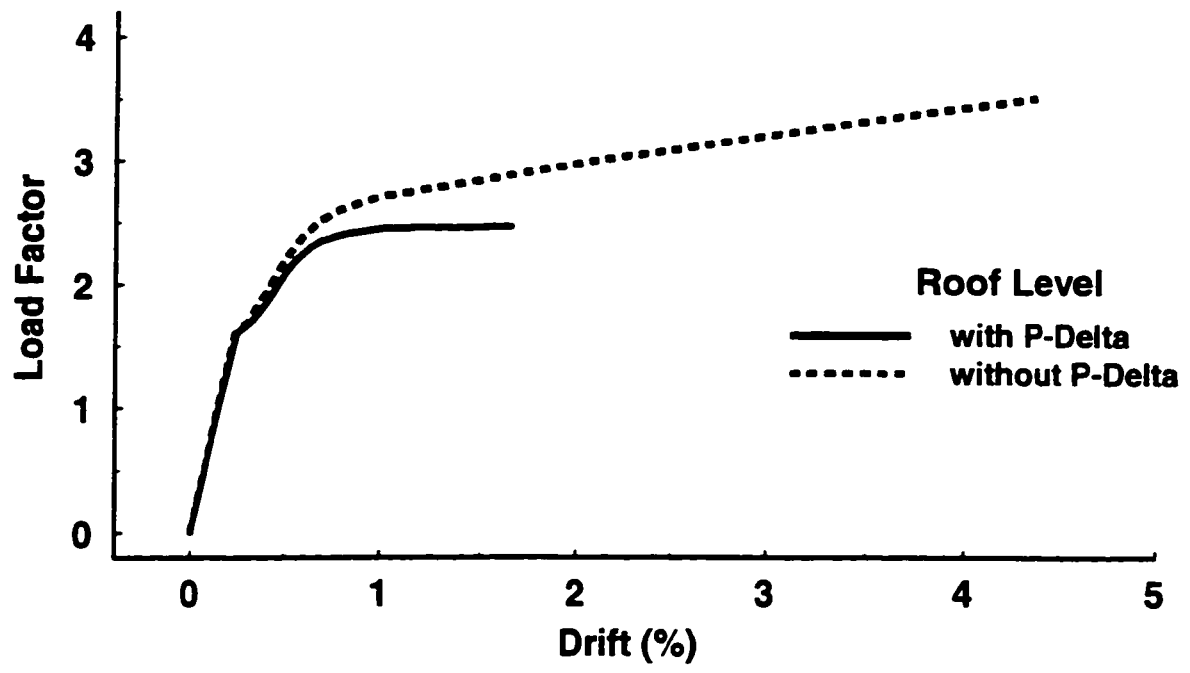


Fig. 6.28 Force-Lateral Drift Relationship for 10-Storey SHW building in Ottawa.

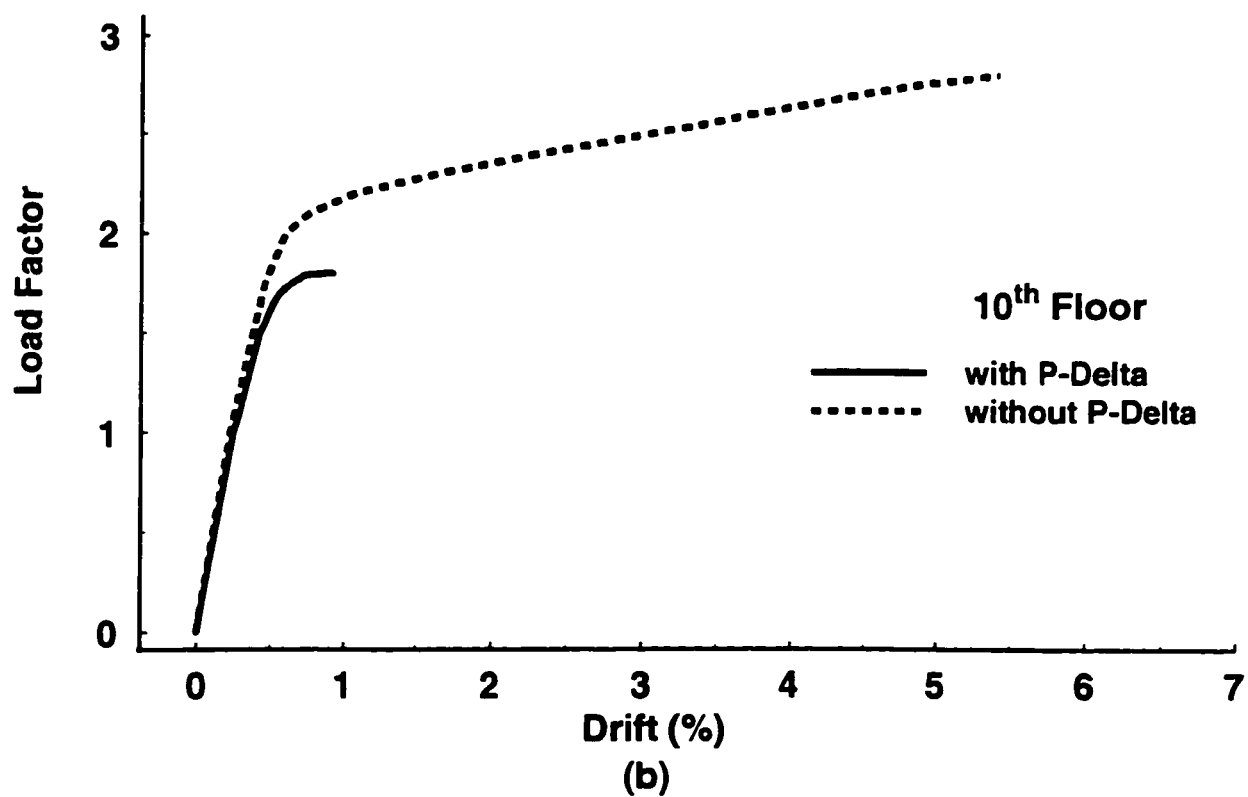
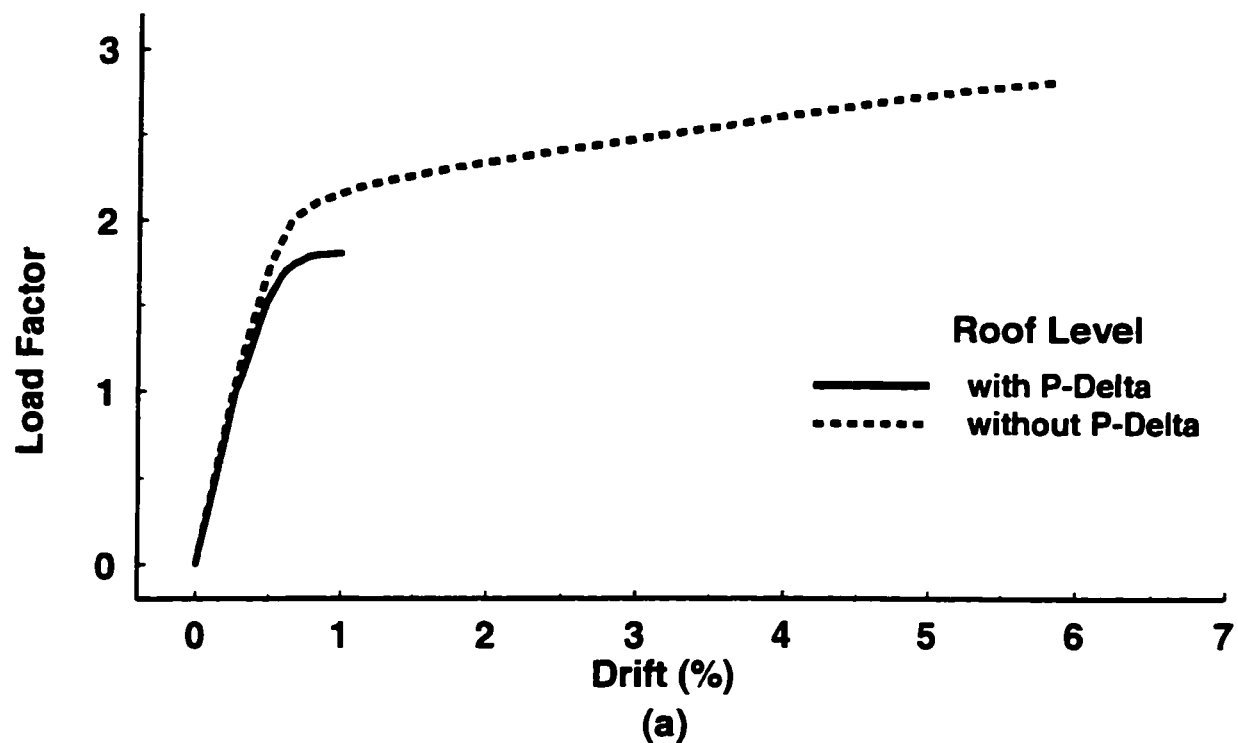


Fig. 6.29 Force-Lateral Drift Relationship for 15-Storey SHW building in Ottawa.

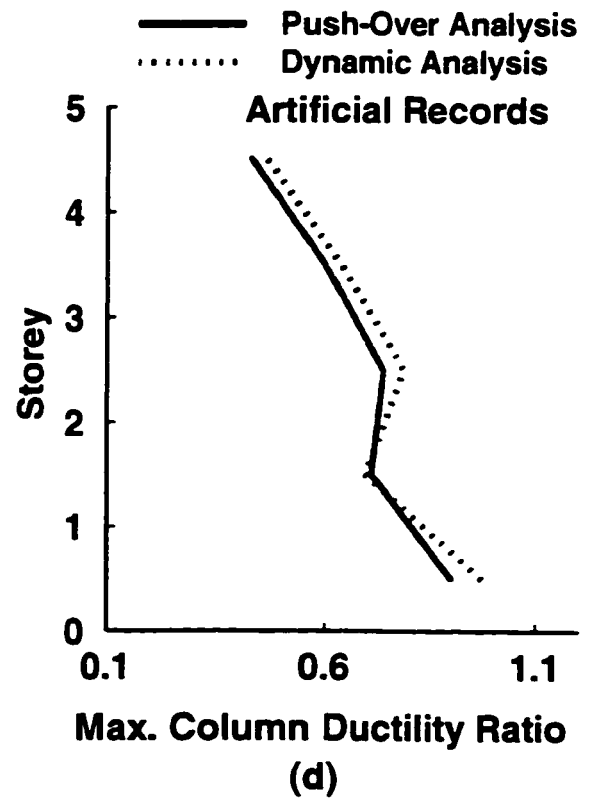
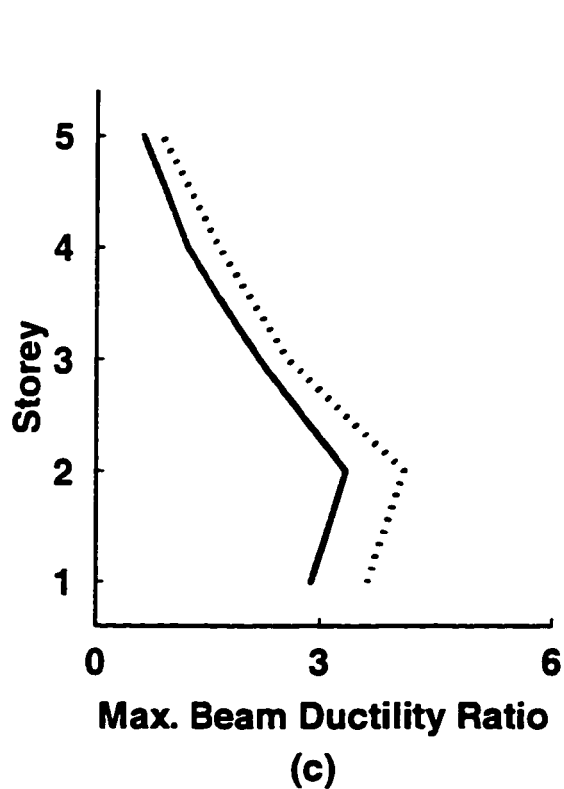
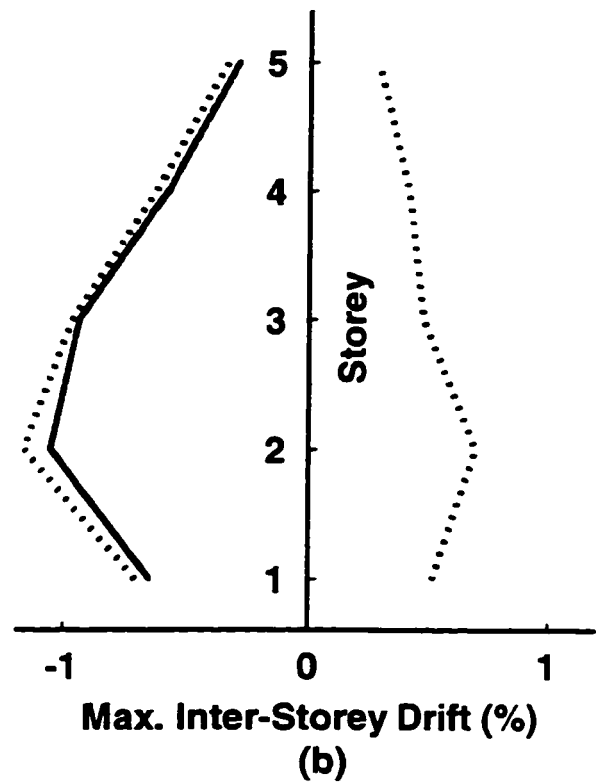
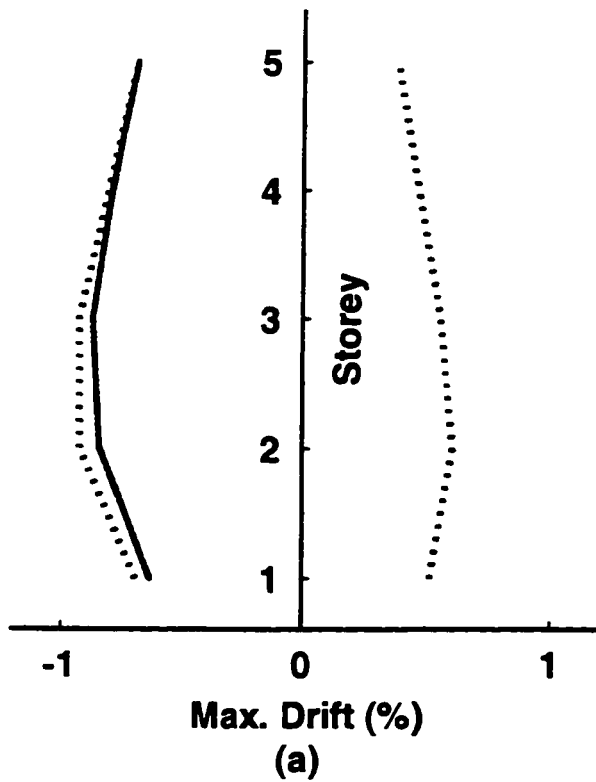


Fig. 6.30 Comparison of Static and Dynamic Drift and Ductility Response for 5-Storey MRF building in Vancouver.

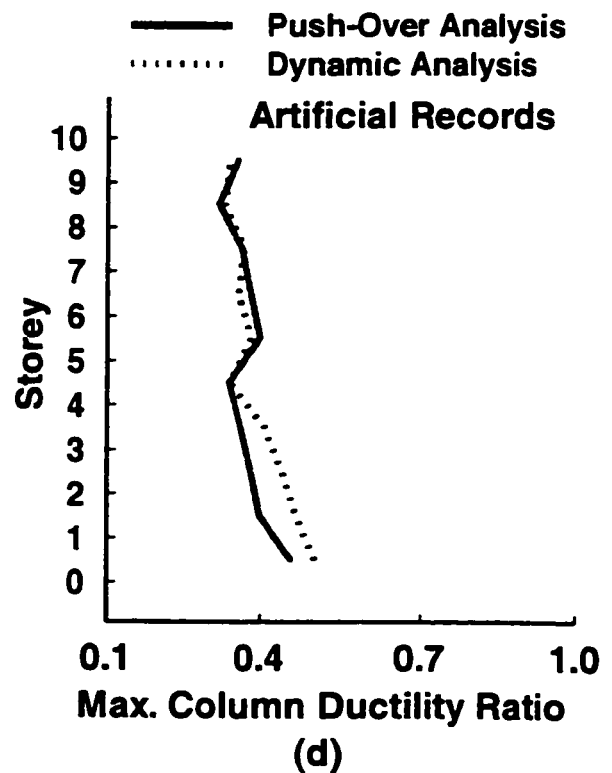
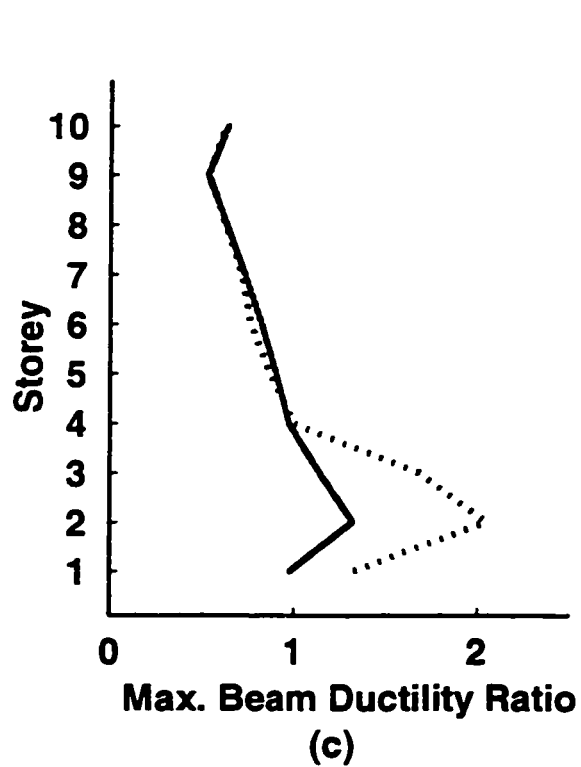
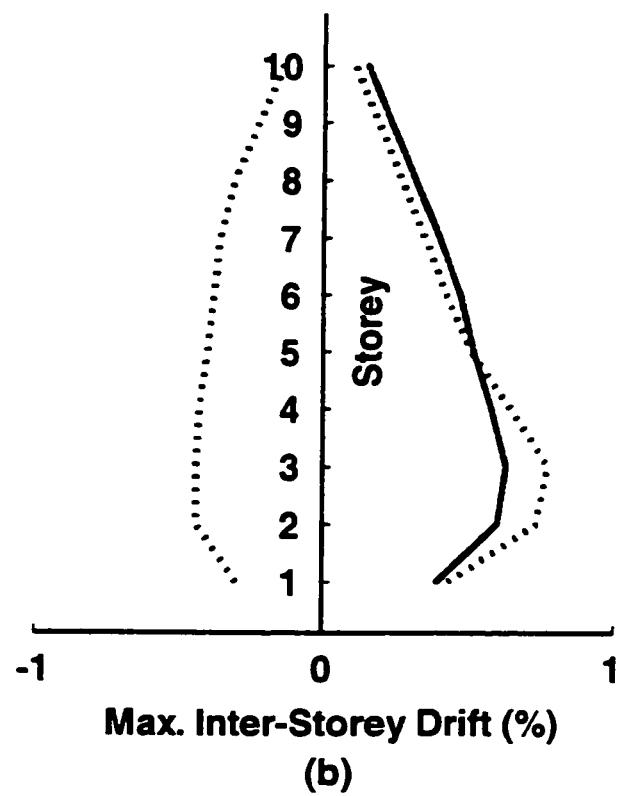
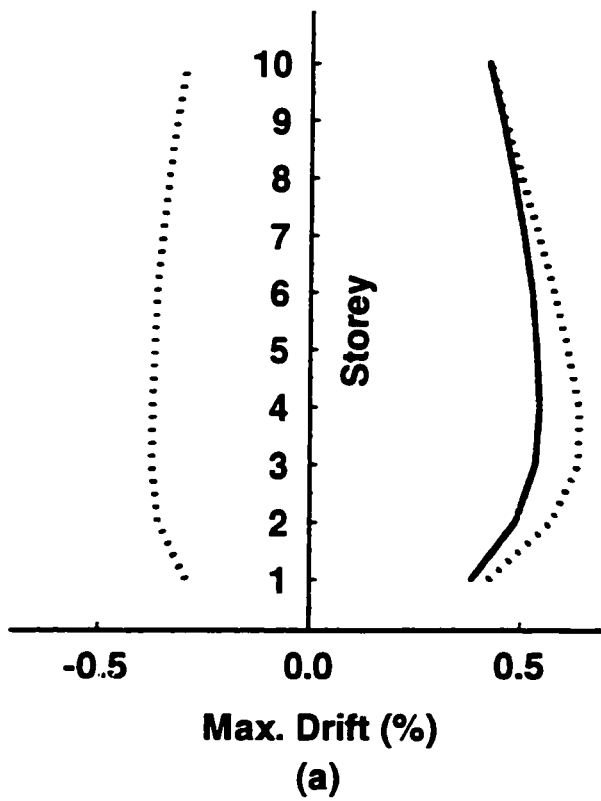


Fig. 6.31 Comparison of Static and Dynamic Drift and Ductility Response for 10-Storey MRF building in Vancouver.

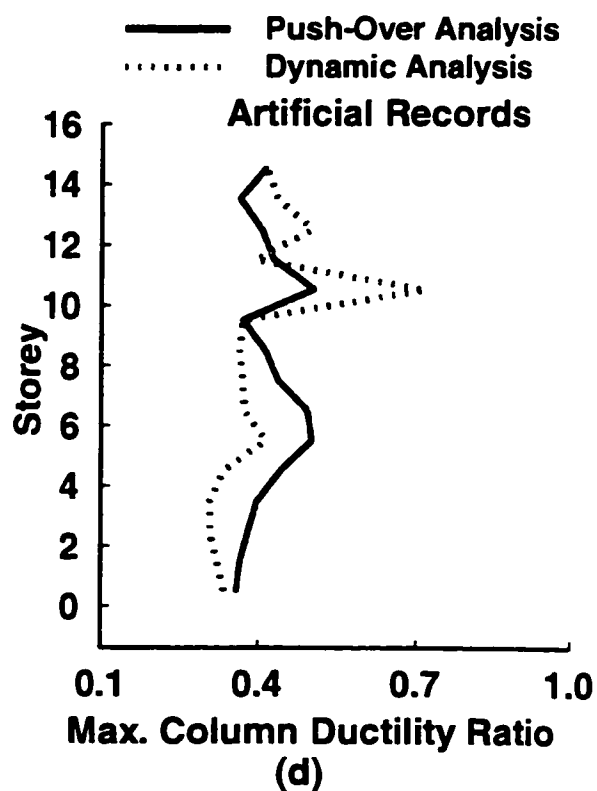
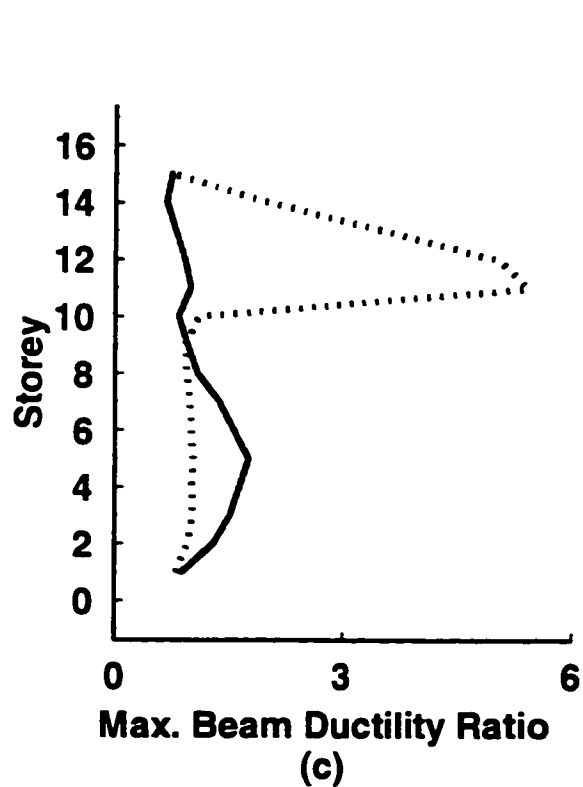
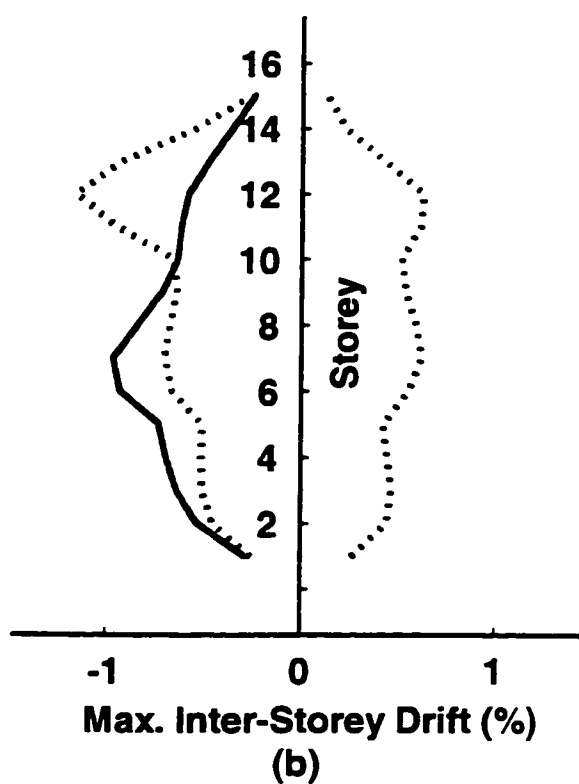
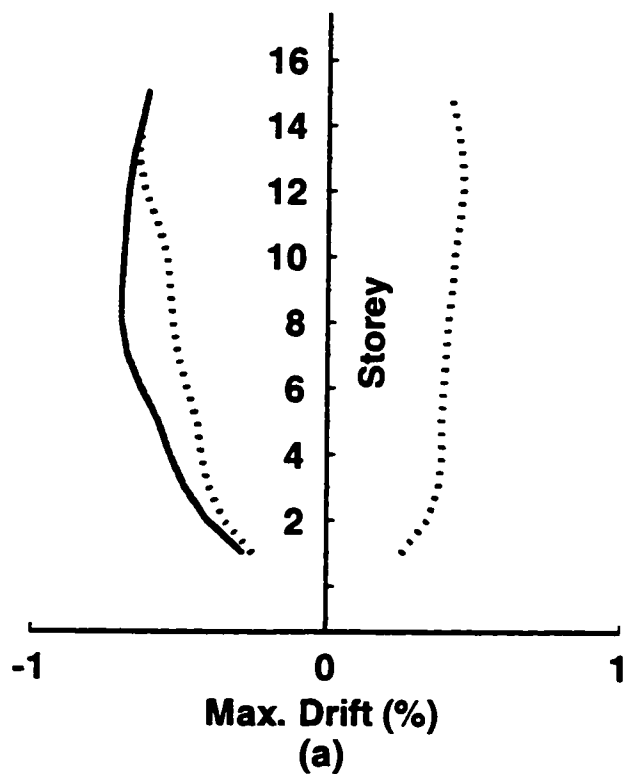
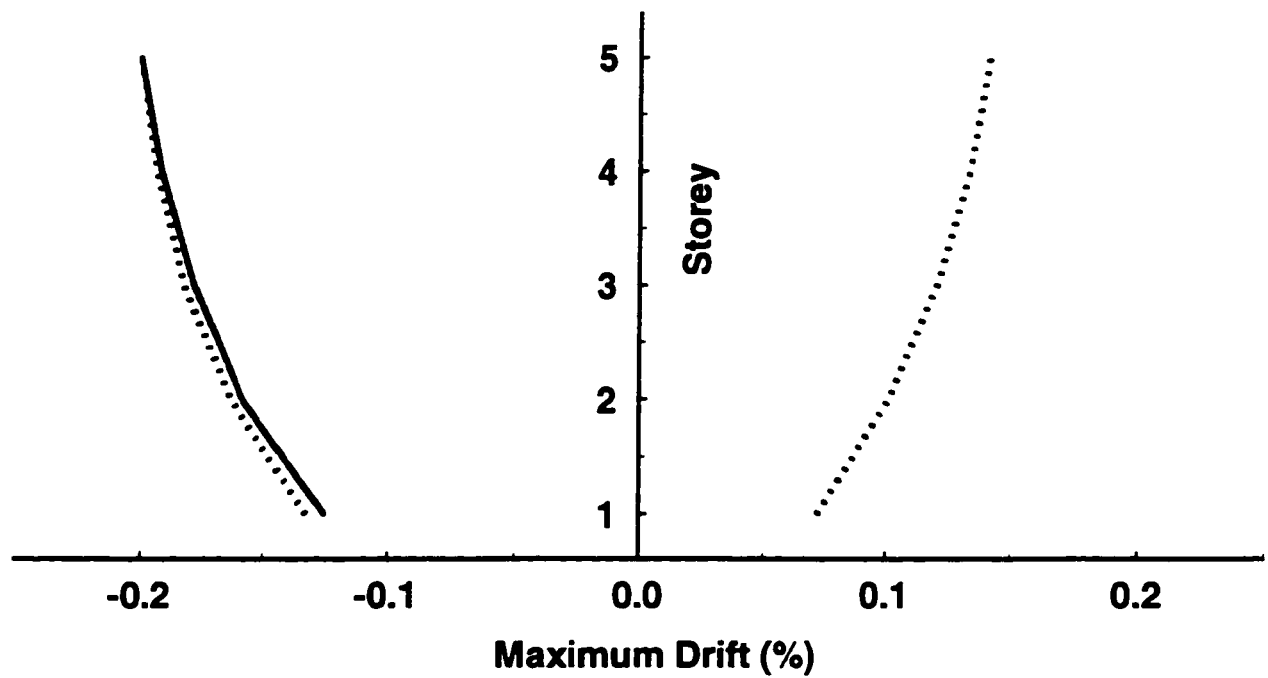
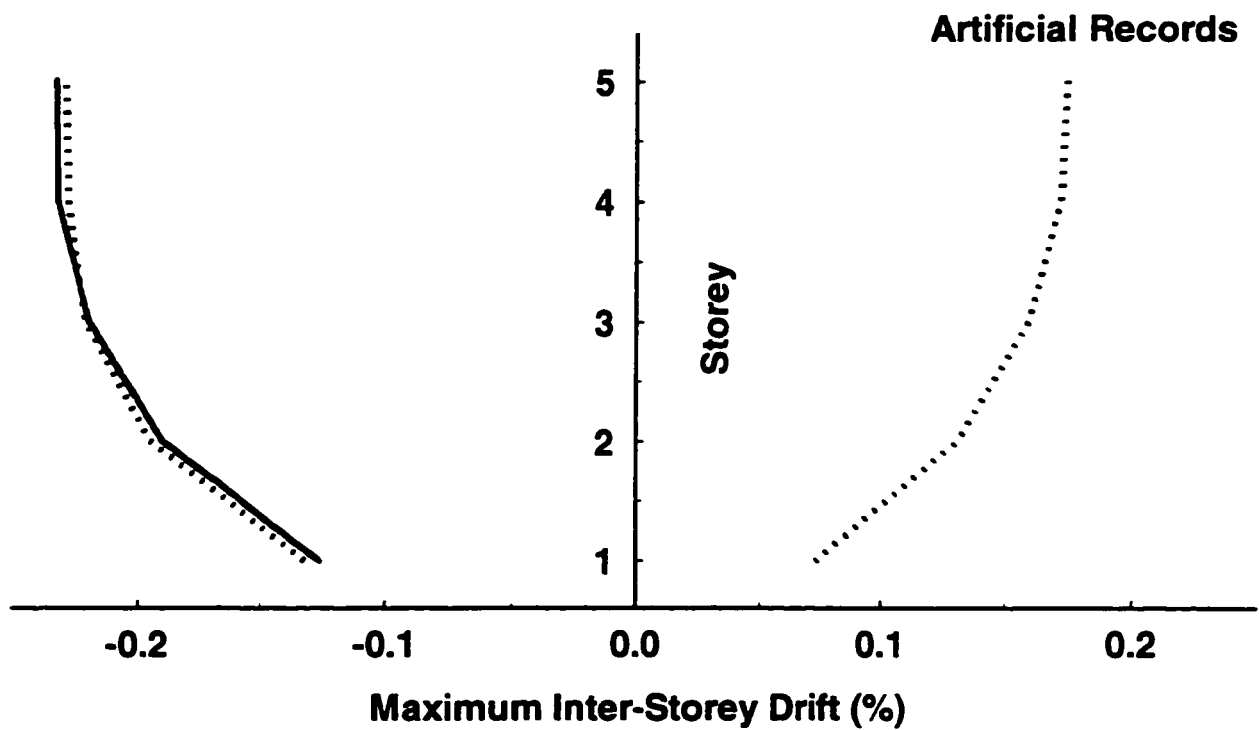


Fig. 6.32 Comparison of Static and Dynamic Drift and Ductility Response for 15-Storey MRF building in Vancouver.



(a)

— Push-Over Analysis
 Dynamic Analysis



(b)

Fig. 6.33 Comparison of Static and Dynamic Drift Response for 5-Storey SHW building in Vancouver.

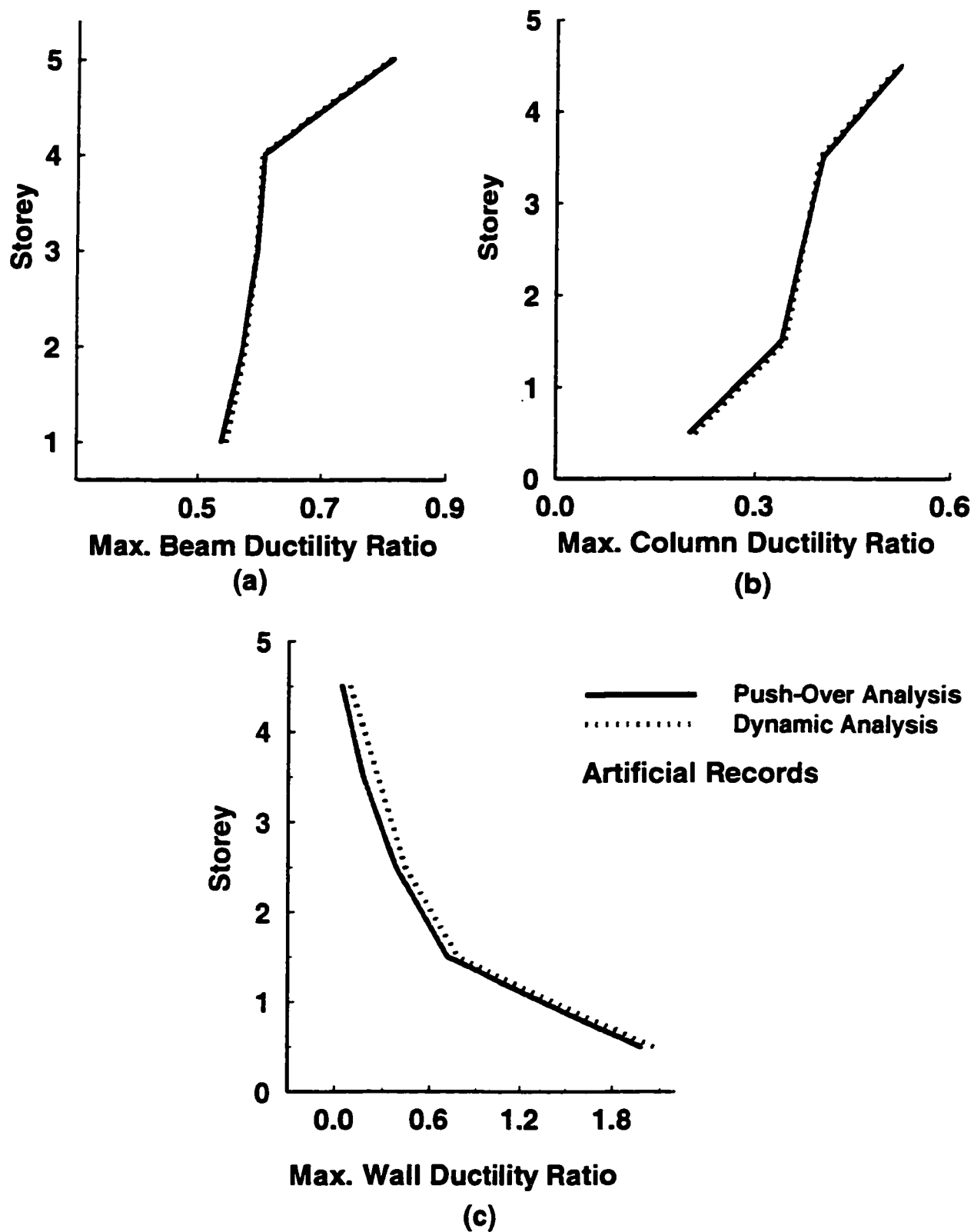
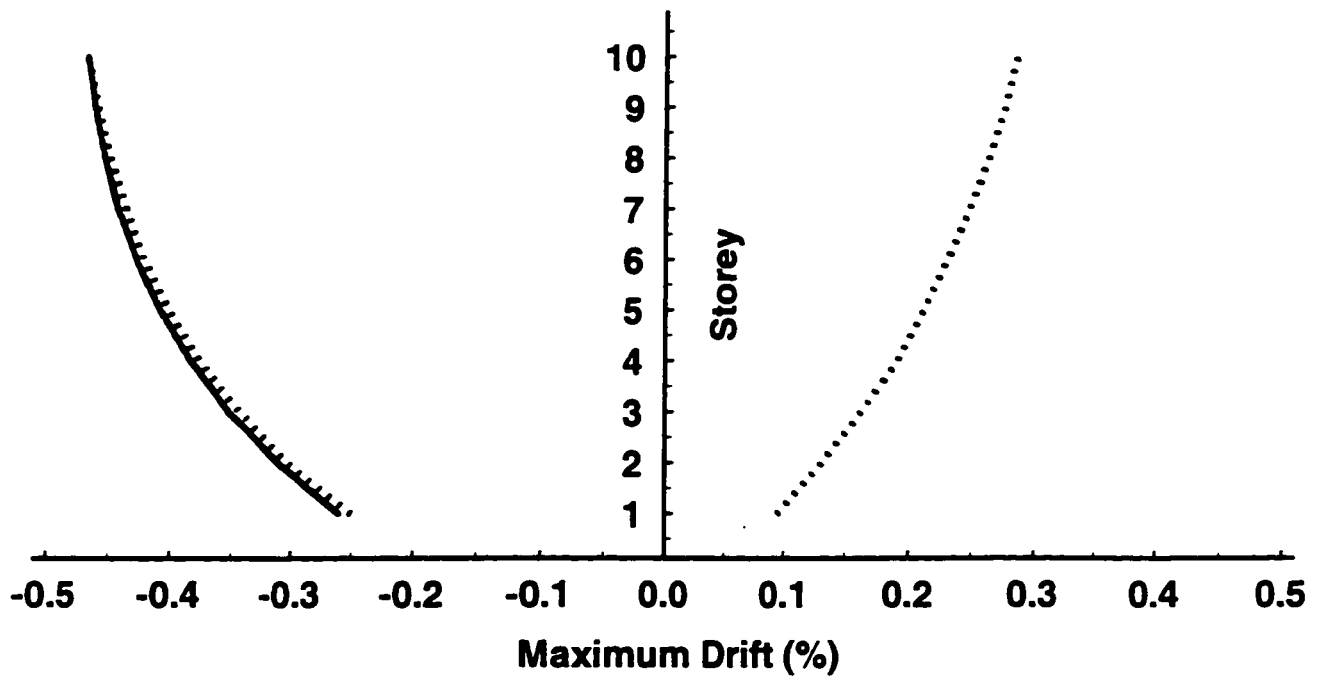
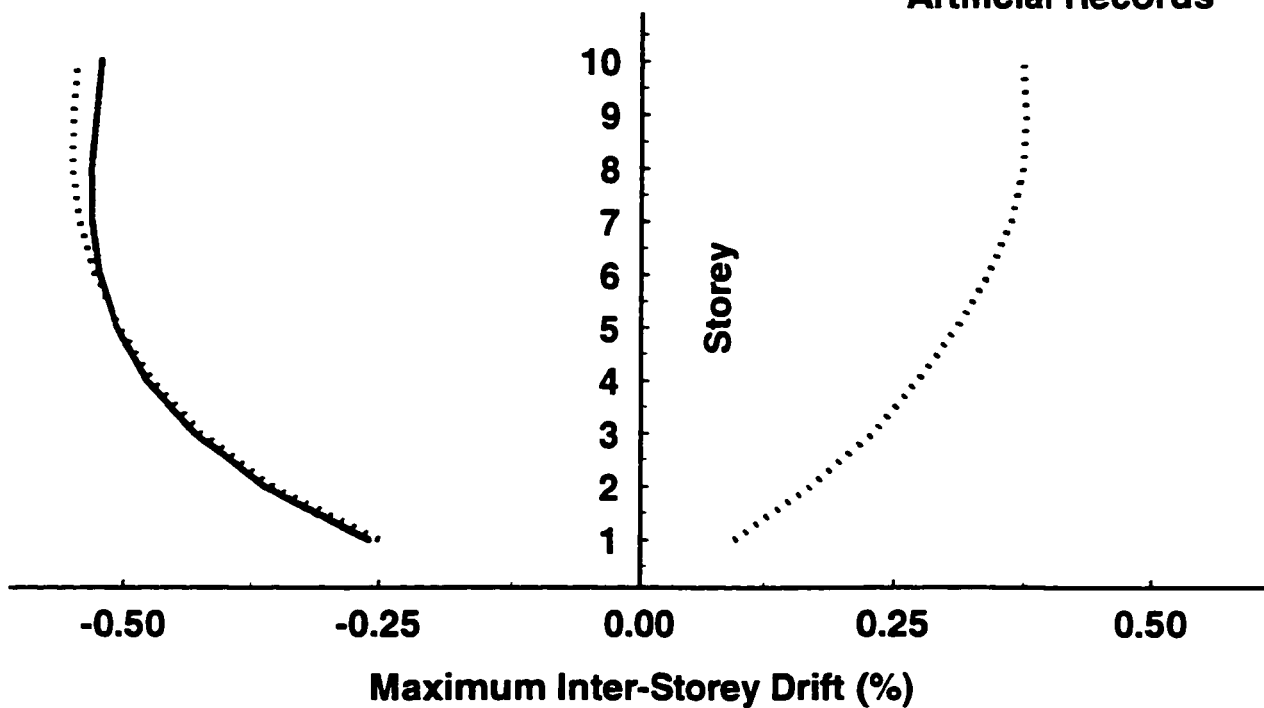


Fig. 6.34 Comparison of Static and Dynamic Ductility Response for 5-Storey SHW building in Vancouver.



(a)

— Push-Over Analysis
 Dynamic Analysis
Artificial Records



(b)

Fig. 6.35 Comparison of Static and Dynamic Drift Response for 10-Storey SHW building in Vancouver.

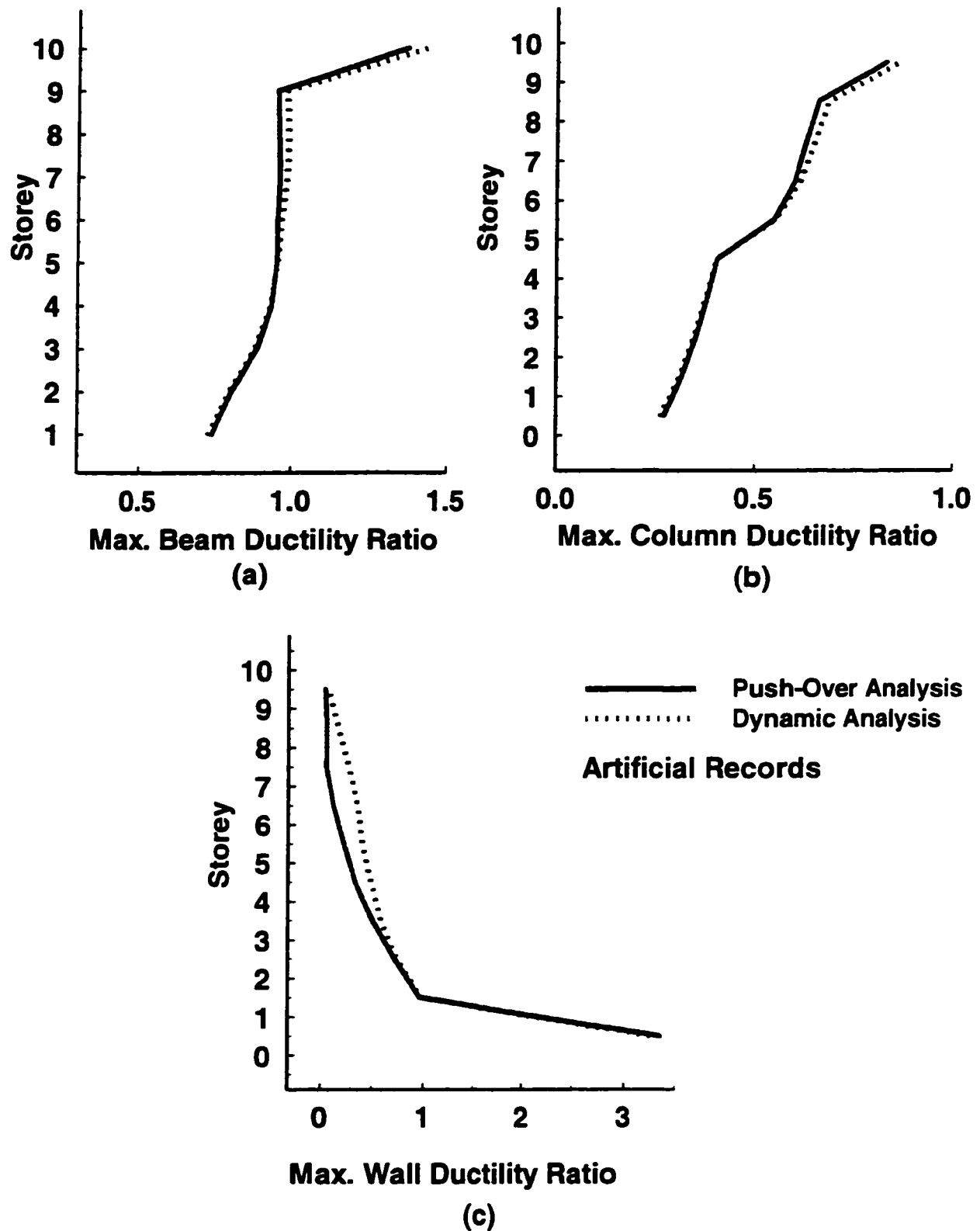
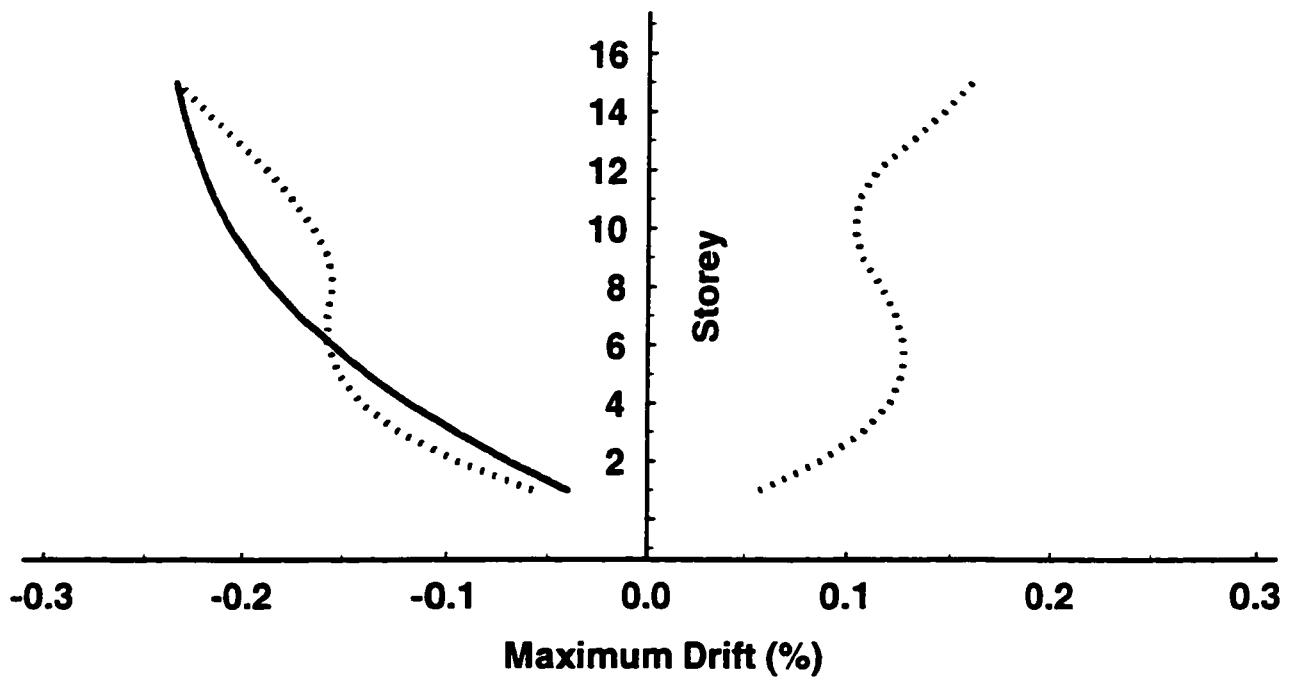
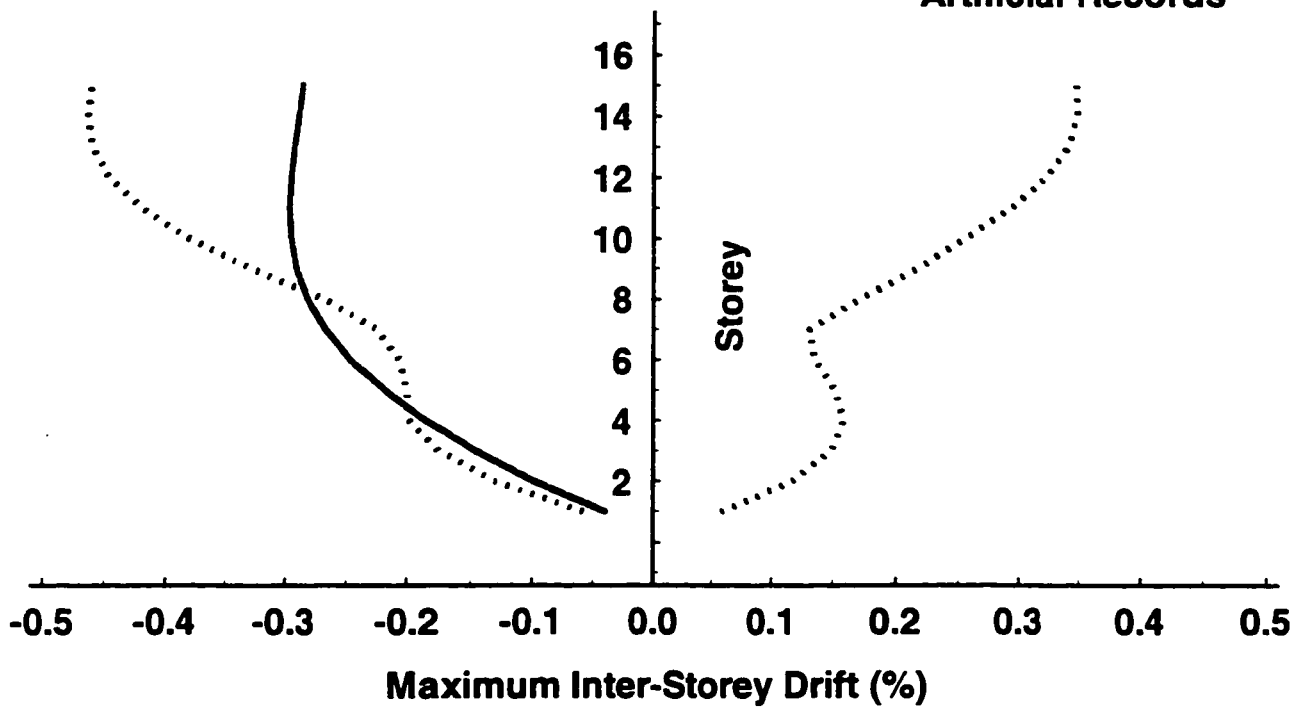


Fig. 6.36 Comparison of Static and Dynamic Ductility Response for 10-Storey SHW building in Vancouver.



(a)

— Push-Over Analysis
 Dynamic Analysis
Artificial Records



(b)

Fig. 6.37 Comparison of Static and Dynamic Drift Response for 15-Storey SHW building in Vancouver.

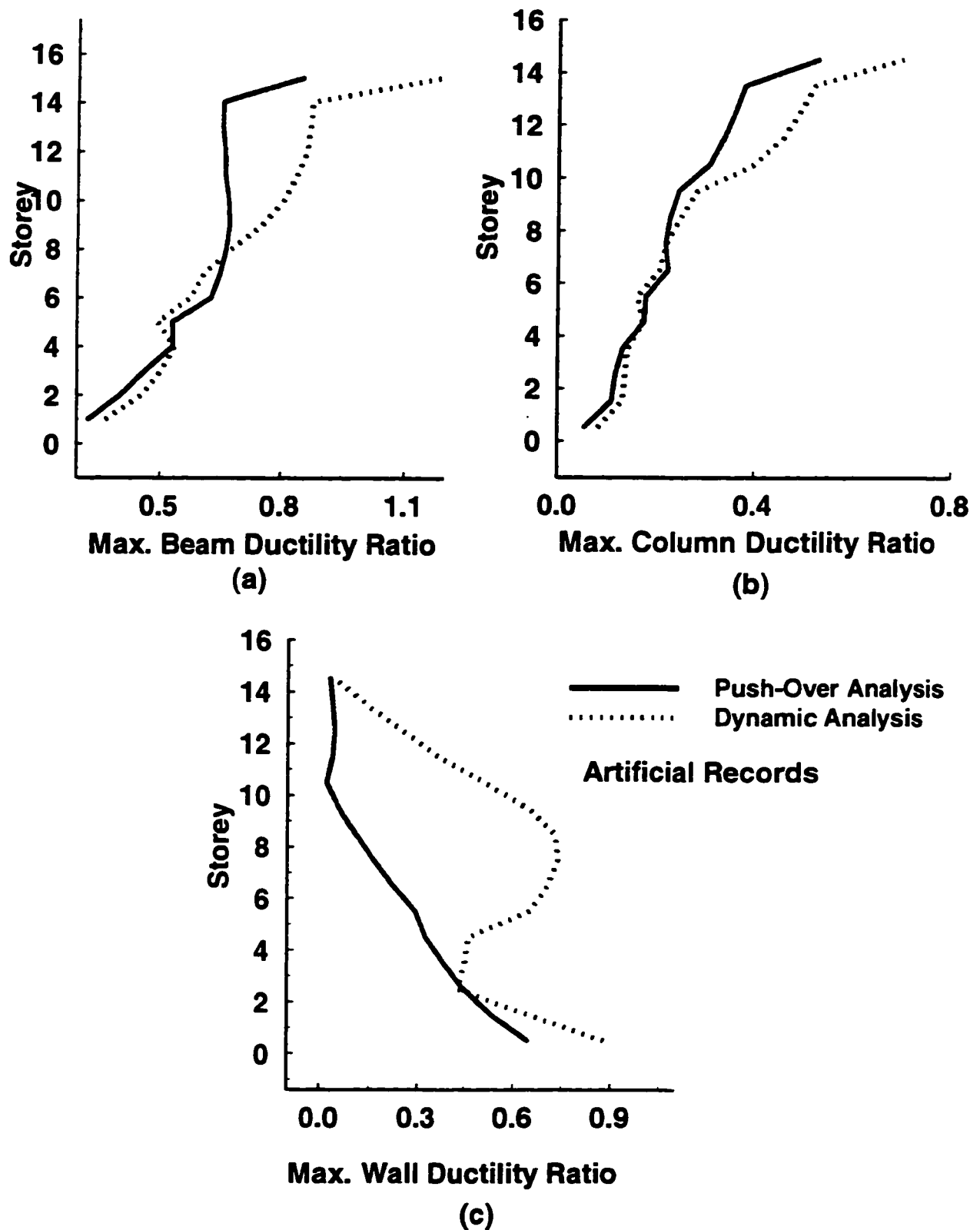


Fig. 6.38 Comparison of Static and Dynamic Ductility Response for 15-Storey SHW building in Vancouver.

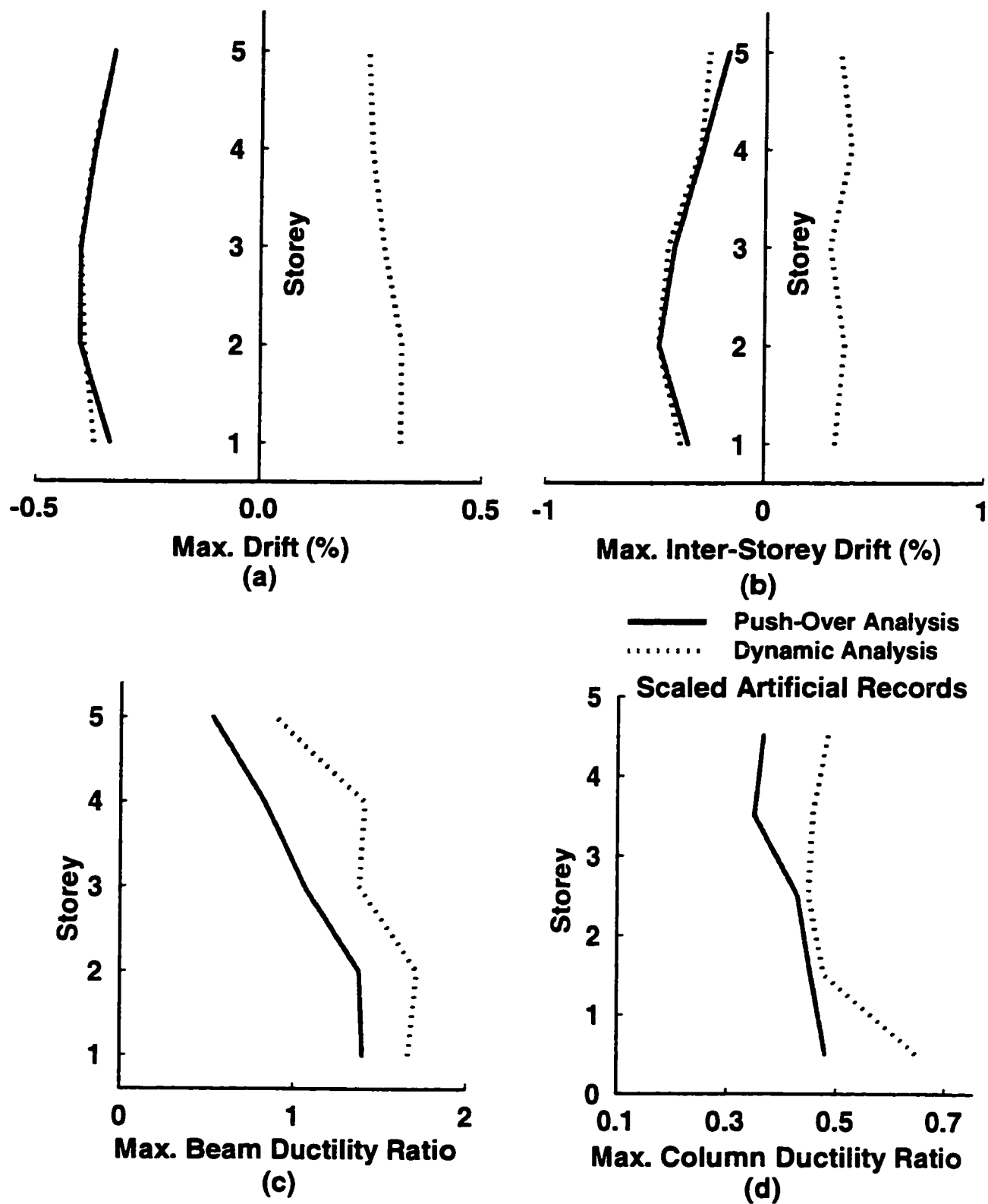


Fig. 6.39 Comparison of Static and Dynamic Drift and Ductility Response for 5-Storey MRF building in Ottawa.

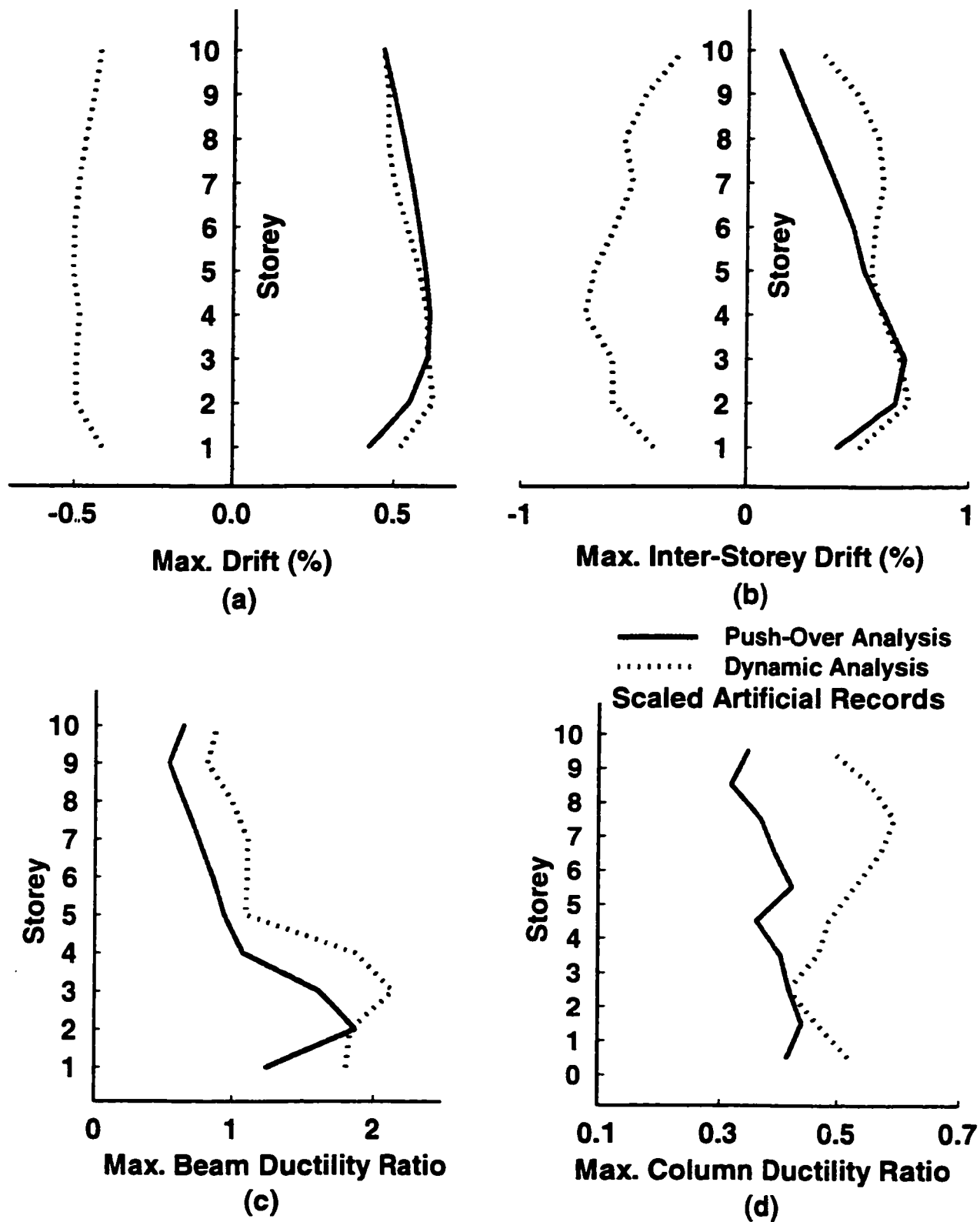


Fig. 6.40 Comparison of Static and Dynamic Drift and Ductility Response for 10-Storey MRF building in Ottawa.

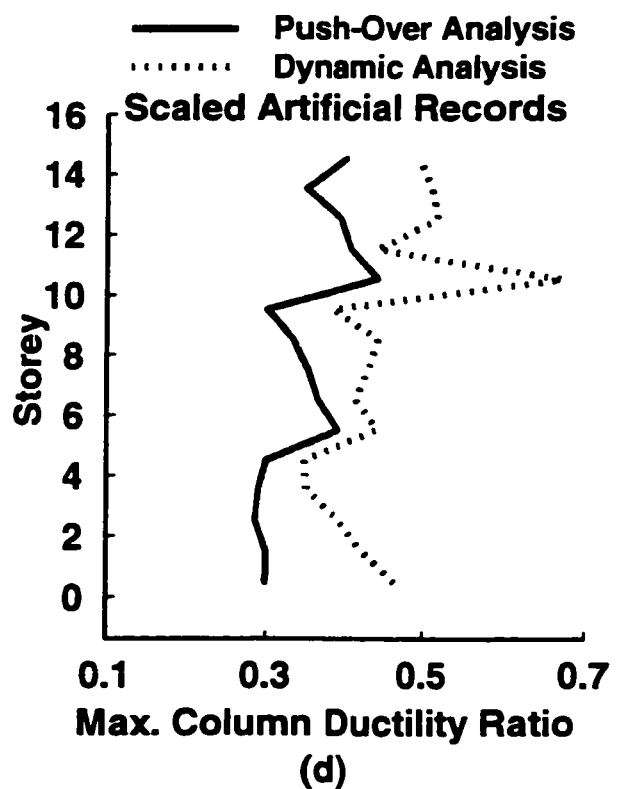
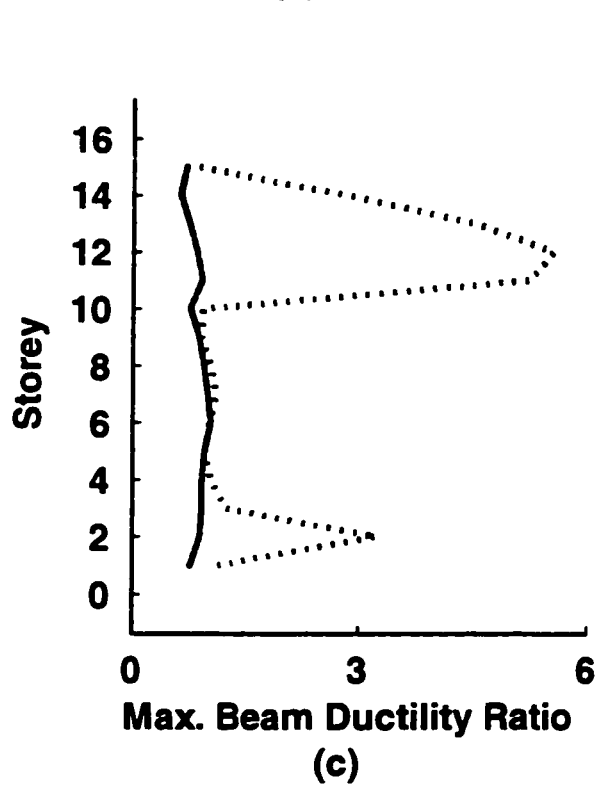
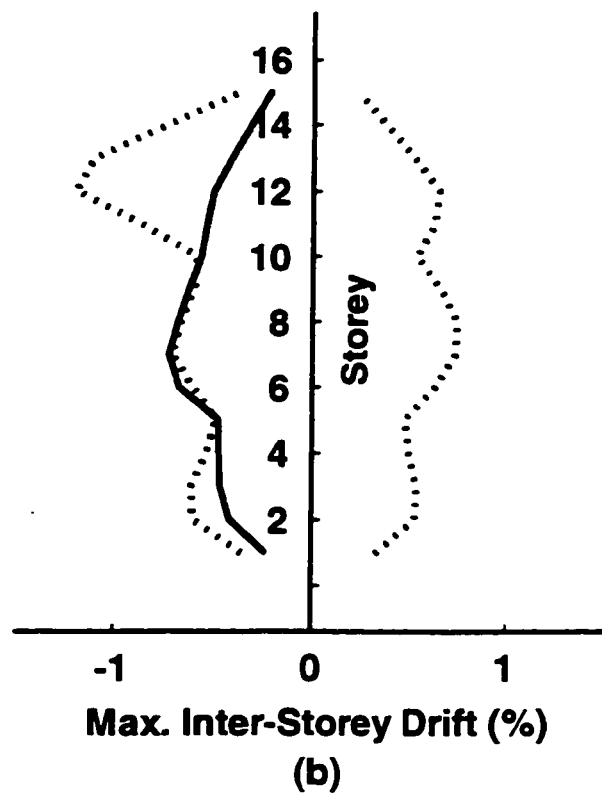
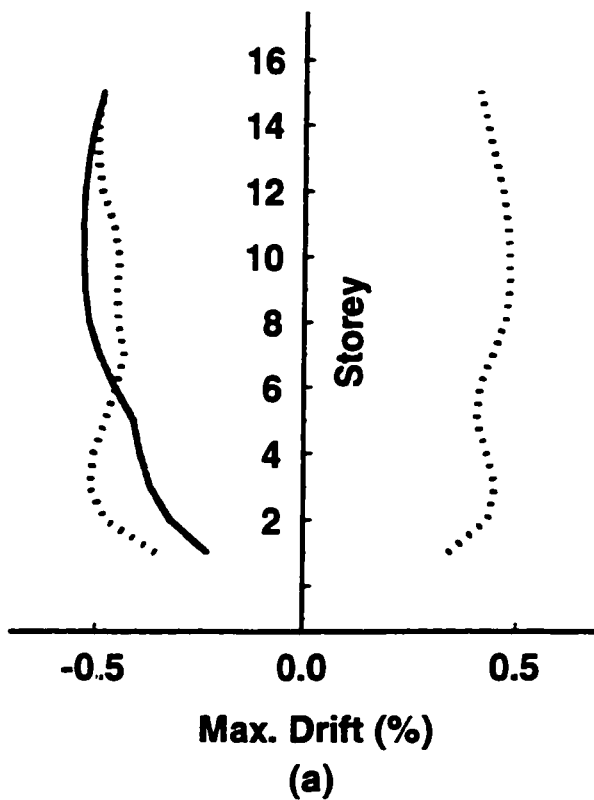
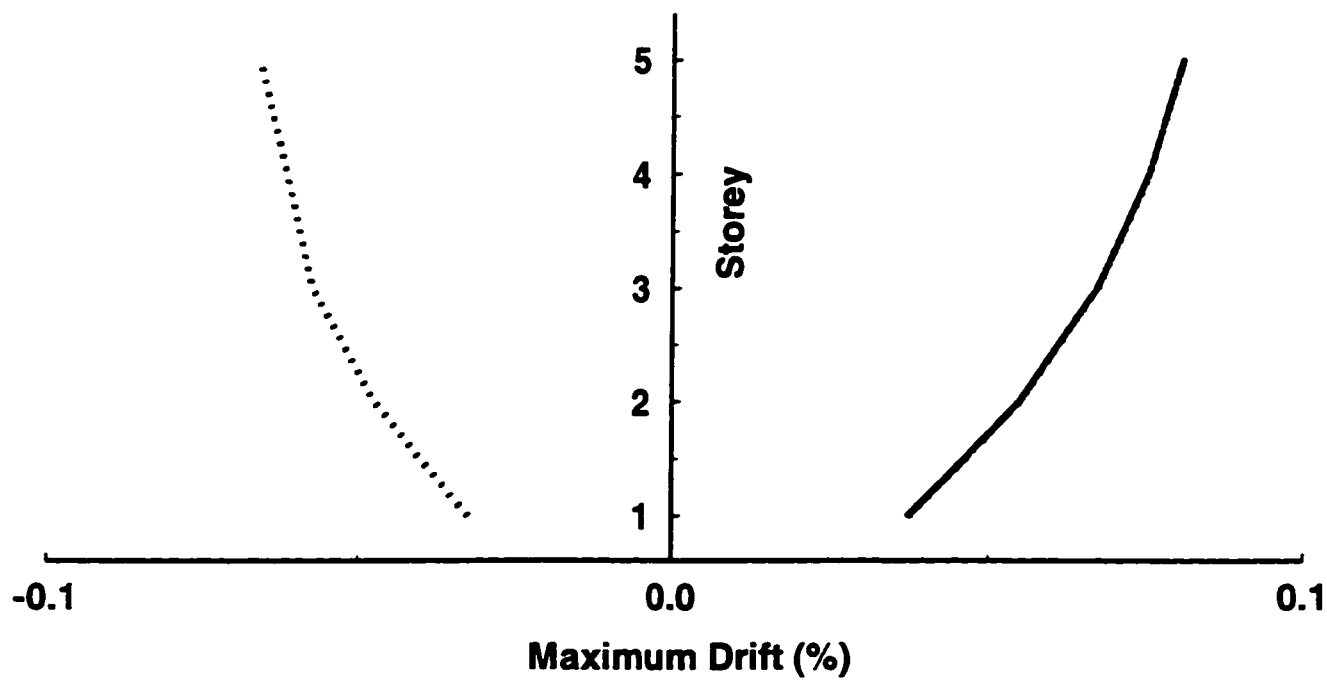
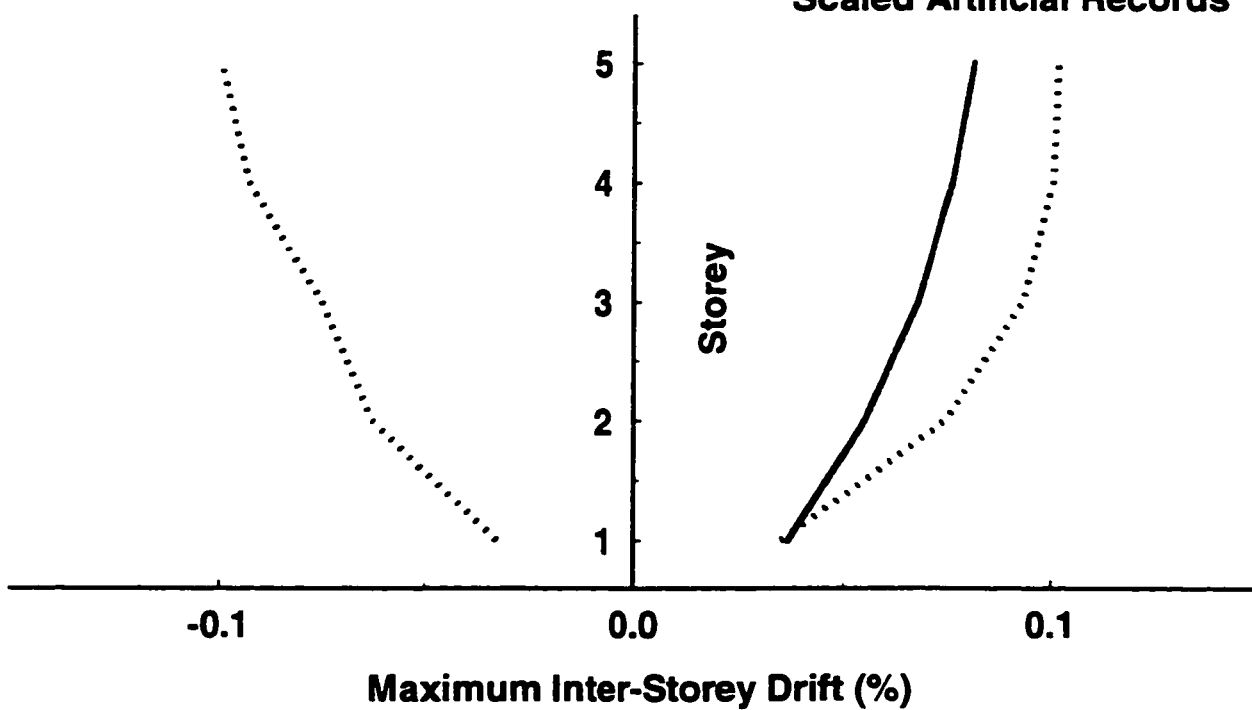


Fig. 6.41 Comparison of Static and Dynamic Drift and Ductility Response for 15-Storey MRF building in Ottawa.



(a)

— Push-Over Analysis
 Dynamic Analysis
Scaled Artificial Records



(b)

Fig. 6.42 Comparison of Static and Dynamic Drift Response for 5-Storey SHW building in Ottawa.

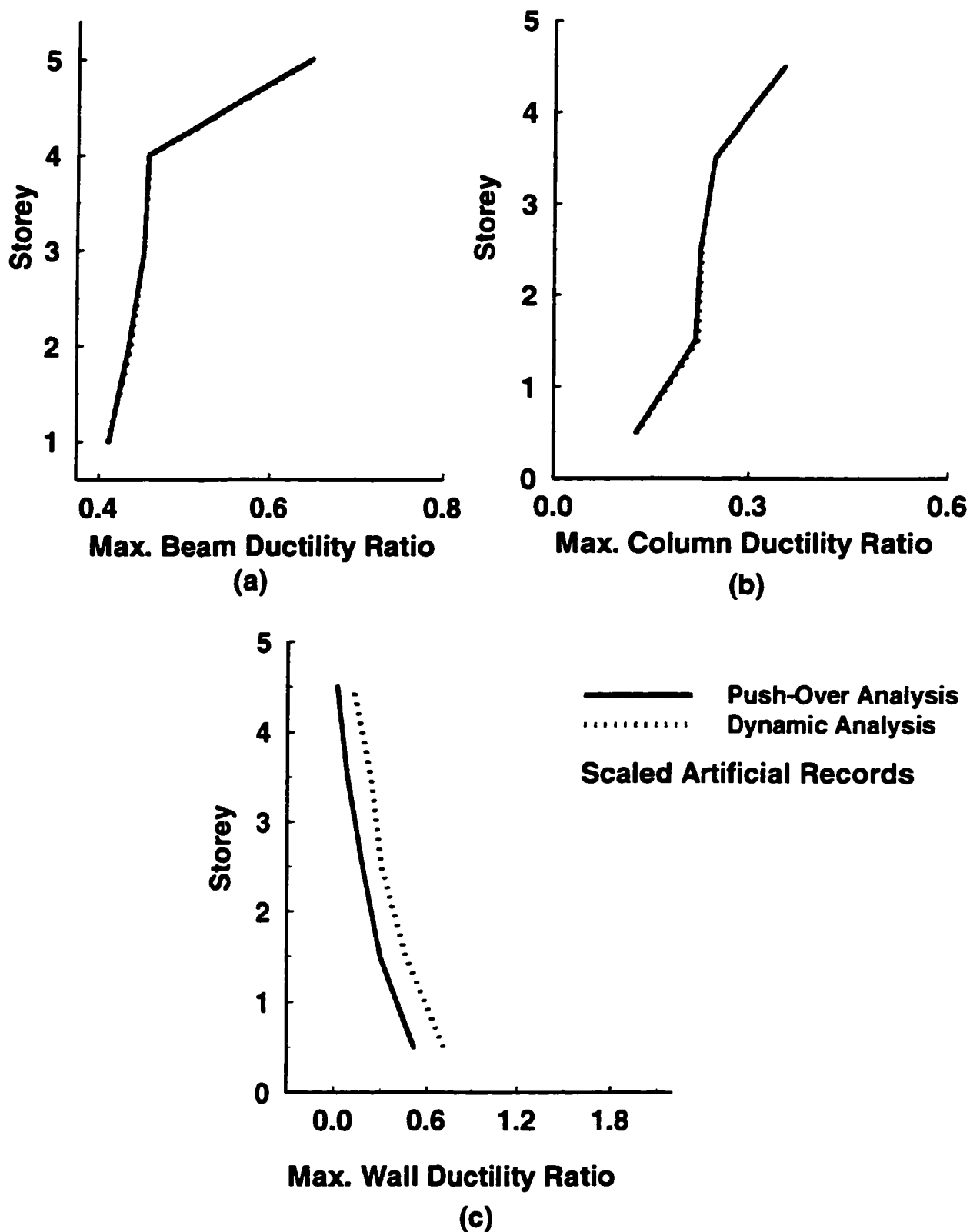
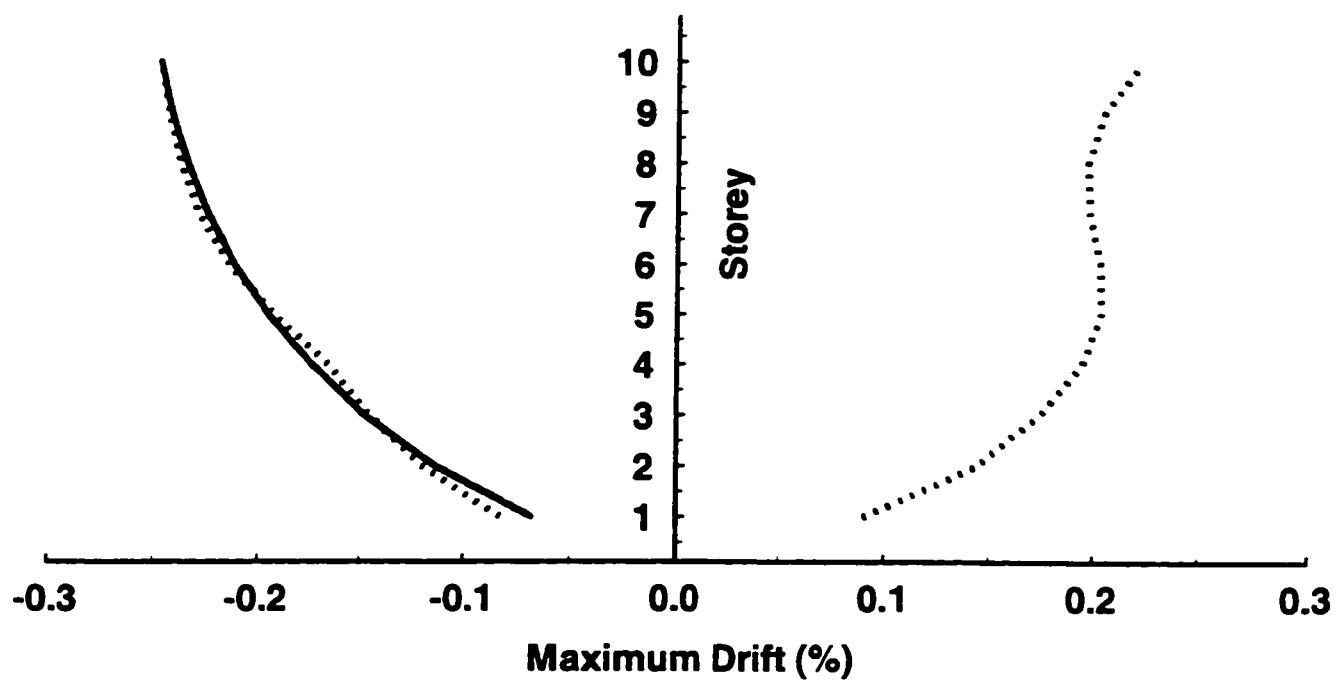
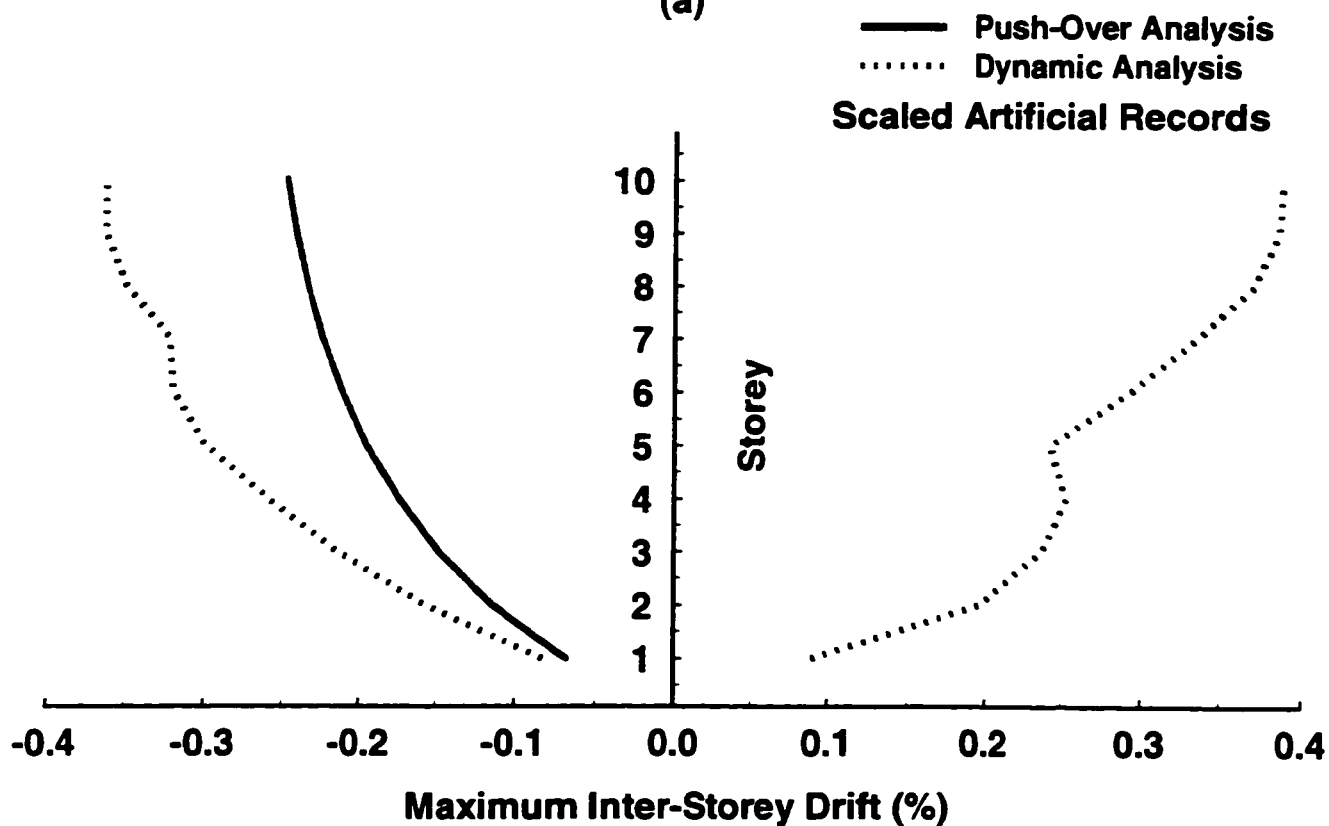


Fig. 6.43 Comparison of Static and Dynamic Ductility Response for 5-Storey SHW building in Ottawa.



(a)



(b)

Fig. 6.44 Comparison of Static and Dynamic Drift Response for 10-Storey SHW building in Ottawa.

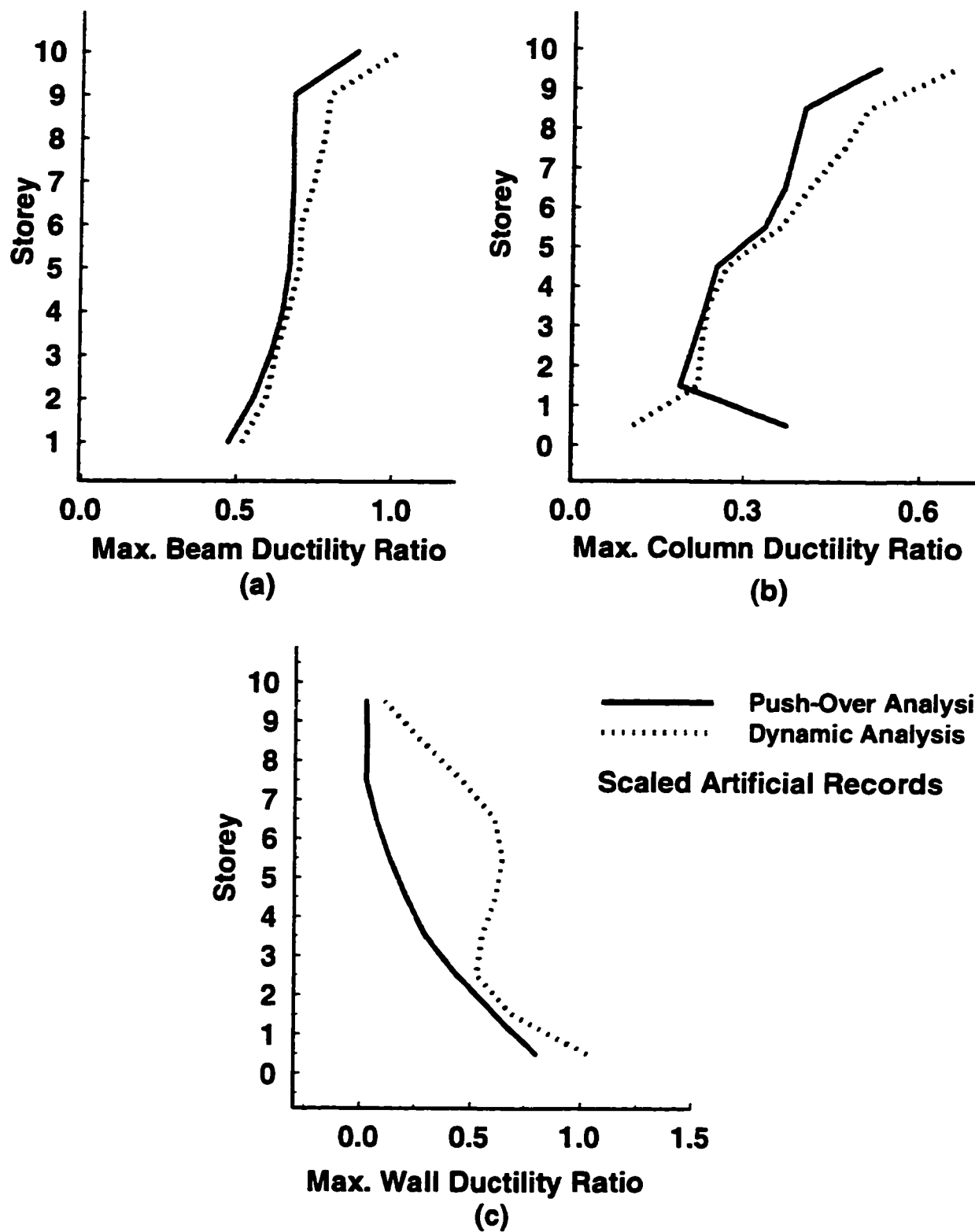
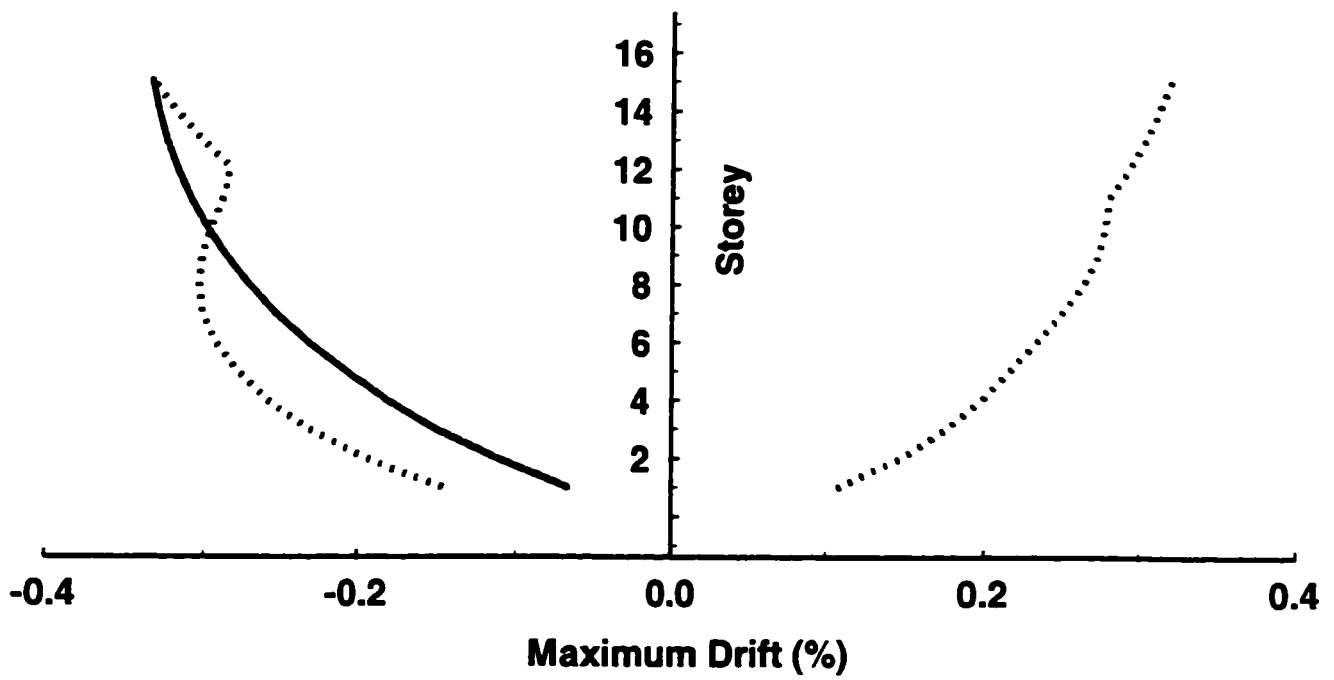
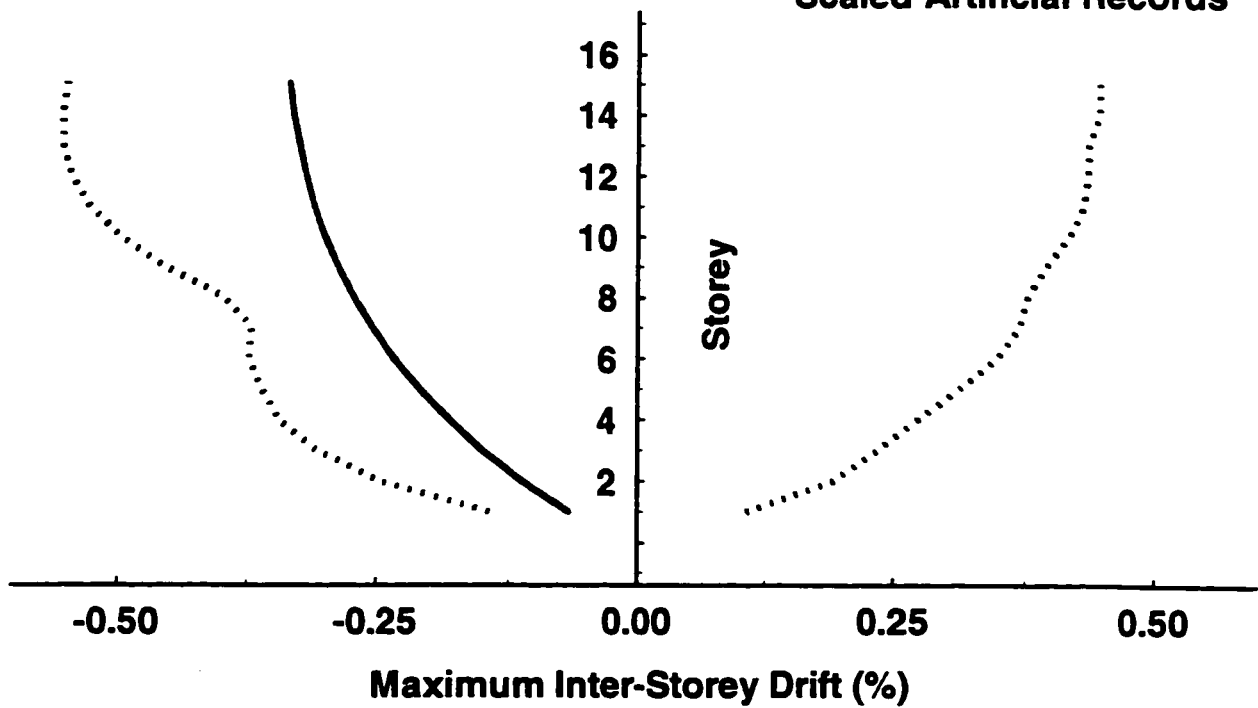


Fig. 6.45 Comparison of Static and Dynamic Ductility Response for 10-Storey SHW building in Ottawa.



(a)

— Push-Over Analysis
 Dynamic Analysis
 Scaled Artificial Records



(b)

Fig. 6.46 Comparison of Static and Dynamic Drift Response for 15-Storey SHW building in Ottawa.

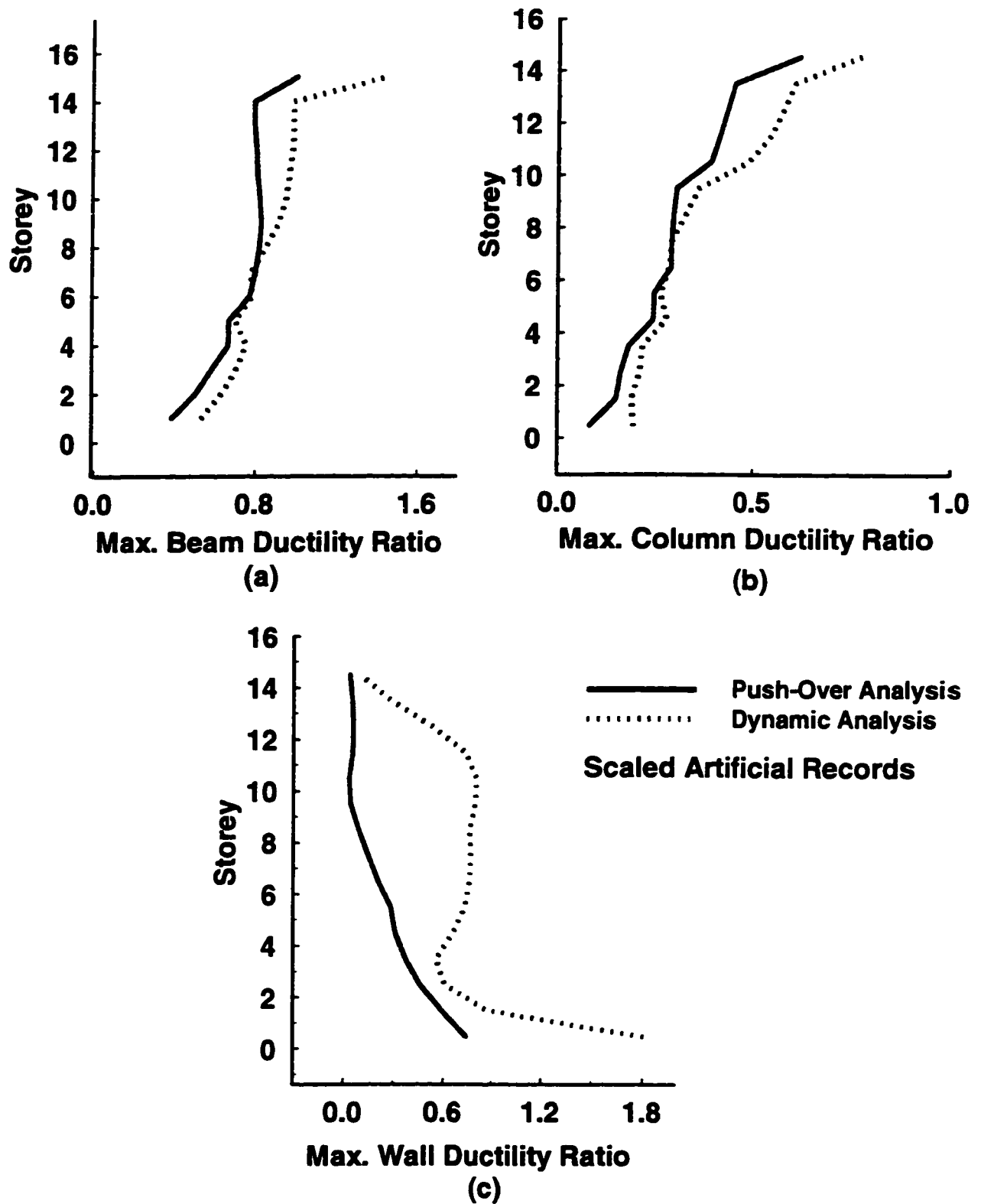


Fig. 6.47 Comparison of Static and Dynamic Ductility Response for 15-Storey SHW building in Ottawa.

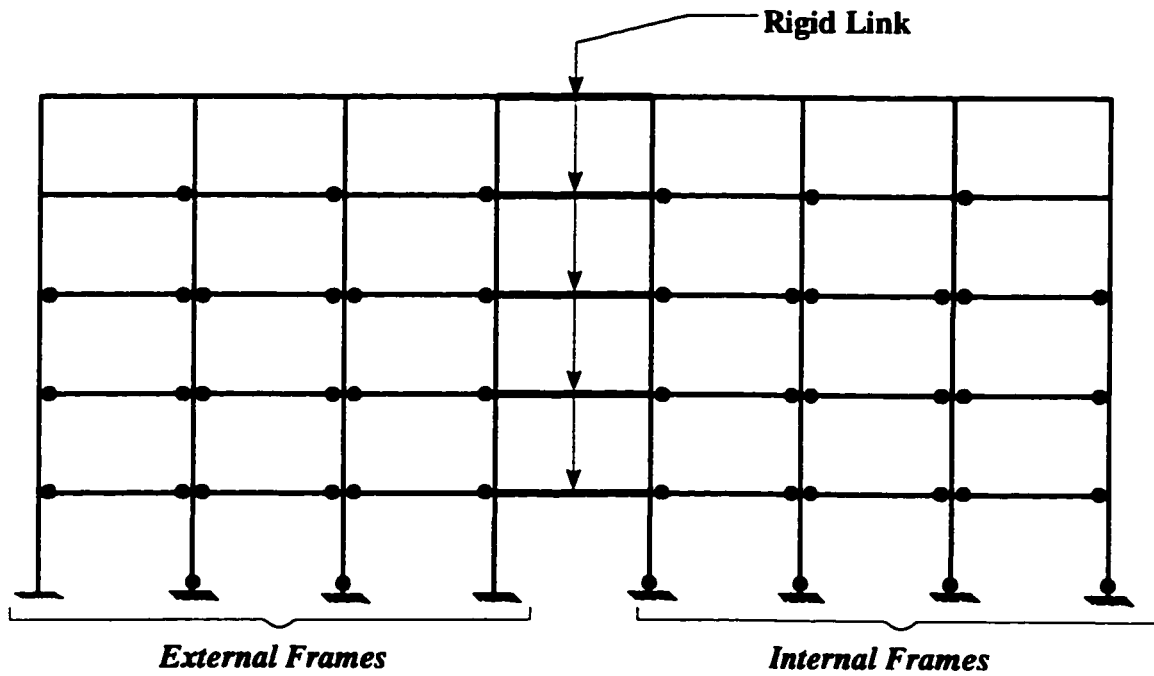
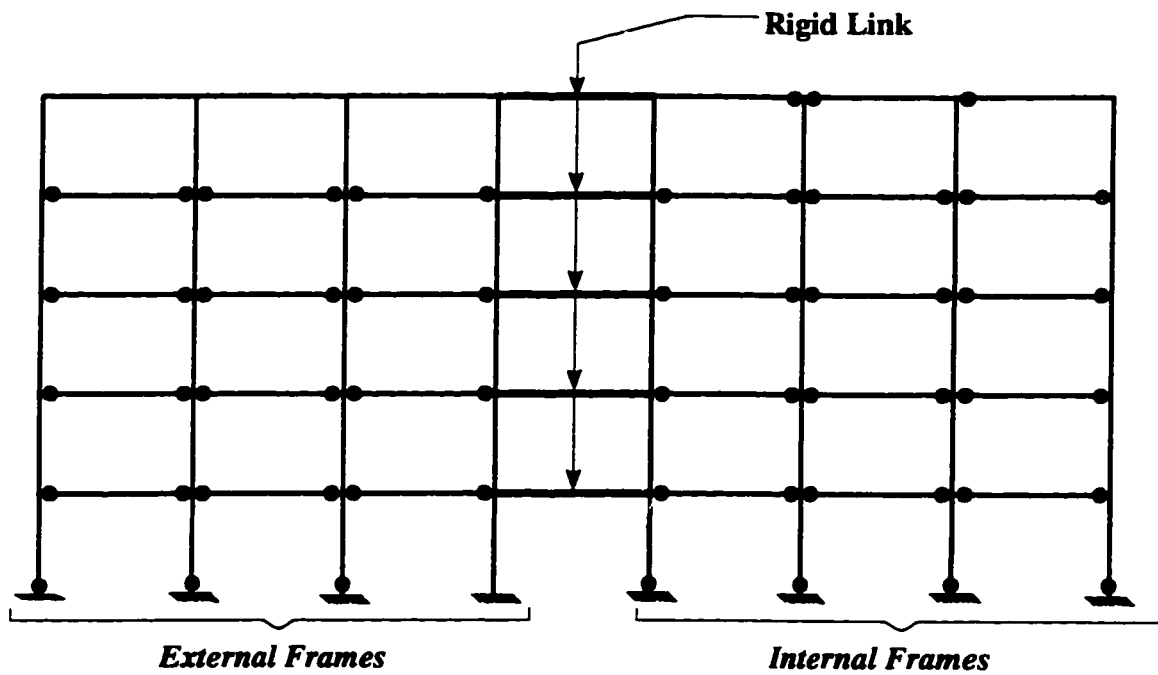
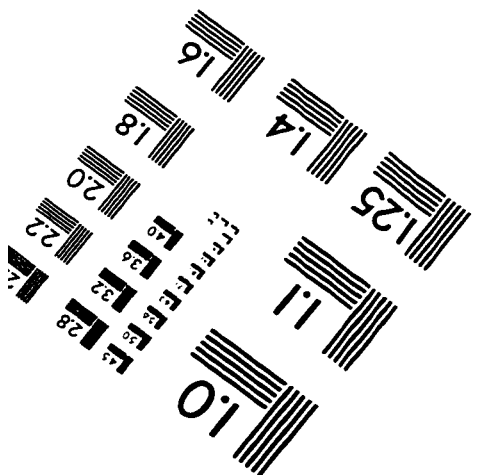
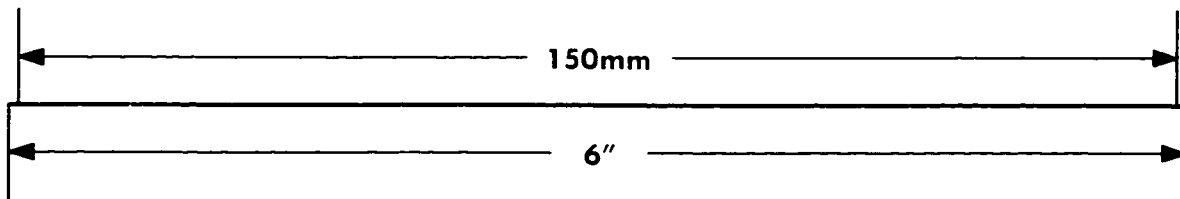
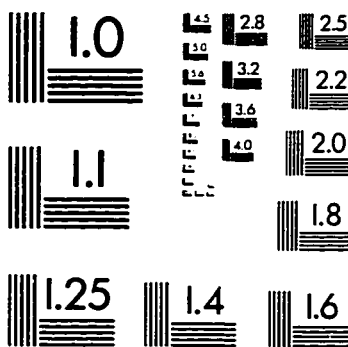
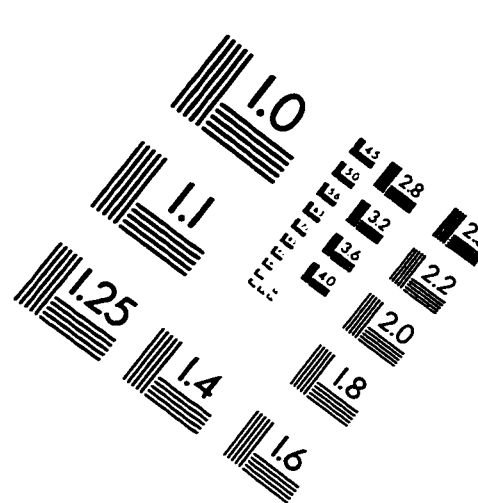
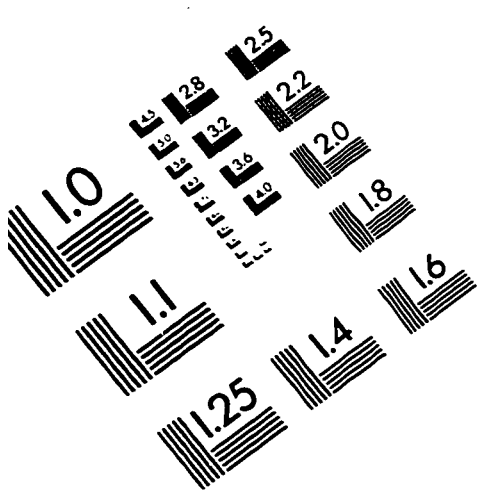


Fig. 6.48 Yielding Pattern Obtained by Dynamic and Push-Over Analysis of 5-storey MRF in Vancouver.

IMAGE EVALUATION TEST TARGET (QA-3)



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