

**Numerical and Experimental Investigation of the Seismic
Performance of Steel Concentrically Braced Frame Structures**

by
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Declaration

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Summary

The concepts of performance-based earthquake engineering (PBEE) and performance based seismic design (PBSD) emerged after the 1994 Northridge and 1995 Kobe earthquakes, where the level of damage to structures, economic loss due to loss of use and cost of repair were unexpectedly high. PBSD involves achieving stated performance objectives when a structure is subjected to a particular level of seismic hazard. In order to allow PBSD to be implemented, performance assessment procedures have been developed and refined through the work of Pacific Earthquake Engineering Research (PEER) centre and the FEMA P-58 research project.

Steel Concentrically Braced Frames (CBFs) are stiff, lightweight, potentially low-cost structures that efficiently resist lateral load through diagonal bracing members. Under dissipative seismic design principles, these bracing members are allowed to behave inelastically during infrequent high-intensity earthquakes. Recent earthquakes have shown that steel frame structures designed to modern codes are unlikely to collapse even under very severe seismic loads. However significant damage, and subsequent losses, can be incurred by both structural and particularly non-structural building components, highlighting the need for PBSD-type procedures. This is particularly important for CBFs, where the high inherent stiffness, and the resulting difficulty in coordinating structural and non-structural response, makes achieving adequate performance challenging. Therefore, this thesis investigates the lifetime seismic performance, and examines methods to improve performance assessment, for CBFs designed to Eurocode 8.

The impact of decisions taken at the design stage on performance of CBFs designed to Eurocode 8 is assessed. This is done by using the FEMA P-58 performance assessment methodology to calculate lifetime losses suffered by a set of CBFs designed using different behaviour factors to vary frame stiffness and strength. Expected losses are shown to increase with the behaviour factor and seismic hazard. The results of the work suggest that the designer has the ability to limit drift dependant losses to by increasing frame stiffness, but that acceleration dependant losses are more difficult to control.

A so-called 'rapid' performance assessment methodology specifically for CBFs designed to Eurocode 8 was proposed. This is based on the development of methods to predict response parameters of interest based on nonlinear static analysis. This avoids the need to carry out computationally intensive nonlinear time history analyses (NLTHA), thus allowing for more practicable application of performance assessment methodologies. Similar values for

performance measures were obtained from the ‘rapid’ method developed here and from conventional NLTHA-based performance assessment.

The calculation of performance measures like expected repair costs or downtime is based on the ability to accurately predict response parameters of interest for various earthquake scenarios. However, this is challenging for CBFs, where response can be highly nonlinear and difficult to model correctly. In order to assess model accuracy, an experimental substructured hybrid simulation testing programme examining the response of a three storey CBF structure was undertaken. A novel model updating algorithm to optimize modelling parameters was developed and validated. This was applied to the results of the hybrid simulations. The primary conclusion from this process was that the numerical models employed appear to overestimate the stiffness of CBF structures, something which can be improved by reducing the elastic modulus. This was supported by analysis of the results of shaking table tests from a previous research project. The conclusions of the performance assessment previously carried out were reassessed in this context. It was demonstrated that the calculation of performance measures is quite robust and reasonably insensitive to modelling assumptions or error.

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Table of Contents

Numerical and Experimental Investigation of the Seismic Performance of Steel Concentrically Braced Frame Structures	1
Declaration	i
Summary.....	ii
Acknowledgements	iv
Table of Contents	v
List of Figures.....	xii
List of Tables.....	xxvii
List of Symbols & Abbreviations.....	xxix
1 Introduction.....	1
1.1 Earthquakes and Structural Design	1
1.2 Statement of Problem	2
1.3 Objectives	3
1.4 Organisation of Thesis	3
2 Seismic Response and Design of CBFs	7
2.1 Introduction.....	7
2.2 CBF Response to Earthquake Loading	7
2.2.1 Global Frame Behaviour	7
2.2.2 CBF Response to Past Earthquakes	9
2.2.3 Brace Behaviour.....	10
2.2.4 Gusset Plates	12
2.3 CBF Design to Eurocode 8	14
2.3.1 Seismic Hazard and Earthquake Actions	14
2.3.2 Design Seismic Actions	15
2.3.3 Dissipative Design & Behaviour Factor.....	17
2.3.4 Slenderness, Overstrength & Other Design Criteria.....	18
2.4 Conclusion	21

3	Numerical Analysis of Seismic Response	22
3.1	Introduction	22
3.2	Numerical Analysis.....	22
3.2.1	Nonlinear Static Analysis.....	22
3.2.2	Nonlinear Dynamic Analysis.....	23
3.3	OpenSees	23
3.3.1	OpenSees Architecture	24
3.3.2	Time History Analysis Using OpenSees	25
3.4	Modelling CBFs in OpenSees.....	29
3.5	Conclusions	37
4	Performance Based Earthquake Engineering	38
4.1	Introduction	38
4.1.1	Probabilistic Seismic Hazard Assessment (PSHA)	40
4.1.2	PEER Performance Assessment Methodology.....	42
4.1.3	FEMA P-58 & PACT	47
4.1.4	Lifetime Performance Assessment of Various Building Types – Numerical Studies 49	
4.2	Selection of Ground Motion Records for Performance Assessment.....	54
4.2.1	Techniques for Selecting Ground Motion Records – UHS, CMS & CS.....	55
4.2.2	Selecting Records to Match a Spectrum	58
4.3	Estimation of <i>EDPs</i> without NLTHA.....	60
4.3.1	Drift Prediction.....	60
4.3.2	Peak Floor Acceleration Response.....	64
4.4	Conclusions & Areas for Further Research	67
5	Hybrid Simulation.....	71
5.1	Introduction	71
5.2	Experimental Testing Procedures	71
5.3	Hybrid Simulation.....	73

5.4	Development of the Hybrid Test Method	74
5.5	OpenFresco.....	76
5.6	Integration Schemes in Hybrid Simulation	77
5.7	Errors in Hybrid Testing	79
5.8	Process Synchronization– Predictor Corrector.....	81
5.9	Online Model Updating in Hybrid Simulation	82
5.10	Hybrid Simulation Facility at Trinity College Dublin	85
5.11	Conclusions & Areas for Further Research.....	88
6	Lifetime Performance Assessment of CBF Structures	90
6.1	Introduction.....	90
6.2	Procedure	91
6.2.1	Case Study Buildings.....	92
6.2.2	Structural Modelling.....	95
6.2.3	Selection of Ground Motion Records for NLTHA.....	98
6.3	Results	100
6.3.1	Seismic Response Analysis.....	100
6.3.2	Expected Losses.....	107
6.3.3	Analysis of Lifetime Losses	118
	Comparison between Two Storey CBFs and MRFs.....	121
6.3.4	<i>EDPs</i>	122
6.3.5	Performance Metrics.....	123
6.3.6	LIF and Return on Investment	124
6.4	Discussion of Results	125
6.5	Conclusions.....	129
7	Rapid Performance Assessment of CBF Structures	131
7.1	Introduction.....	131
7.2	Proposed Rapid Performance Assessment Framework	132
7.2.1	FEMA P-58 Simplified Analysis Procedure.....	132

7.3	Dataset Development	133
7.3.1	Structural Design & Modelling	133
7.3.2	Ground Motion Records	139
7.3.3	Analysis Procedure.....	142
7.3.4	<i>EDP</i> Databank.....	143
7.4	Peak Drift Response Prediction.....	149
7.4.1	Normalised Drift Factors	149
7.4.2	Obtaining Peak Inter-Storey Drift for Performance Assessment.....	164
7.4.3	Variation in δ_{mod} and ϑ_{mod}	168
7.5	Peak Floor Acceleration	172
7.5.1	Floor Acceleration Magnification.....	172
7.6	Rapid Performance Assessment Application	182
7.6.1	<i>EDP</i> Evaluation	183
7.6.2	Collapse & Excessive Residual Drift Estimate	187
7.6.3	Performance Metrics	188
7.7	Conclusion.....	193
8	Experimental Setup for Hybrid Simulation	195
8.1	Introduction	195
8.2	Experimental Setup.....	196
8.2.1	Experimental Facility.....	196
8.2.2	Test Structure Design.....	196
8.2.3	Instrumentation	205
8.3	Material Testing	205
8.4	Hybrid Simulation Setup at TCD.....	206
8.5	Finite Element Model.....	207
8.5.1	Numerical Substructure	207
8.5.2	Integration Algorithm	208
8.5.3	Representation of Experimental Substructure	210

8.5.4	Validation of Experimental Substructure Modelling	214
8.6	Preliminary Testing	216
8.6.1	Random Long Delays	217
8.6.2	NaN Errors	223
8.6.3	Other Issues	226
8.7	Conclusion	226
9	Results of Hybrid Simulations	228
9.1	Introduction	228
9.2	Experimental Programme	228
9.2.1	Selection of Ground Motion Records for Experimental Testing	228
9.3	Experimental Results & Discussion	232
9.3.1	Test Results and Observations	232
9.3.2	Comments & Trends	257
9.4	Comparison with Numerical Models	258
9.5	Conclusion	272
10	Hybrid Simulation with Offline Model Updating	274
10.1	Introduction	274
10.2	Proposed Algorithm for Hybrid Simulation with Online Model Updating in OpenSees-OpenFresco	275
10.2.1	Implementation of Model Updating in OpenSees & OpenFresco	278
10.3	Numerical Verifications	282
10.3.1	Updating in Linear Analysis	282
10.3.2	Multiple Parameter Updating in Non Linear Analysis	286
10.3.3	Updating Multiple Parameters	286
10.3.4	Linear & Nonlinear Parameters	287
10.4	Offline Updating Using Experimental Data	297
10.4.1	Linear Hybrid Simulations	298
10.4.2	Hybrid Simulations at 10% in 50 Year Ground Motion Intensity	301

10.4.3	Recommendations for Numerical Modelling.....	311
10.5	Conclusion.....	312
11	Review of Experimental Results.....	313
11.1	Introduction	313
11.2	Investigation of Modelling Accuracy using BRACED Shaking Table Tests.....	313
11.2.1	BRACED Shaking Table Tests.....	314
11.2.2	OpenSees Modelling of BRACED Tests.....	315
11.2.3	Updated Modelling Assumptions.....	320
11.3	Impact of modelling inaccuracies on Performance Measures.....	324
11.3.1	BRACED Test Results	324
11.4	Performance Assessment Using Updated Modelling Assumptions.....	328
11.5	Assessment of Rapid Performance Assessment Methods Using Experimental Results	
	333	
11.5.1	Global Drift – δ_{mod}	335
11.5.2	Inter-Storey Drift – ϑ_{mod}	335
11.5.3	Peak Floor Acceleration	341
11.6	Conclusions	343
12	Conclusions	345
12.1	Completed Work.....	345
12.2	Main Findings.....	347
12.3	Recommendations for Future Work	349
13	References	352
Appendix A	Step-by-step Summary of Performance Assessment Procedure.....	362
Appendix B	Details of Structural Designs	365
Appendix C	Details of Selected Ground Motion Records.....	367
Appendix D	Sample OpenSees Code for NLTHA.....	375
Appendix E	Connection Design Calculations.....	400
Appendix F	Drawings of Experimental Substructu	404

Appendix G	Model Updating OpenSees Code	412
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List of Figures

Figure 1-1 Examples of frame configuration: a) Concentrically Braced Frame (CBF) b) Moment Resisting Frame (MRF) c) Eccentrically Braced Frame (EBF).....	2
Figure 2-1 CBF behaviour under lateral load (adapted from Tremblay (2002))	7
Figure 2-2 Post-elastic CBF behaviour under lateral load (adapted from Tremblay (2002)).....	8
Figure 2-3 Simplified representation of inelastic and elastic behaviour; it can be seen that reduced forces are experienced when the structure is allowed to yield as opposed to remaining elastic	9
Figure 2-4 Example of a buckled brace in a CBF structure after the 2010 Christchurch earthquake (Clifton et al., 2011)	10
Figure 2-5 Typical post-elastic response of a bracing member under cyclic axial load: a) A single loading cycle and b) multiple load cycles (Goggins et al., 2006)	12
Figure 2-6 Illustrations of gusset plate behaviour and designs; a) Whitmore width as used in gusset plate design (Hunt, 2013), b) Illustration of out-of-plane brace buckling and hinge formation in gusset plates (Cochran, 2003), c) Typically employed Standard Linear Clearance model (Hunt, 2013) and d) Innovative elliptical clearance design (Lehman et al., 2008)	13
Figure 2-7 (a) Seismic hazard zonation map of Italy (Solomos et al., 2008) and (b) design level seismic hazard in Europe (Giardini et al., 2013)	15
Figure 2-8 Examples of Response Spectra: a) Response spectra from four different ground motion records showing typical spectral shape; b) Eurocode 8 Type I elastic spectra for different ground conditions calculated using a design ground acceleration value of 0.3g assuming 5% damping. Ground conditions are defined by the average shear wave velocity and typically soil becomes softer moving from A (typically rock) to D (typically cohesion-less soil).	16
Figure 2-9 Eurocode 8 elastic spectrum, $S_e(T_1)$, and design spectra, $S_d(T_1)$, calculated using $q=2$, $q=3$ and $q=4$ for a design ground acceleration of $a_g = 0.3g$ and site class C ground conditions	16
Figure 2-10 Distribution of seismic base shear as horizontal loads at each floor level for design of regular structures to EC8 using the equivalent static force design method	17
Figure 3-1 High level OpenSees objects.....	24
Figure 3-2 Components of the OpenSees Domain	25
Figure 3-3 Analysis object in OpenSees	26
Figure 3-4 Illustration of how the <i>BandGeneral</i> object efficiently stores a banded 6x6 matrix as a 6x3 matrix (http://opensees.berkeley.edu/wiki/index.php/BandGeneral_SOE).....	27
Figure 3-5 Concentrated and distributed plasticity models (Deierlein et al., 2010)	30

Figure 3-6 OpenSees element discretization showing: a) Brace finite element model with initial camber, elements and nodes; b) Integration points along an element; c) Fibre discretization of the cross section and d) Material stress-strain response associated with a fibre	31
Figure 3-7 Example stress-strain relation for the Steel02 material	32
Figure 3-8 Variation in recommended initial brace camber calibrated from experimental data with the range of non-dimensional slenderness values of the bracing members used in the experimental investigation.....	33
Figure 3-9 Illustration of gusset plate models in OpenSees; (a) Pinned Connection Model, (b) Rotational Spring Model (Hsiao et al., 2012), (c) Fibre Connection Model (Uriz et al., 2008).....	34
Figure 3-10 Illustration of a CBF structure where seismic resistance is provided by central bay of the perimeter frame only	36
Figure 3-11 Illustration of gravity loads in an OpenSees model including fictitious leaning column element for a 5 storey structure with seismic resistance provided by perimeter CBFs	37
Figure 4-1 Non-structural damage after the 2011 Christchurch earthquake (Clifton and MacRae, 2013).....	38
Figure 4-2 Damage windows and ceiling in a low-rise steel CBF in the Maule earthquake in Chile 2010 (Saatcioglu et al., 2013)	39
Figure 4-3 Summary of Performance Based Design introducing a) the basic idea of performance objectives (SEAOC, 1995) and b) an outline of the Performance Based Design procedure (FEMA 365, 2000).....	40
Figure 4-4 Example seismic hazard curve for a site in Oakland, California showing the Mean Annual Frequency of Exceedance (MAFE) of different ground motion intensities for structures with a fundamental period of 0.3s	41
Figure 4-5 Examples of fragility functions for (a) HSS bracing members, $f_y = 355 \text{ N/m}^2$ (Karamanci and Lignos, 2014) and (b) suspended ceiling (FEMA, 2012a).....	44
Figure 4-6 Example of a collapse fragility function for a 10 storey CBF in Seattle; with some intensity levels of interest marked	45
Figure 4-7 Examples of probability distributions describing (a) the probability of demolition given residual drift, $P(D RISD)$ and (b) the probability of residual drift for a given earthquake intensity $P(RISD MCE)$ for a case study building.....	46
Figure 4-8 Flowchart for assessing the performance outcome in each realization in PACT (FEMA, 2012a).....	48
Figure 4-9 Hazard deaggregation for 2% in 50 year probability of exceedance for the Oakland site (United States Geological Survey, 2008)	56

Figure 4-10 Uniform hazard spectra with probabilities of exceedance of 2%, 10% and 50% in 50 years at a site in Oakland, California.....	57
Figure 4-11 Relation between UHS and CMS conditioned at different periods, (Lin et al., 2013b)	58
Figure 4-12 Comparing the impact of different target spectra on results of risk-based assessment for peak inter-storey drift for a 20 storey frame Lin et al. (2013c). It can be seen that: 1) results using CS as the target spectrum are insensitive to the conditioning period, 2) CMS based results are sensitive to the conditioning period 3) UHS based results are conservative and overestimate probability of exceedance of engineering demand parameters.	60
Figure 4-13 Relationships between (a) δ_{mod} and T_1/T_m and (b) ϑ_{mod} and T_1/T_m in MRFs predicted by Kumar, et al. (2013).....	63
Figure 4-14 Comparison between the floor acceleration magnification obtained from NLTHA and using the first mode reduced method (Rodriguez et al., 2002).....	65
Figure 4-15 <i>PFA/PGA</i> for various models of a 2 storey steel MRF for elastic and inelastic response (Flores et al., 2015)	66
Figure 4-16 <i>PFA/PGA</i> for various models of an 8 storey steel MRF for elastic and inelastic response (Flores et al., 2015).....	66
Figure 5-1 Peak drift and acceleration response calculated through NLTHA shown as a percentage of experimentally recorded values (Ryan et al., 2017).....	72
Figure 5-2 Summary of Hybrid Simulation (Carrion and Spencer, 2007).....	74
Figure 5-3 Development of the Hybrid Test Method (Carrion and Spencer, 2007)	74
Figure 5-4 (a) Conventional and (b) continuous PsD tests, as illustrated by (Magonette, 2001).....	75
Figure 5-5 OpenFresco architecture (Schellenberg et al., 2009b)	77
Figure 5-6 a) Event driven strategy for dealing with delays in hybrid simulation (Stojadinovic et al., 2006); b) Illustration (Mosqueda et al., 2005)of the extrapolation and interpolation of the command displacement signal during one integration step as proposed by Nakashima and Masaoka (1999)	82
Figure 5-7 Setup for model updating using weighted average of numerical counterparts as implemented by Kwon and Kammula (2013)	83
Figure 5-8 Illustration of how a weighted average of numerical counterpart responses can be used to match experimental response (Kwon and Kammula, 2013). For the majority of the simulation the highest weighting is given to counterpart 3 followed by counterpart 1, which generally have the closest match to the numerical response	84
Figure 5-9 Illustration of the model updating algorithm implemented by Wu et al. (2018).....	85
Figure 5-10 Hybrid test set up at Trinity College Dublin (McCrum, 2012).....	86

Figure 5-11 Previous hybrid simulation at Trinity College Dublin (McCrum and Broderick, 2013)	86
Figure 5-12 Hybrid simulation setup employed by Yang et al. (2009) showing (a) the experimental substructure and (b) the numerical model	87
Figure 5-13 (a) Schematic and (b) photograph of experimental substructure and (c) numerical model using the GenericElement approach employed by Lin et al. (2012) for hybrid simulation of a 3 storey frame with buckling restrained braces	88
Figure 6-1 Illustration of the procedure followed for performance assessment	91
Figure 6-2 Elevations of case study CBFs	94
Figure 6-3 Plan and Elevation of braced gridlines (GL-1, GL-4, GL-A and GL-D) for the 5 storey case study CBFs	94
Figure 6-4 a) EC8 Type 1 elastic spectrum, S_e , and Uniform Hazard Spectrum for the two sites with fundamental period of case study CBFs marked and b) the seismic hazard curve showing Mean Annual Frequency of Exceedance (MAFE) of ground motion intensity, measured by spectral acceleration at the fundamental period $S_a(T_1)$, for each of the sets of CBFs. Intensity levels where analysis is performed are marked with dots, showing a reasonably even distribution across the hazard curve	95
Figure 6-5 OpenSees Model of 5 storey CBF	97
Figure 6-6 Discretized fiber section distributed plasticity bracing element and gusset plate model	98
Figure 6-7 Examples of ground motion records selected to match the conditional spectrum (CS) at the maximum credible and design level earthquake intensities (2% and 10% probability of exceedance in 50 years respectively) for the 5 storey CBFs at the Oakland site	99
Figure 6-8 Oakland site 2 Storey CBF	101
Figure 6-9 Seattle site 2 Storey CBF	101
Figure 6-10 Oakland site 5 Storey CBF	102
Figure 6-11 Seattle site 5 Storey CBF	102
Figure 6-12 Oakland site 10 Storey CBFs	102
Figure 6-13 Seattle site 10 Storey CBFs	102
Figure 6-14 Median peak inter-storey drift and peak floor acceleration at each storey level for the 5 storey CBFs at the Seattle site at MCE, DLE and SLE	103
Figure 6-15 Value of peak residual inter-storey drift obtained from NLTHA and calculated using Equation 6.1 for (a) 5 storey CBFs at the Oakland site designed using $q=2$ and $q=4$ and (b) 10 storey CBFs at the Oakland site designed using $q=2$ and $q=4$. The probability of demolition given these residual inter-storey drift values is also marked.	104

Figure 6-16 Example of fitting a lognormal distribution to collapse data in order to develop collapse fragility curve	106
Figure 6-17 Collapse fragility functions for 10 storey CBFs; showing also the spectral acceleration at the DLE, MCE and 1% in 50 year (the maximum spectral acceleration considered in the FEMA P-58 performance assessment calculation) intensity levels.....	107
Figure 6-18 Simplified representation of the impact of β_m on EDP distribution for the development of PACT realizations.....	110
Figure 6-19 Present value of economic losses.....	112
Figure 6-20 Present value of lifetime losses as a percentage of initial costs for the case study CBFs.....	113
Figure 6-21 Intensity-specific losses for the two storey CBFs.....	115
Figure 6-22 Intensity-specific losses for 5 storey CBFs	116
Figure 6-23 Intensity specific losses for 10 storey CBFs	116
Figure 6-24 Impact of residual drift calculation method on the lifetimes losses experienced by the 10 storey CBFs at the Oakland site	118
Figure 6-25 Impact of residual drift calculation method on the intensity-specific losses experienced by the 10 storey CBFs at the Oakland site	118
Figure 6-26 Fitting a linear trend line to total expected losses for the 5 storey CBFs; the slope of this line is the LIF_q value.....	119
Figure 6-27 Elevation and Plan of 2 storey moment resisting frame (MRF).....	122
Figure 6-28 Comparison of Peak Inter-storey Drift and Peak Floor Acceleration for 2 storey CBF and MRF for Oakland site	123
Figure 6-29 Present value of expected losses for 2 storey CBFs and MRFs at the Oakland site	124
Figure 6-30 Influence of choice of Brace Fragility Function (BFF1 = FEMA P-58 ID B1033.022, BFF2 = FEMA P-58 ID B1033.032) on lifetime losses for the 10 storey CBFs at the Oakland site.	128
Figure 7-1 (a) Performance assessment procedure employed in Chapter 6 and (b) proposed 'rapid' performance assessment procedure developed here	132
Figure 7-2 Elevations of frames analysed: bay widths = 6m, storey heights = 3.6m.....	135
Figure 7-3 OpenSees model of frame 3_B_0.2 including leaning column element	139
Figure 7-4 Distribution of selected ground motion records with respect to distance and moment magnitude for selected ground motions shown by NEHRP site class (ASCE/SEI, 2010).....	140
Figure 7-5 Illustration of mean period, T_m ; (a) Ground acceleration time history, (b) Fourier transform of ground acceleration showing T_m and (c) Spectral acceleration showing T_m	141
Figure 7-6 Peak inter-storey drift response for 2 storey CBFs for each q_{eff} value	144

Figure 7-7 Peak inter-storey drift response for 3 storey CBFs for each q_{eff} value.....	144
Figure 7-8 Peak inter-storey drift response for 4 storey CBFs for each q_{eff} value.....	145
Figure 7-9 Peak inter-storey drift response for 5 storey CBFs for each q_{eff} value.....	145
Figure 7-10 Peak inter-storey drift response for 6 storey CBFs for each q_{eff} value.....	146
Figure 7-11 Peak inter-storey drift response for 7 storey CBFs for each q_{eff} value.....	146
Figure 7-12 Peak acceleration response for 2 storey CBFs for each q_{eff} value.....	147
Figure 7-13 Peak acceleration response for 3 storey CBFs for each q_{eff} value.....	147
Figure 7-14 Peak acceleration response for 4 storey CBFs for each q_{eff} value.....	148
Figure 7-15 Peak acceleration response for 5 storey CBFs for each q_{eff} value.....	148
Figure 7-16 Peak acceleration response for 6 storey CBFs for each q_{eff} value.....	149
Figure 7-17 Peak acceleration response for 7 storey CBFs for each q_{eff} value.....	149
Figure 7-18 (a) Variation in global drift modification factor, δ_{mod} , and (b) Maximum Drift Modification factor, ϑ_{mod} , with T_1/T_m . Regression lines and data points; $q = 2, 3, 4, 5$	151
Figure 7-19 Variation in model residuals with a series of structural characteristics for δ_{mod}	154
Figure 7-20 Residuals for ϑ_{mod} calculated using Equation 7.9, with γ , β_3 and β_k not included in the regression model.....	155
Figure 7-21 Residuals for ϑ_{mod} calculated using Equation 7.11; showing improved residual behaviour with the developed equation including γ , β_3 and β_k	155
Figure 7-22 δ_{mod} : Regression Models and 95% prediction bounds for each q_{eff} value.....	156
Figure 7-23 ϑ_{mod}: Regression Models and 95% prediction bounds for each q_{eff} value	157
Figure 7-24 Influence of first mode participation ratio, γ , on ϑ_{mod}	158
Figure 7-25 Variation in ϑ_{mod} with T_1/T_m for maximum, average and minimum γ values.....	158
Figure 7-26 Influence of column to brace stiffness ratio, β_k , on ϑ_{mod}	159
Figure 7-27 Variation in ϑ_{mod} with T_1/T_m for maximum, average and minimum β_k	159
Figure 7-28 Influence of column to brace stiffness ratio, β_3 , on ϑ_{mod}	160
Figure 7-29 Variation in ϑ_{mod} with T_1/T_m for maximum, average and minimum β_3 values	160
Figure 7-30 δ_{mod}: Comparison with other models	162
Figure 7-31 ϑ_{mod}: Comparison with other models	163
Figure 7-32 (a)Global drift modification factor, δ_{mod} and (b) Maximum drift modification factor, ϑ_{mod} : regression models for $q = 2, 3, 4, 5$, Comparison with EC8 and AISC Provisions	164
Figure 7-33 Calculation of the normalized ISD Profile from the first mode shape, ϕ^1	165
Figure 7-34 Illustration of the development of inter-storey drift profile for performance assessment by Equation 7.21	166

Figure 7-35 Illustration of how the model for PISD as a function of relative storey height struggles to capture soft-story type behaviour, particularly for higher q_{eff} values for taller structures	168
Figure 7-36 Change in standard deviation of model residuals for δ_{mod} with various parameters.....	169
Figure 7-37 Change in standard deviation of model residuals for ϑ_{mod} with various parameters	169
Figure 7-38 Variation in model residuals and fitted model for lognormal standard deviation with the number of stories	170
Figure 7-39 Data, fitted model and 95% prediction bounds for each q_{eff} examined.....	172
Figure 7-40 Median floor acceleration magnification for each frame height examined.....	173
Figure 7-41 Variation in floor acceleration magnification with T_1/T_m	174
Figure 7-42 Floor acceleration magnification predicted by the model for various T_1/T_m values for 2 storey buildings.....	175
Figure 7-43 Floor acceleration magnification predicted by the model for various T_1/T_m values for 7 storey buildings.....	175
Figure 7-44 Impact of various parameters on model residuals for floor acceleration magnification	176
Figure 7-45 Comparison of various model for PFA/PGA.....	177
Figure 7-46 Distribution of residuals (a) including and (b) excluding the values at ground level	179
Figure 7-47 Model residuals and variation in model residuals with various parameters; standard deviation ($\pm\sigma$) across the dataset shown in red, variation in 16 th and 84 th percentiles (i.e. change in σ) with each parameter shown in blue	180
Figure 7-48 Variation in model residuals with various parameters; standard deviation ($\pm\sigma$) across the dataset shown in red, variation in 16 th and 84 th percentiles (i.e change in σ) each parameter in blue	180
Figure 7-49 Comparison of the trends in variation with the final value for lognormal standard deviation of 0.3	181
Figure 7-50 Comparison of median peak inter-storey drift profile obtained from NLTHA with values predicted by the model developed here using δ_{mod} and the FEMA P-58 simplified analysis method.....	184
Figure 7-51 Comparison of median peak inter-storey drift profile obtained from NLTHA with values predicted by the model developed here using ϑ_{mod} and the FEMA P-58 simplified analysis method.....	185
Figure 7-52 Comparison of median peak inter-storey drift profile obtained from NLTHA with values predicted by the model developed here and the FEMA P-58 simplified analysis method	187
Figure 7-53 Comparison between the Collapse Fragility Functions developed through NLTHA, as discussed in Chapter 6, and using the FEMA P-58 Judgment Based method.....	188

Figure 7-54 Expected losses calculated using <i>EDPs</i> obtained through NTLHA and the rapid method using δ_{mod} for the $q=4, 5$ storey CBF at the Oakland site.....	189
Figure 7-55 Expected losses calculated using <i>EDPs</i> obtained through NTLHA and the rapid method using ϑ_{mod} for the $q=4, 5$ storey CBF at the Oakland site.....	189
Figure 7-56 Comparison of intensity-specific losses calculated using δ_{mod} for the MCE, DLE and SLE for the $q=4$ design at the Oakland site.....	190
Figure 7-57 Comparison of intensity-specific losses calculated using ϑ_{mod} for the MCE, DLE and SLE for the $q=4$ design at the Oakland site.....	190
Figure 7-58 Expected losses calculated using <i>EDPs</i> obtained through NTLHA and the rapid method using δ_{mod} for the $q=2, 5$ storey CBF at the Oakland site.....	191
Figure 7-59 Expected losses calculated using <i>EDPs</i> obtained through NTLHA and the rapid method using ϑ_{mod} for the $q=2, 5$ storey CBF at the Oakland site.....	192
Figure 7-60 Expected losses calculated using <i>EDPs</i> obtained through NTLHA and the rapid method for the $q=4, 5$ storey CBF at the Seattle site; with inter-storey drift at each level calculated based on predicted global drift, i.e. using δ_{mod}	192
Figure 7-61 Expected losses calculated using <i>EDPs</i> obtained through NTLHA and the rapid method for the $q=4, 5$ storey CBF at the Seattle site; with inter-storey drift at each level calculated based on predicted peak inter-storey drift, i.e. using ϑ_{mod}	193
Figure 8-1 Experimental substructure.....	197
Figure 8-2 Schematic of test setup showing experimental substructure in black, reaction frames in yellow, actuator in red and roller guides in blue.....	198
Figure 8-3 Photograph of the test setup at the TCD structure laboratory, showing experimental substructure, parallel reaction frames, roller guides and actuator	198
Figure 8-4 (a) Isometric view of the experimental substructure and (b) Experimental and numerical substructures.....	199
Figure 8-5 Gusset plate design action calculation.....	202
Figure 8-6 Theoretical buckled shape of gusset plate and effective length factor of $K = 1.2$, as proposed by Hunt (2013)	203
Figure 8-7 Design of gusset plate for tensile resistance.....	203
Figure 8-8 Gusset plate design for buckling resistance.....	204
Figure 8-9 (a) Gusset plate dimensions, (b) gusset plate showing yield line, Whitmore width and dimensions for Thornton buckling model	205
Figure 8-10 Gusset and end plate assembly for efficient interchange of brace samples	205
Figure 8-11 Schematic of three-loop hardware architecture at TCD Structures Laboratory.....	207

Figure 8-12 Influence of using elastic or distributed plasticity beams & columns in finite element model on response	208
Figure 8-13 Influence of number of iterations at each time step in the integration algorithm on response, compared to a regular Newmark algorithm where the number of iterations is decided by a convergence test based on the displacement increment at iteration.....	210
Figure 8-14 Experimental setup and finite element model from (McCrum, 2012); the experimental element is represented by two fixed-base columns, one of which is simulated	212
Figure 8-15 Model of the experimental substructure, with nodes and active degrees of freedom labelled, used to develop the macro-element stiffness matrix for hybrid simulation	213
Figure 8-16 Experimental setup and finite element model adopted for this test; experimental substructure modelled as macro-element	214
Figure 8-17 Comparison between actuator node displacement for purely numerical mode and experimental test run with 99% - 1% lateral stiffness distribution and generic macro-element approach (Taiwan40 ground motion record, scale factor = 10.0, PGA = 0.15g).....	215
Figure 8-18 First storey displacement – base shear hysteresis comparison between purely numerical and experimental for a sample ground motion (Coalinga 01 (PEER ID 357) Ground Motion Record, Scale Factor = 1.24, PGA = 0.19)	216
Figure 8-19 Comparison of actuator and roof displacement time histories from purely numerical OpenSees model and hybrid simulation using macro-experimental element for a sample ground motion (Coalinga 01 (PEER ID 357) Ground Motion Record, Scale Factor = 1.24, PGA = 0.19).....	216
Figure 8-20 Illustration of the Predictor-Corrector model controlling actuator movement by predicting before receiving command displacement and correcting after receiving command displacement.....	219
Figure 8-21 Example timestep, showing the workings of the counter within the Predictor-Corrector model.....	220
Figure 8-22 Variation of the atTargetID signal with the counter.....	221
Figure 8-23 Example of correct counter behaviour, including some 'slow down' states.....	221
Figure 8-24 Examples of erratic counter misbehaviour; in both cases the counter fails to equal 20 within a timestep resulting in a delay at the next step due to (a) counter 'overshoot' and (b) counter 'undershoot'	222
Figure 8-25 'Counter controlled' versus 'State controlled' atTargetID for a normal step	223
Figure 8-26 'Counter controlled' versus 'State controlled' atTargetID for a step containing counter misbehaviour.....	223

Figure 8-27 Subsystem to prevent NaN values reaching measSignal, which is read in by OpenFresco.....	225
Figure 8-28 (a)The CheckNaN block which outputs a 1 if either the force or displacement signal value is NaN and (b) the Filter Nan block which	225
Figure 9-1 Spectra of Site Specific (SS) ground motion records selected to match the CMS at the LA Bulk Mail site at 2%, 10% and 50% probability of exceedance in 50 years intensity levels.....	229
Figure 9-2 Spectra of non-Site Specific (NSS) ground motion records, selected to broaden the range of T_m values examined.....	230
Figure 9-3 Location of selected records on T_1/T_m plot	231
Figure 9-4 Experimental response for the SS-GM1-50 simulation.....	234
Figure 9-5 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system.....	235
Figure 9-6 Experimental response for the SS-GM2-50 simulation.....	237
Figure 9-7 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for SS-GM2-50 over the first 20 seconds of the simulation	238
Figure 9-8 Experimental response for the SS-GM3-50 simulation.....	239
Figure 9-9 Example of a spike in the displacement measurement and its impact on force-displacement response	240
Figure 9-10 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM3-50 test run	240
Figure 9-11 Experimental response of the NSS-GM4-50 simulation.....	241
Figure 9-12 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM4-50 test run.....	242
Figure 9-13 Experimental response for the NSS-GM5-50 simulation	243
Figure 9-14 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM5-50 test run.....	244
Figure 9-15 Experimental response for the SS-GM1-10 simulation.....	246
Figure 9-16 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM1-10 test run	247
Figure 9-17 Experimental response for the SS-GM2-10 simulation.....	248
Figure 9-18 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM2-10 test run	249
Figure 9-19 Experimental response for the SS-GM3-10 simulation.....	250

Figure 9-20 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM3-10 test run.....	251
Figure 9-21 Experimental response for the NSS-GM4-10 simulation.....	252
Figure 9-22 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM4-10 test run	253
Figure 9-23 Experimental response for the NSS-GM5-10 simulation.....	254
Figure 9-24 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM5-10 test run	255
Figure 9-25 Illustration of the damaged component in the test system	257
Figure 9-26 SS-GM1-50: Comparison between numerical model and hybrid test results	259
Figure 9-27 SS-GM2-50: Comparison between numerical model and hybrid test results	260
Figure 9-28 SS-GM3-50: Comparison between numerical model and hybrid test results	261
Figure 9-29 NSS-GM4-50: Comparison between numerical model and hybrid test results.....	262
Figure 9-30 NSS-GM5-50: Comparison between numerical model and hybrid test results.....	263
Figure 9-31 Comparison between hybrid simulation and numerical results for peak displacement for 50% in 50 year intensity level ground motions for (a) Level 1, i.e. the location of the actuator, and (b) roof level.....	264
Figure 9-32 Comparison between hybrid simulation and numerical results for peak acceleration for 50% in 50 year intensity level ground motions for (a) Level 1, i.e. the location of the actuator, and (b) roof level.....	264
Figure 9-33 SS-GM1-10: Comparison between numerical model and hybrid test results	266
Figure 9-34 SS-GM2-10: Comparison between numerical model and hybrid test results	267
Figure 9-35 SS-GM3-10: Comparison between numerical model and hybrid test results	268
Figure 9-36 NSS-GM4-10: Comparison between numerical model and hybrid test results.....	269
Figure 9-37 NSS-GM5-10: Comparison between numerical model and hybrid test results.....	270
Figure 9-38 Comparison between hybrid simulation and numerical results for peak displacements for 10% in 50 year intensity level ground motions for (a) actuator node and (b) roof level	271
Figure 9-39 Comparison between hybrid simulation and numerical results for peak acceleration for 10% in 50 year intensity level ground motions for (a) actuator node and (b) roof level.....	271
Figure 10-1 Basic setup for proposed model updating algorithm showing the ‘control substructure’ (previously the ‘experimental substructure’), the ‘updated substructure’ (previously the ‘numerical substructure’) and the numerical counterparts	275
Figure 10-2 Illustration of the proposed model updating algorithm.....	277

Figure 10-3 Verification of the concept of running dynamic and displacement controlled analysis concurrently in a single OpenSees model: (a) OpenSees ‘multiframe’ model for concurrent dynamic and displacement controlled analysis; (b) Regular OpenSees model for dynamic analysis; (c) Regular OpenSees model for displacement controlled analysis; (d) Comparison of dynamic analysis from the ‘multiframe’ and regular models; displacement time history is identical; (e) Comparison of displacement controlled analysis from the ‘multiframe’ and regular models; pushover curves are identical.....	280
Figure 10-4 Frame properties and OpenSees Model for validation of parameter updating algorithm	283
Figure 10-5 Validation of parameter updating; value of the updated elastic modulus matches the true value from the control substructure	284
Figure 10-6 Frame properties and illustration of the OpenSees model to examine the impact of model updating	285
Figure 10-7 Comparison of the ‘true’ response from the control substructure and the modelled response from the updated substructure with no model updating	285
Figure 10-8 Comparison of the ‘true’ response from the control substructure and the modelled response from the updated substructure with model updating applied.....	286
Figure 10-9 Numerical counterparts for updating multiple parameters; the only variation between counterparts in each set is for the parameter being updated.....	287
Figure 10-10 Example of spurious updating of the yield strength whilst the control substructure is behaving elastically	288
Figure 10-11 Illustration of the method for identifying whether the control substructure has yielded or not	289
Figure 10-12 Hysteretic response of the control substructure stays inside the yield limits and hence is not deemed to have yielded	290
Figure 10-13 Hysteretic response of the control substructure stays inside the yield limits and hence is deemed to have yielded.....	290
Figure 10-14 Setup for numerical verification of multi-parameter nonlinear updating.....	291
Figure 10-15 Updated value of Elastic Modulus.....	292
Figure 10-16 Updated value of Yield Strength	292
Figure 10-17 Impact of continuing updating for the entire duration of the test on elastic modulus, which moves away from the ‘true’ value assigned to the control substructure after yield	293

Figure 10-18 Variation in weighting of various numerical counterparts with time during a simulation with model updating by Kwon and Kammula (2013), showing how weighting factors fluctuate initially then stabilize once a sufficient number of timesteps are included in the optimization	293
Figure 10-19 Setup for numerical verification of multiparameter updating for nonlinear analysis ...	294
Figure 10-20 Difference in response between the control substructure, Frame A in Figure 10-19, and the updated substructure, Frame F, when no updating is applied	295
Figure 10-21 Updated values of elastic modulus and yield strength; which are close to the ‘true’ values	296
Figure 10-22 Updated hysteresis and time history response of the two substructures, which are very similar, demonstrating the validity of the updating algorithm.....	296
Figure 10-23 Impact of model updating; comparison of responses of updated substructure and control substructure with and without parameter updating	297
Figure 10-24 Test SS-GM1-50: Updated Elastic Modulus	298
Figure 10-25 Test SS-GM2-50: Updated Elastic Modulus	299
Figure 10-26 Test SS-GM3-50: Updated Elastic Modulus	299
Figure 10-27 Test NSS-GM4-50: Updated Elastic Modulus.....	300
Figure 10-28 Test NSS-GM5-50: Updated Elastic Modulus.....	300
Figure 10-29 Updated elastic modulus from SS-GM1-10; note the experimental substructure is not deemed to have yielded	302
Figure 10-30 Updated elastic modulus from SS-GM2-10	302
Figure 10-31 Updated elastic modulus from SS-GM3-10	303
Figure 10-32 Updated elastic modulus from NSS-GM4-10.....	303
Figure 10-33 Updated elastic modulus from NSS-GM5-10.....	304
Figure 10-34 Updated yield strength from SS-GM2-10	305
Figure 10-35 Updated yield strength from SS-GM3-10	305
Figure 10-36 Updated yield strength from NSS-GM4-10.....	306
Figure 10-37 Updated yield strength from NSS-GM5-10.....	306
Figure 10-38 Updated initial brace camber from SS-GM2-10	307
Figure 10-39 Updated initial brace camber from SS-GM3-10	307
Figure 10-40 Updated initial brace camber from NSS-GM4-10.....	308
Figure 10-41 Updated initial brace camber from NSS-GM5-10.....	308
Figure 10-42 SS GM3-10; variation in numerical counterpart response with initial camber: $E = 126 \text{ kN/mm}^2$	309

Figure 10-43 SS GM3-10; variation in numerical counterpart response with initial camber: $E = 200 \text{ kN/mm}^2$	310
Figure 10-44 Illustration of the reason for greater nonlinear behaviour in a model with conventional stiffness compared to a model with reduced stiffness	310
Figure 11-1 Schematic of test frame used in BRACED shaking table tests. Both gusset connection types are shown in the one diagram; the gusset plate on the left is connected to the beam only, while on the right a more conventional connection to both the beam and column is shown. In reality, the same connection was employed for both gusset plates in each individual test.	315
Figure 11-2 (a) OpenSees model of the BRACED test frame and (b) model superimposed on schematic of test structure	317
Figure 11-3 Comparison of experimental and numerical response for test run S4-CA-G1 at the 50% in 50 year intensity level.....	317
Figure 11-4 Comparison of experimental and numerical response for test run S4-CA-G1 at the 2% in 50 year intensity level.....	318
Figure 11-5 (a) Comparison of experimentally recorded peak inter-storey drift with values obtained from OpenSees model; (b) Comparison of experimentally recorded peak floor acceleration with values obtained from OpenSees model	319
Figure 11-6 Comparison of experimental and numerical response using an <i>ESMF</i> of 0.9 for test run S4-CA-G1 at the 50% in 50 year intensity level.....	321
Figure 11-7 Comparison of experimental and numerical response using an <i>ESMF</i> of 0.8 for test run S4-CA-G1 at the 50% in 50 year intensity level.....	321
Figure 11-8 Comparison of experimental and numerical response using an <i>ESMF</i> of 0.7 for test run S4-CA-G1 at the 50% in 50 year intensity level.....	322
Figure 11-9 Comparison of experimental and numerical response using an <i>ESMF</i> of 0.7 for test run S4-CA-G1 at the 2% in 50 year intensity level.....	322
Figure 11-10 S4-CA-G1 at the 50% in 50 year intensity level.....	323
Figure 11-11 Impact of the <i>ESMF</i> on (a) median peak inter-storey drift and (b) median peak floor acceleration response	323
Figure 11-12 Elevation and Plan of case study building for comparison of performance assessments using experimental and numerical <i>EDPs</i> ; BRACED test frame used as the seismic resistant component in all directions.....	325
Figure 11-13 Comparison of the present value of expected losses, due to repair costs and downtime, for the case study building calculated using experimental <i>EDPs</i> , numerical <i>EDPs</i> and numerical <i>EDPs</i> with $\beta_m = 0.5$	326

Figure 11-14 Comparison of the present value of expected losses, due to repair costs and downtime, for the case study building calculated using the experimental <i>EDPs</i> and numerical <i>EPDs</i> from <i>ESMF</i> =0.7 and 0.8 models. Performance measures calculated with $\beta_m = 0$	328
Figure 11-15 Comparison of the present value of expected losses, due to repair costs and downtime, for the case study building calculated using the experimental <i>EDPs</i> and numerical <i>EPDs</i> from <i>ESMF</i> =0.7 and 0.8 models. Performance measures calculated with $\beta_m = 0.5$	328
Figure 11-16 Comparison between median peak <i>EDPs</i> for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using <i>ESMF</i> = 0.75	329
Figure 11-17 Comparison between median <i>EDPs</i> for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using <i>ESMF</i> = 0.75 at the MCE, DLE and SLE	329
Figure 11-18 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using <i>ESMF</i> = 0.75.....	330
Figure 11-19 Comparison between intensity specific losses for the regular and reduced stiffness models for the 5 storey frames designed using $q = 4$ at the Oakland site	331
Figure 11-20 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 2$ at the Oakland site for the regular structural model and using <i>ESMF</i> = 0.75.....	331
Figure 11-21 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 4$ at the Seattle site for the regular structural model and using <i>ESMF</i> = 0.75.....	332
Figure 11-22 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using <i>ESMF</i> = 0.75.....	332
Figure 11-23 Predicted and calculated δ_{mod} values for the Hybrid Simulations	335
Figure 11-24 Predicted and calculated θ_{mod} values for the Hybrid Simulations	336
Figure 11-25 (a) Illustration of the impact of β_3 on ϑ_{mod} . Range of β_3 values used in the particular study are marked by solid lines, extrapolated values are dashed lines (Plot developed for $T_1/T_m = 0.7$, $q_{eff} = 3$, $\gamma = 0.9$, $\beta_k = 0.0108$, but the basic trends shown hold for all values). (b) Illustration of the impact of β_3 on θ_{mod} , including the improved model which uses $ 1-\beta_3 $ as a variable instead of β_3 and appears to capture the trends noted in the two studies better.	337
Figure 11-26 Comparison between models developed for ϑ_{mod} (using $\beta_3 = 1.16$; the average value for the case study frames in the development of the models in Chapter 7).....	338
Figure 11-27 Comparison between models developed for (using $\beta_3 = 0.71$, the value for the hybrid simulation test structure)	338
Figure 11-28 Predicted and calculated θ_{mod} values for the Hybrid Simulations, with updated model for θ_{mod}	339

Figure 11-29 Experimentally recorded and modelled inter-storey drift profiles for the 50% in 50 year simulations	340
Figure 11-30 Experimentally recorded and modelled inter-storey drift profiles for the 10% in 50 year simulations	341
Figure 11-31 Experimentally recorded and modelled floor acceleration magnification profiles for the 50% in 50 year simulations	342
Figure 11-32 Experimentally recorded and modelled floor acceleration magnification profiles for the 10% in 50 year simulations	343

List of Tables

Table 2-1 Behaviour Factor, q , and Force Reduction Factor, R , for CBFs as recommended by Eurocode 8 and AISC	18
Table 3-1 Recommended initial brace camber to capture brace hysteretic behaviour in various studies	32
Table 4-1 Summary of recent studies examining earthquake induced lifetime losses for steel frame structures; showing how the proposed study fits in with and furthers existing work.....	68
Table 6-1 Details of case study frames; showing the influence of the behaviour factor, q , on the mass of steel required and fundamental structural period, T_1	95
Table 7-1 Design details and structural characteristics of frames analysed: first mode mass participation ratio (γ), minimum and maximum non-dimensional brace slenderness ($\bar{\lambda}$), column to brace stiffness ratio β_k (as defined by Karavasilis et al. (2007)) and β_3 , relative storey stiffness (as defined by Kumar et al. (2013))	136
Table 7-2 Details of beam, column and brace section sizes selected for frames	138
Table 7-3 Ground Motion Records used for incremental dynamic analysis	142
Table 7-4 Coefficients and 95% confidence intervals for regression equation for Global Modification Factor, δ_{mod}	152
Table 7-5 Coefficients and 95% confidence intervals for regression equation for Maximum Drift Modification Factor, ϑ_{mod}	153
Table 7-6 Coefficients and confidence intervals for the alternative model describing the variation of inter-storey drift over the frame height	167
Table 7-7 Coefficients and 95% confidence intervals for regression equation for Floor Acceleration Magnification, PFA/PGA	174

Table 7-8 Comparison between lifetime losses calculated by NLTHA and the proposed simplified method for three case study buildings	191
Table 8-1 Design of the experimental substructure	201
Table 8-2 Design details of test structure	201
Table 8-3 Impact of using elastic elements compared to distributed plasticity elements for modelling beams and columns of the test structure.....	208
Table 8-4 Influence of the number of iterations at each time step on the Peak Inter-storey Drift and Peak Floor Acceleration	210
Table 9-1 Details of selected ground motion records.....	231
Table 9-2 Comparison of peak roof drift and acceleration with SDOF system response for the 50% in 50 year hybrid simulations.....	236
Table 9-3 Comparison of peak roof drift and acceleration with SDOF system response for the 10% in 50 year hybrid simulations.....	245
Table 9-4 Summary of response of experimental substructure	256
Table 9-5 Comparison of <i>EDPs</i> from experimental and numerical analyses	264
Table 9-6 <i>EDPs</i> from numerical analyses expressed as a percentage of experimental value	264
Table 9-7 Comparison of <i>EDPs</i> from experimental and numerical analyses	271
Table 9-8 <i>EDPs</i> from numerical analyses expressed as a percentage of experimental value	271
Table 10-1 Values of the elastic modulus proposed by the updating algorithm and the <i>ESMF</i> , assuming a true value of $E = 210 \text{ kN/mm}^2$, for the 50% in 50 year hybrid simulations	301
Table 10-2 Values of the elastic modulus (when updating is stopped at the point of yield and when updating is allowed to continue throughout the test), yield strength and initial brace camber proposed by the updating algorithm and the <i>ESMF</i> , assuming a true value of $E = 210 \text{ kN/mm}^2$, for the 10% in 50 year hybrid simulations.....	309
Table 11-1 Details of the BRACED shake table test	315
Table 11-2 Comparison between experimentally recorded and numerically calculated drift (in %) .	319
Table 11-3 Comparison between experimentally recorded and numerically calculated acceleration (in <i>g</i>)	319
Table 11-4 Comparison between performance measures obtained using experimental and numerically calculated <i>EDPs</i>	325
Table 11-5 Comparison between performance measures obtained using experimental and numerically calculated <i>EDPs</i> using <i>ESMFs</i> of 0.7 and 0.8	327
Table 11-6 Effective behaviour factor (q_{eff}) used to calculate δ_{mod} and ϑ_{mod} for each intensity level	334

List of Symbols & Abbreviations

A	Cross sectional area
a_g	Design ground acceleration for Eurocode 8
C	Damping matrix
C_d	Seismic drift amplification factor
C_i	Fourier amplitude coefficient
CBF	Concentrically Braced Frame
CLE	Contingency Level Earthquake
CMS	Conditional Mean Spectrum
CS	Conditional Spectrum
DCH	Ductility Class High
DCL	Ductility Class Low
DCM	Ductility Class Medium
DLE	Design Level Earthquake
DM	Damage Measure
DV	Decision variable
E	Elastic modulus
$E(L)$	Expected losses
EAL	Expected Annual Losses
EAL	Eccentrically Braced Frame
$EC8$	Eurocode 8
EDP	Engineering Demand Parameter
$ESMF$	Elastic Stiffness Modification Factor
F_b	Eurocode 8 design base shear
f_i	Frequency
f_y	Yield strength
$GMSM$	Ground Motion Selection and Modification
g	Acceleration due to gravity
H	Building height
h_i	Height of storey i
i	Radius of gyration
I	Second moment of area
IDA	Incremental Dynamic Analysis
IM	Earthquake Intensity Measure

K	Stiffness matrix; effective length factor
K_T	Tangent stiffness matrix
L_{cr}	Critical buckling length
LIF	Loss Impact Factor
M	Mass matrix; Earthquake magnitude
$MAFE$	Mean annual frequency of exceedance
MCE	Maximum Credible Earthquake
$MDOF$	Multi-Degree of Freedom
MRF	Moment Resisting Frame
MSA	Multiple Stripe Analysis
m	Seismic mass
m_1	First mode effective mass
$N_{b,Rd}$	Design buckling capacity
N_{cr}	Critical elastic force for the relevant buckling mode
N_{Ed}	Axial design action
$N_{Ed,G}$	Axial design force resulting from gravity actions
$N_{Ed,E}$	Axial design force resulting from earthquake actions
$N_{pl,Rd}$	Axial plastic capacity
NLTHA	Nonlinear Time History Analysis
NS	Number of stories
OBE	Operation basis earthquake
$OCBF$	Ordinary concentrically braced frame
P	Probability; Applied load
$PACT$	Performance Assessment Calculation Tool
$PBEE$	Performance Based Earthquake Engineering
$PBSD$	Performance Based Seismic Design
PFA	Peak floor acceleration
PGA	Peak ground acceleration
$PISD$	Peak inter-storey drifts
PsD	Pseudo Dynamic test
$PSHA$	Probabilistic seismic hazard assessment
PV	Present value of lifetime losses
q	Eurocode 8 design behaviour factor
q_d	Eurocode 8 displacement amplification factor

q_{eff}	Effective behaviour factor
R	Force reduction factor; Earthquake source-to-site distance; Restoring force
R_E	Restoring force from experimental substructure
R_{ctrl}	Restoring force from the control substructure
R_N	Restoring force from numerical substructure
$R_{NC,i}$	Restoring force from numerical counterpart, i
RHS	Rectangular Hollow Section
$RISD$	Residual inter-storey drift
ROI	Return on Investment
$RTHT$	Real Time Hybrid Simulation
r	Discount factor
S	FEMA P-58 strength ratio
S_a	Spectral acceleration
S_d	Eurocode 8 design spectral acceleration
S_e	Eurocode 8 elastic spectral acceleration
S_s	ASCE short period spectral acceleration
S_1	ASCE 1 second spectral acceleration
$SCBF$	Special Concentrically Braced Frame
$SDOF$	Single Degree of Freedom
SHS	Square Hollow Section
SLE	Serviceability Level Earthquake
SLS	Serviceability Limit State
T_1	Fundamental structural period
T_c	Ground motion corner period
T_m	Ground motion mean period
T_α	Fundamental period of non-structural component
T^*	Conditioning Period for Conditional Spectrum
t	Time
tcl	Tool command language
UHS	Uniform Hazard Spectrum
UKB	UK beam section
UKC	UK column section
UKF	Unscented Kalman Filter
ULS	Ultimate Limit State

<i>USGS</i>	United States Geological Survey
u_i	Displacement at timestep i
$\dot{u}_i$	Displacement at timestep i
$\ddot{u}_i$	Acceleration at timestep i
$V_{bs,yield}$	Base shear at first yield
$V_{s,30}$	Shear wave velocity
w_i	Weighting factor for numerical counterpart, i
β_3	Relative storey stiffness
β_{aa}	FEMA P-58 Record-to-record variability in peak floor acceleration drift
$\beta_{a\Delta}$	FEMA P-58 Record-to-record variability in inter-storey drift
β_c	Uncertainty associated with building construction quality in FEMA P-58
β_{FA}	FEMA P-58 variation in peak floor acceleration
β_k	Column to brace stiffness
β_m	Modelling uncertainty parameter in FEMA P-58
β_q	Uncertainty associated with structural modelling completeness in FEMA P-58
β_{SD}	FEMA P-58 variation in inter-storey drift
Γ	Modal participation factor
Δ_1	Roof drift at first yield
Δ_{max}	Maximum roof drift
Δ_r	Residual inter-storey drift
Ω	Eurocode 8 brace overstrength
δ_{mod}	Global drift modification factor
ϑ_1	Inter-storey drift at first yield
ϑ_{max}	Maximum inter-storey drift
ϑ_{mod}	Maximum drift modification factor
ϕ^1	Fundamental mode shape
λ	Annual rate of occurrence of an earthquake event
$\bar{\lambda}$	Eurocode 8 non-dimensional slenderness
γ	First mode participation ratio
γ_{ov}	Eurocode 8 material overstrength factor

1 Introduction

1.1 Earthquakes and Structural Design

Earthquakes are caused by sudden movement of tectonic plates deep below the earth's surface. Energy released from these movements travels to the surface and causes ground movement and accelerations, primarily horizontally but also vertically. This presents a problem for structures in seismic regions as large inertia forces, and dynamic response oscillations, are induced. This extra lateral load and associated deformations must be considered in the design of structures in areas susceptible to seismic behaviour.

Steel frame buildings typically resist lateral loading through either moment resistant connections or extra bracing members. Moment resistant, or fixed, connections between beams and columns are designed to prevent rotation of the joint, allowing the structure to transfer lateral loads to the ground through frame action. Unlike fixed connections, the pinned connections used in braced frames do allow rotation, but the extra diagonally orientated bracing members provide lateral stability by resisting frame deformations through axial tension and compression. Braced frames are typically both stiffer and lighter than comparable moment resisting frames, hence providing an efficient method for resisting lateral load. Moment resisting frames are also generally have higher construction costs than equivalent braced frames, but remain popular as they provide architects and building owners with open-space floor plans (Uang and Bruneau, 2018).

Illustrations of both braced and moment resisting frames are given in Figure 1-1. It can be seen that braced frames consist of orthogonal beam and column and diagonal bracing members. Braced frames can be classified as either concentrically or eccentrically braced, with Eurocode 8 (CEN, 2004) providing precise definitions of each. In practice, this generally means that in Concentrically Braced Frames (CBFs) the axis of the bracing members is coincident with the intersection of the beam and column members, whereas in eccentrically braced frames (EBFs) this is not the case. Under dissipative design principles, the bracing members in CBFs are allowed to yield in tension and buckle in compression during infrequent, high intensity earthquakes. This inelastic brace deformation is complex and highly nonlinear, making it challenging to model accurately.

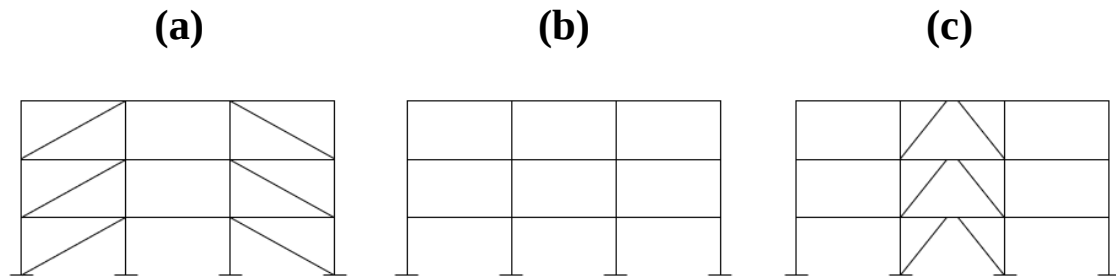


Figure 1-1 Examples of frame configuration: a) Concentrically Braced Frame (CBF) b) Moment Resisting Frame (MRF) c) Eccentrically Braced Frame (EBF)

The traditional aim of seismic design has been to prevent structural collapse. However, in the last 20 years, following the 1994 Northridge and 1995 Kobe earthquakes, when the unacceptably high levels of damage and economic loss were experienced, there has been a shift in focus in seismic engineering towards building performance. This has led to the development of the concepts of Performance Based Earthquake Engineering (PBEE) and Performance Based Seismic Design (PBSD), with the aim of designing and constructing buildings with full understanding of the risk to life, occupancy and economic value that may occur. This approach has been based on the ability to assess a building's design to determine the probability of experiencing different types of losses, considering the range of potential earthquakes that may affect the structure. To this end, the FEMA P-58 research project (FEMA, 2012a) developed previous work by the Pacific Earthquake Engineering Research (PEER) centre and proposed a methodology for the seismic performance assessment of individual buildings. This allows assessments to be conducted for earthquakes of a specified intensity or considering all potential earthquakes that may occur over the lifetime of a building along with the probability of their occurrence.

1.2 Statement of Problem

Braced frames are an efficient structural form for resisting lateral loads, providing high stiffness at relatively low cost. Recent earthquakes have shown that steel frame structures designed to modern codes are unlikely to collapse even under very severe seismic loads (Clifton et al., 2011). However, significant damage, and subsequent losses, can be incurred by both structural and particularly non-structural elements (Miranda et al., 2012). Therefore, achieving a desired seismic performance requires the coordination of structural and non-structural response (Del Gobbo et al., 2018). This is challenging for CBFs where the inherent lateral stiffness can lead to high response acceleration demands. Therefore, as design becomes more focussed on performance, it is necessary to develop a full understanding of the type of losses suffered by CBFs and if and how these can be controlled at the design stage.

The application of performance assessment is predicated on being able to accurately predict the response of CBF structures to earthquakes. At present, even with advanced computational methods, this is a challenging and time consuming activity (Malaga-Chuquitaype and Elghazouli, 2012). Hence there is still a requirement to improve and validate the capability of numerical models through experimental testing. Furthermore, there is a need to develop efficient methods to predict seismic response if performance based design is to be adopted beyond academia.

1.3 Objectives

Broadly speaking, the aim of this research project is to use both numerical and experimental simulation to investigate the seismic performance of CBFs and to examine whether improvements can be made to performance assessment procedures for CBFs. A number of research objectives are set out to allow these aims to be fulfilled:

- Various studies (Terzic et al., 2014, Hwang and Lignos, 2017a, Del Gobbo et al., 2018) have employed the FEMA P-58 performance assessment methodology to examine the performance of CBF structures. This study will also use this methodology to assess the potential for lifetime losses to be minimised for CBFs designed to Eurocode 8 through design alterations.
- A major problem with performance assessment is that its application has been largely limited to academic research (Günay and Mosalam, 2013). Practical application is hindered by the considerable time and expertise required to carry out the necessary response analyses. This study shall attempt to address this barrier through the development of alternative efficient methods to predict key response parameters for performance assessment specifically for CBFs designed to Eurocode 8.
- Performance assessment methodologies are dependent on the ability of structural models to accurately predict response parameters of interest. A hybrid simulation experimental testing programme, including online model updating, will be implemented in order to evaluate and improve the accuracy of numerical models employed to analyse CBFs.

1.4 Organisation of Thesis

This section describes the layout of this thesis. A summary of each chapter is provided so that the rationale behind the structure of the document as a whole is explained.

Chapter 2

Chapter 2 introduces some of the basic concepts of CBF behaviour and design, such as the behaviour of bracing members and gusset plate connections. Particular focus is placed on design to Eurocode 8 and the specific regulations imposed pertaining to CBF structures.

Chapter 3

Chapter 3 provides background on the numerical analysis of structures subjected to seismic loads. In particular, the use of the OpenSees computational platform for nonlinear time history analysis is discussed, with a review of previous research and modelling techniques typically used in the analysis of CBF structures.

Chapter 4

Chapter 4 introduces the concepts of PBEE and performance assessment, with a focus on the FEMA P-58 methodology. Through a review of the relevant literature, a number of research questions that need to be answered in order to improve the ability to apply Performance Based Design techniques for CBFs are identified.

Chapter 5

Chapter 5 discusses experimental techniques for earthquake structural testing, particularly hybrid simulation. The concept of model updating is also introduced. A review of previous research examining various aspects of the development of the test method is presented. In light of this, the ways in which hybrid simulation and model updating can be used to improve numerical modelling capabilities are outlined.

Chapter 6

Chapter 6 examines the lifetime performance of CBFs designed to EC8 using OpenSees nonlinear time history response simulations. The FEMA P-58 performance assessment methodology is employed to examine multiple CBF structures designed using various behaviour factors such that a wide range of potential structural designs are investigated. Thus, the impact of decisions made by the designer on the lifetime performance of CBFs is investigated. This allows a comprehensive picture of CBF lifetime performance, and the ability of the designer to influence this, to be developed.

Chapter 7

Chapter 7 looks at the development of performance assessment techniques that avoid the need for computationally advanced and time-consuming earthquake response analyses. Predictive models for key response parameters, namely peak inter-storey drift and peak floor acceleration, are developed. Performance measures calculated based on these predicted response parameters are compared with those calculated based on more conventional time history analyses.

Chapter 8

Chapter 8 discusses the setup of substructured hybrid simulation for a CBF structure. Details are provided about both the design of the test structure and the experimental control system employed. Various technical challenges encountered in the implementation of hybrid simulation, including incorporating the experimental substructure in the numerical model and local control issues, are addressed.

Chapter 9

Chapter 9 presents results from a set of hybrid simulations performed for various earthquake scenarios, with the primary focus on peak drift and acceleration response, which are the key response parameters for performance assessment. These results are examined in detail and responses obtained are compared to those calculated using the numerical modelling techniques previously employed in Chapter 6.

Chapter 10

Chapter 10 details the development and verification of a model updating algorithm, which is applied offline to the hybrid simulation results presented in Chapter 9 to systematically investigate the observed differences between experimental and corresponding numerical results. This allows a series of recommendations to be presented for alternative modelling of CBFs, which ultimately allow for improved performance assessment procedures.

Chapter 11

Chapter 11 re-examines the conclusions of earlier chapters using experimental data. The modelling recommendations emerging for the offline model updating in Chapter 10 are further assessed using the results of experimental tests from a previous research programme. The ability of FEMA P-58 to account for differences between *EDPs* recorded from the experiments

and from NLTHA on performance metric is also assessed. Performance assessments that had previously been carried out using modelling assumptions recommended elsewhere in literature are repeated using improved models that incorporate recommendations arising from the examination of experimental results. Finally, the ability of the regression equations developed to predict the peak drift and acceleration response observed in the hybrid simulation testing programme is assessed.

Chapter 12

Chapter 12 summarises the work completed within this research project. Following this, a number of key findings and recommendations for future work are presented.

2 Seismic Response and Design of CBFs

2.1 Introduction

CBF structures use diagonal bracing members to resist lateral loads through a vertical concentric truss system. These bracing members are typically connected to beams and columns using gusset plate connections. CBFs are generally seen as an efficient structural form, providing high stiffness at a relatively low cost compared to other seismic resistant systems.

The purpose of this chapter is to provide background on the behaviour of CBFs during earthquakes. It begins by introducing some basic concepts, such as dissipative design and ductility. The design of CBFs to resist seismic load is then addressed. Particular attention is placed on design to Eurocode 8 and the particular rules imposed by the code for CBFs.

2.2 CBF Response to Earthquake Loading

2.2.1 Global Frame Behaviour

CBFs resist horizontal seismic loads through a combination of tension and compression in alternative bracing members. This concept is illustrated in Figure 2-1.

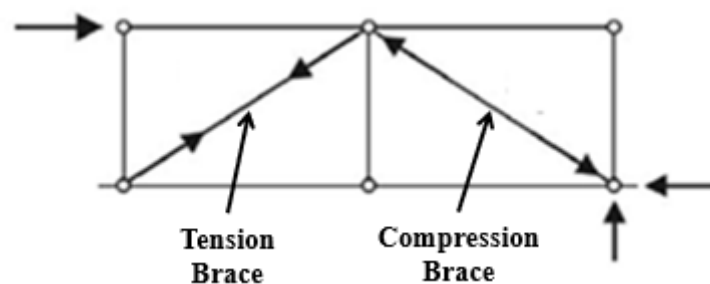


Figure 2-1 CBF behaviour under lateral load (adapted from Tremblay (2002))

In conventional earthquake engineering practice, designing structures to behave elastically under large seismic loads is generally impractical and uneconomic. For building frames subjected to high intensity earthquakes, elastic behaviour requires significant forces to be resisted, necessitating the use of large, and expensive, structural members and connections. However, more efficient design can be achieved by allowing elements of the structure to yield under high-intensity, infrequent ground motions.

When yielding occurs energy is dissipated through the inelastic deformation of certain structural members; hence this is termed dissipative design and the parts of the structure where this occurs are referred to as dissipative zones. In CBFs the main dissipative zones are the bracing

members, which are allowed to yield in tension and buckle in compression. This is illustrated in Figure 2-2.

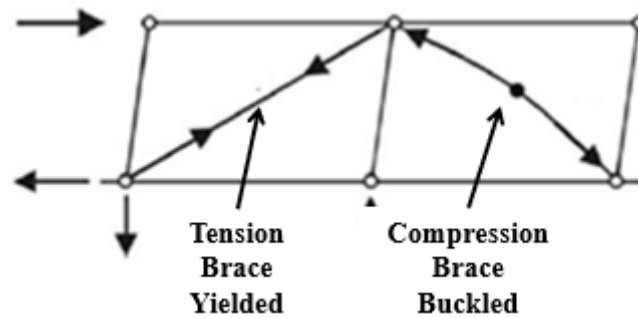


Figure 2-2 Post-elastic CBF behaviour under lateral load (adapted from Tremblay (2002))

Given this, the bracing members must be sufficiently ductile to accommodate this inelastic behaviour. Ductility is defined as the ability of a structure or member to withstand deformations beyond its yield point without fracture. The maximum ductility that a structure will experience during an earthquake is termed the ductility demand, and the maximum ductility the structure can sustain without fracture is referred to as the ductility capacity. In design, ductility capacity should exceed ductility demand, a principle known as capacity design (Williams, 2009).

In Eurocode 8, dissipative design for regular structures is achieved by performing conventional elastic analysis, with the design actions reduced by a single global behaviour factor, q , which can be expressed as:

$$q = \frac{F_{el}}{F_y} \quad 2.1$$

Where F_{el} is the peak seismic force that would develop in a SDOF system behaving linear elastically and F_y is the yield strength of this SDOF system. This concept is illustrated in a simplified manner in Figure 2-3.

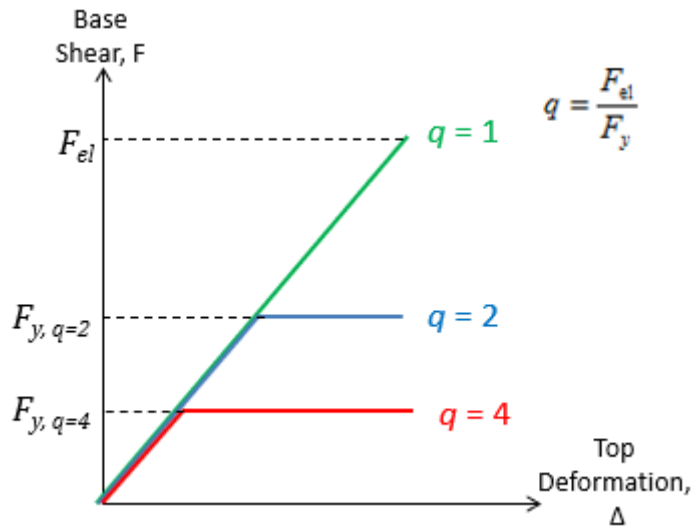


Figure 2-3 Simplified representation of inelastic and elastic behaviour; it can be seen that reduced forces are experienced when the structure is allowed to yield as opposed to remaining elastic

2.2.2 CBF Response to Past Earthquakes

Prior to the 1994 Northridge earthquake in California and the 1995 Kobe earthquake in Japan, moment resisting frames were the most commonly employed method to resist seismic loads in steel frame buildings. However, many MRFs performed poorly in these earthquakes; with brittle fracture at connections a particular problem. This has resulted in an increase in the use of bracing systems to provide seismic resistance, with the percentage of CBFs in new steel construction in California rising from approximately 10% to about 40% in the decade following Northridge (Wijesundara, 2009).

The primary aim of most contemporary seismic design codes is to ensure life safety under severe earthquakes. In recent large earthquakes, such as the 2010 Maule earthquake in Chile (Saatcioglu et al., 2013) and the 2011 Tohoku-Oki earthquake in Japan (Okazaki et al., 2013), the performance of steel structures designed in accordance with modern codes has generally been adequate from a life safety viewpoint. Similarly, steel structures generally performed in 2011 earthquake in Christchurch, New Zealand and many were functional shortly afterwards (Clifton et al., 2011). The performance of most reinforced concrete structures was also generally adequate in terms of life safety. However, the level of structural damage incurred and the difficulty of repair has resulted in many of these buildings being demolished since the earthquake. Consequently, steel buildings have become more popular in the post-earthquake rebuild (MacRae et al., 2015).

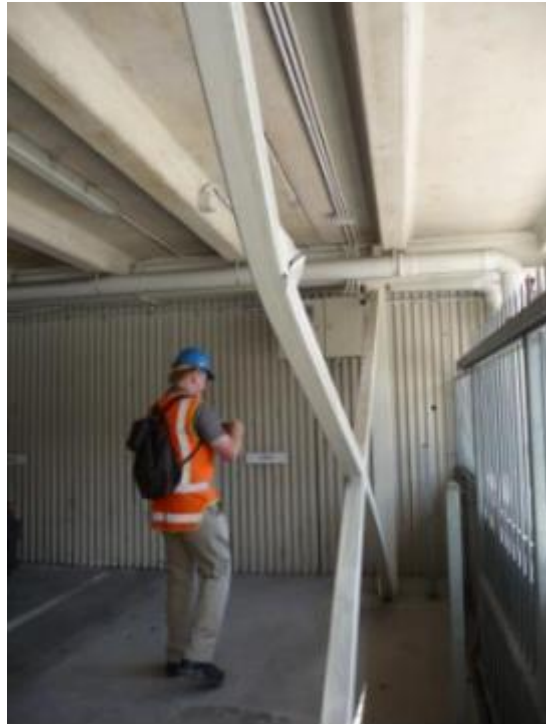


Figure 2-4 Example of a buckled brace in a CBF structure after the 2010 Christchurch earthquake (Clifton et al., 2011)

2.2.3 Brace Behaviour

As horizontal load is applied, the bracing members cycle between tension and compression. In modern CBFs, the bracing members are designed to yield or buckle under large lateral loads induced by infrequent earthquakes. Energy is dissipated through this hysteretic post-elastic behaviour.

A large number of studies have looked at the response of bracing members under cyclic loading. A detailed summary of these and many of the key findings is provided by Hunt (2013).

Figure 2-5 (a) shows a typical axial load versus displacement response of a cyclically loaded member for one compression-tension post elastic load cycle. Starting at point O, the brace is loaded in axial compression until buckling occurs at point A. This is followed by lateral deflection and the formation of a plastic hinge at mid-length, with a reduction in axial force capacity between points A and B. Load reversal and elastic recovery begins at point B, with a reduction in lateral deflections. Tensile yield occurs at point D, with uniaxial plastic elongation until the load direction is reversed and elastic unloading begins at E. At the end of one loading cycle, point F, the brace is straight but with some residual elongation. Broadly speaking, this pattern, with gradual degradation of the compression resistance and yielding in tension occurring at increasing axial deformations, is repeated over further loading cycles, as illustrated in Figure 2-5 (b) (Goggins et al., 2006).

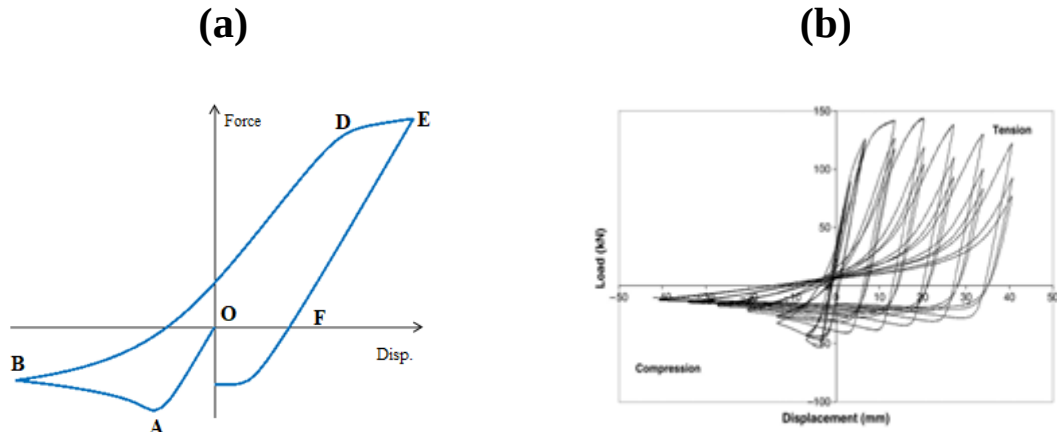


Figure 2-5 Typical post-elastic response of a bracing member under cyclic axial load: a) A single loading cycle and b) multiple load cycles (Goggins et al., 2006)

Under repeated loading, brace fracture can occur due to low cycle fatigue. Fracture typically occurs at the plastic hinge location, i.e. the mid-point, in tension after local buckling has occurred in compression.

The ability of a bracing to resist fracture is dependent on its ductility capacity. Brace displacement ductility, μ_{Δ} , can be defined as:

$$\mu_{\Delta} = \frac{\Delta_u}{\Delta_y} \quad 2.2$$

Where Δ_u is the ultimate deformations at fracture and Δ_y refers to the deformation at first yield. Several studies (Tremblay, 2002, Goggins et al., 2006, Nip et al., 2010) have proposed equations based on experimental data to predict ductility capacity of bracing members. These expressions generally show that brace ductility capacity is increases with slenderness and decreases with the width to thickness ratio and yield strength.

2.2.4 Gusset Plates

Gusset plates are the predominant method used to connect brace members to the rest of the structural frame. Gusset plates are typically designed so that dissipative behaviour is limited to the bracing members. This is done through capacity design procedures by ensuring that the yield strength of the gusset plate in various potential failure modes exceeds the axial demand of the brace. The welds or bolts used to attach the gusset plate to the beam and column and the brace to the gusset plate, as well as the plate itself, must also be capacity designed. The yield and buckling strengths of the gusset plate are determined by modelling the gusset plate as a strut (Thornton, 1991) with the cross sectional area determined using the Whitmore width (Whitmore, 1952), defined as a 30° projected angle from the start to the end of the brace-gusset

joint, as illustrated in Figure 2-6 (a). It should be noted that the European seismic design provisions, in contrast to their US counterparts, only deal with gusset plate design in a conceptual manner leaving designers to adopt details from current literature (Elghazouli, 2016).

Generally, gusset plates are designed as rotationally unrestrained in the out-of-plane direction to accommodate end rotation of post-buckled bracing members, as show in Figure 2-6 (b). When a brace buckles out-of-plane, it attempts to form a hinge, or yield line, in the gusset plate, which in conventional design is typically considered as perpendicular to the brace axis. In order to allow this behaviour, a free length, typically between 2 and 4 time the thickness of the plate, is incorporated in the gusset plate perpendicular to the end of the brace, as illustrated in Figure 2-6 (c). This design method is known as the Standard Linear Clearance (SLC) model.

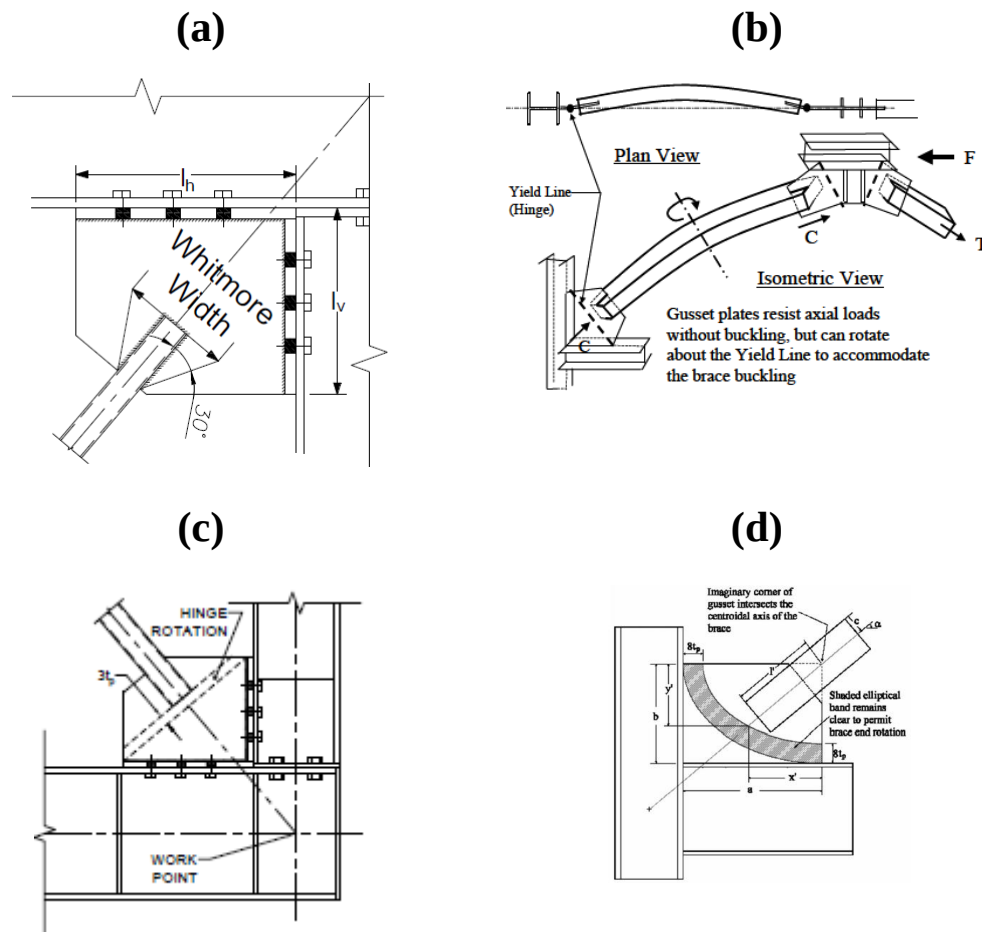


Figure 2-6 Illustrations of gusset plate behaviour and designs; a) Whitmore width as used in gusset plate design (Hunt, 2013), b) Illustration of out-of-plane brace buckling and hinge formation in gusset plates (Cochran, 2003), c) Typically employed Standard Linear Clearance model (Hunt, 2013) and d) Innovative elliptical clearance design (Lehman et al., 2008)

Some more innovative designs have attempted to improve performance by changing the position and shape of the yield line and allowing yielding to occur in the gusset plate as well as in the brace, for example Lehman et al. (2008) and Roeder et al. (2011) as shown in Figure 2-6 (d).

Again, as with the section discussing brace behaviour, the preceding section only very briefly summarises gusset plate behaviour and design. For more detail the reader is referred to Cochran (2003), Lehman et al. (2008) or Hunt (2013).

2.3 CBF Design to Eurocode 8

In Europe, Eurocode 8 (CEN, 2004) is the design standard generally used to design structures to resist seismic action. Some of the key underlying principles are discussed in the following section.

Eurocode 8 implements a force based design procedure. Broadly speaking the steps in the design process can be broken down into quantifying of the seismic hazard, estimating the forces the structure is likely to experience as a result of this seismic hazard and selecting appropriate structural members to resist these forces. Thus the following discussion begins with an introduction to calculating the seismic hazard and consequent design actions for a structure, before discussing the concepts of capacity design and dissipative behaviour. Some of the regulations pertinent to CBFs imposed by Eurocode 8 are then introduced with brief discussion provided on the background theory behind some of these rules.

2.3.1 Seismic Hazard and Earthquake Actions

The first step in seismic design is to quantify the earthquakes likely to be experienced by a structure during its lifetime. This involves calculating an intensity measure, such as the Peak Ground Acceleration (PGA), that captures probable seismic events at the building's site. This is done through Probabilistic Seismic Hazard Analysis (PSHA) (Baker, 2008). The earthquake intensity measure used to define the seismic hazard in design to Eurocode 8 is the PGA on type A (stiff) ground with a 475 year return period, which corresponds to a 10% chance of exceedance in 50 years. This is termed design ground acceleration a_g . The value of a_g for a particular site is typically obtained from seismic zonation maps which are produced at a national level. Figure 2-7 (a) (Solomos et al., 2008) shows an example of one of these for Italy, while Figure 2-7(b) shows the design level seismic hazard throughout Europe (Giardini et al., 2013).

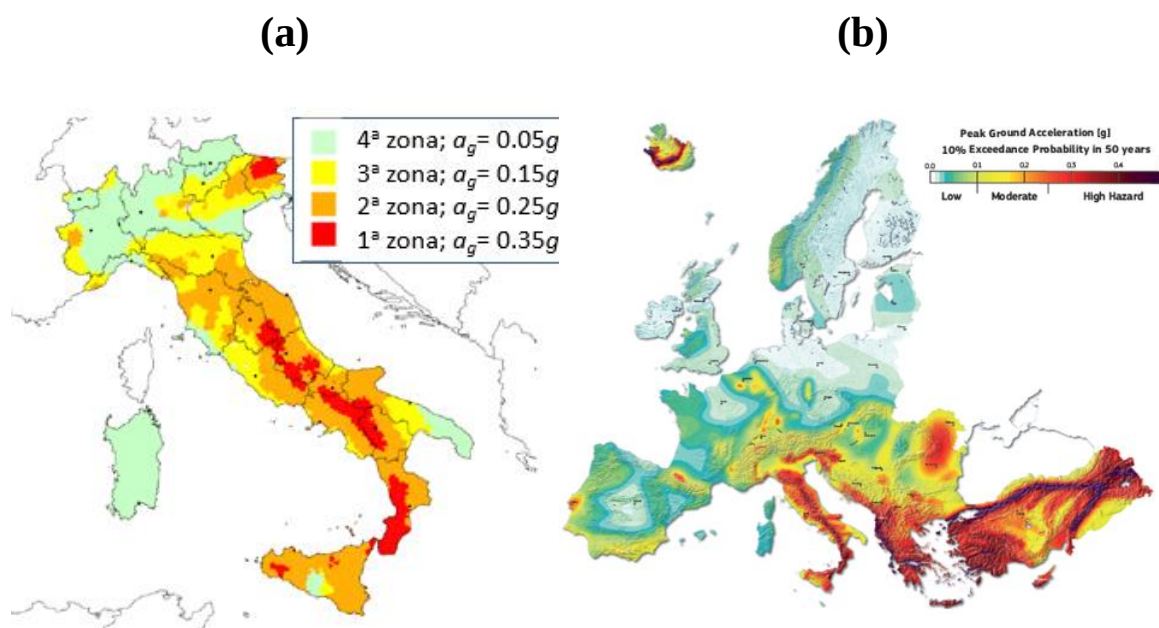


Figure 2-7 (a) Seismic hazard zonation map of Italy (Solomos et al., 2008) and (b) design level seismic hazard in Europe (Giardini et al., 2013)

2.3.2 Design Seismic Actions

Once the seismic hazard for a site is established, the next step in design for seismic resistance is to convert this into forces that the structure will be required to withstand. This is done using a design spectrum. The response spectrum of an earthquake relates the maximum response (displacement, velocity or acceleration) of a Single Degree of Freedom (SDOF) structure to the earthquake with the fundamental period of the structure. Examples of acceleration response spectra of a typical earthquake records are given in Figure 2-8(a). Broadly speaking, most acceleration response spectra have this general shape. In design codes such as Eurocode 8, generic acceleration response spectra that envelope all the likely levels of response are typically used to obtain forces for use in design. Eurocode 8 provides procedures for developing these spectra based on a design ground acceleration value, a_g , the ground conditions at the site being considered and the assumed level of damping. Two types of spectra, Type I and Type II are specified in EC8, with Type I spectra to be used when the earthquakes that contribute most to the seismic hazard at the site are calculated to have a magnitude greater than 5.5. The spectrum initially developed is known as the Elastic Response Spectrum, $S_e(T)$. Examples of such spectra for different ground conditions assuming 5% damping are given in Figure 2-8(b). From this it can be appreciated that the value of the elastic spectral acceleration for a structure is a function of the design ground acceleration, structural period and soil type as well as the level of damping anticipated.

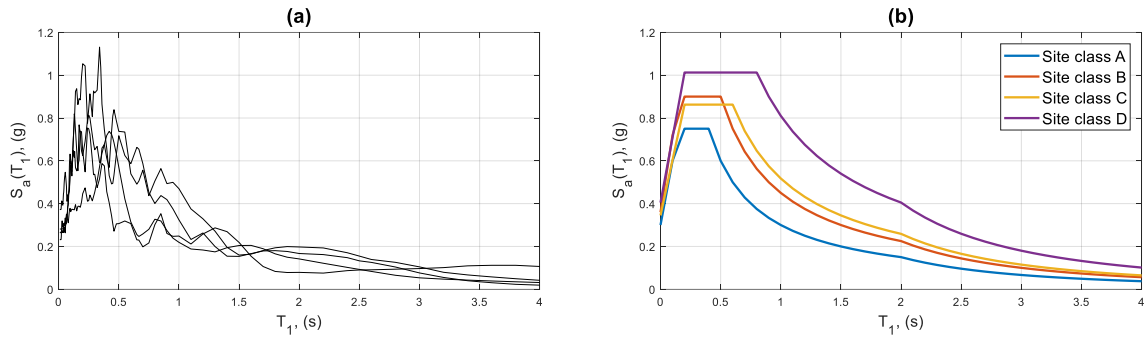


Figure 2-8 Examples of Response Spectra: a) Response spectra from four different ground motion records showing typical spectral shape; b) Eurocode 8 Type I elastic spectra for different ground conditions calculated using a design ground acceleration value of $0.3g$ assuming 5% damping. Ground conditions are defined by the average shear wave velocity and typically soil becomes softer moving from A (typically rock) to D (typically cohesion-less soil).

For regular structures, satisfying a number of criteria specified in the code, a simplified equivalent static method of analysis may be applied to determine the design forces. Firstly, the elastic spectrum is converted to a design acceleration spectrum, $S_d(T)$. In most of cases, this is done by dividing the elastic response spectrum by a behaviour factor, q , although for structures with very low fundamental periods a more complex relation is used. This reduction, illustrated in Figure 2-9, accounts for dissipative behaviour and subsequently the reduced force required to be resisted by the structure.

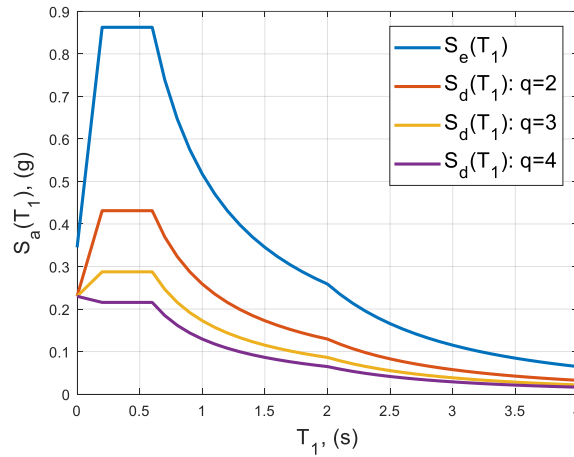


Figure 2-9 Eurocode 8 elastic spectrum, $S_e(T_1)$, and design spectra, $S_d(T_1)$, calculated using $q=2$, $q=3$ and $q=4$ for a design ground acceleration of $a_g = 0.3g$ and site class C ground conditions

The seismic base shear force to be resisted by the structure, F_b , is then obtained from the product of the design spectral acceleration at the fundamental period of the structure $S_d(T)$, the seismic mass, m , and a factor representing first mode participation, λ :

$$F_b = S_d(T) \times m \times \lambda \quad 2.3$$

For regular structures, the base shear is distributed throughout the height of the structure as equivalent static forces at each floor level. These forces are calculated as the product of the mass and height from the base using the equation:

$$F_i = F_b \frac{z_i m_i}{\sum z_j m_j} \quad 2.4$$

Where F_i is the lateral force at floor level i , and z_i and z_j and m_i and m_j are the respective heights and seismic masses at each floor level. This results in an inverted triangular distribution over the height if the masses are equal at all floors, as shown in Figure 2-10. Linear elastic static analysis, and subsequently design, is then carried out using this loading configuration.

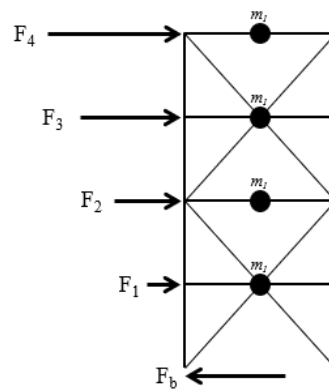


Figure 2-10 Distribution of seismic base shear as horizontal loads at each floor level for design of regular structures to EC8 using the equivalent static force design method

For irregular structures more thorough analysis procedures must be performed. These include, in order of increasing complexity, modal analysis, nonlinear static analysis and nonlinear time history analysis. Details of some of these analysis procedures are given in Chapter 3.

The US provisions (ASCE/SEI, 2010) similarly define base shear as the product of seismic mass and spectral acceleration. However, the spectrum is calculated based on site specific values of spectral acceleration at 0.2s and 1s, known as the S_s and S_1 , rather than using a design ground acceleration value. A force reduction factor, R , is incorporated to account for nonlinear behaviour, performing a similar role to q in Eurocode 8. As with Eurocode 8, equivalent lateral force static analysis is allowed for regular structures with more thorough analysis procedures required in more complex cases.

2.3.3 Dissipative Design & Behaviour Factor

As mentioned in the previous section, in Eurocode 8, for all except very stiff structures, the elastic spectral acceleration, $S_e(T)$, is divided by q to obtain the design spectral acceleration, $S_d(T)$. This has the effect of limiting the peak force the structure is required to sustain, as

happens when a structure yields. This is illustrated graphically in Figure 2-3. Hence, using the behaviour factor, q , allows explicit inelastic analysis to be avoided whilst still accounting for nonlinear behaviour.

The code provides recommended maximum values of q for use in design, as shown in Table 2-1. These are dependent on the ductility capacity of the bracing members. For steel structures, Eurocode 8 recognises three ductility classes, low (DCL), medium (DCM) and high (DCH), which are determined by the cross sectional classification, as defined by Eurocode 3 (CEN, 2005), of the dissipative members. For diagonally braced CBFs the upper limit of the behaviour factor is given as $q=4$ for both DCM and DCH, while for V or inverted-V bracing configurations the upper limits are 2 and 2.5 for DCM and DCH respectively. DCL is used in regions of low seismicity where dissipative design procedures are not applied, however a value of $q=1.5$ may be used accounting for the presence for a minimal inherent level of overstrength and ductility.

In the US, the force reduction factor, R , plays a similar role to the Eurocode 8 behaviour factor. The AISC Seismic Provisions (AISC, 2010) divide CBFs into two categories; Ordinary (OCBF) and Special (SCBF) concentrically braced frames. OCBFs have low ductility demand and are designed for large base shears and limited inelastic deformation. In contrast, SCBFs are designed with significant elastic deformation capacity.

Table 2-1 Behaviour Factor, q , and Force Reduction Factor, R , for CBFs as recommended by Eurocode 8 and AISC

Frame Type	Eurocode 8: q			AISC: R	
	DCL	DCM	DCH	OCBF	SCBF
Diagonal Bracing	1.5	4.0	4.0	3.75	6
V- Bracing	1.5	2.0	2.5	3.75	6

2.3.4 Slenderness, Overstrength & Other Design Criteria

Several other design rules and restrictions are imposed by Eurocode 8 for CBFs. These are discussed in the following sections.

2.3.4.1 Brace Design Resistance

In Eurocode 8, for non-V type CBFs, the entire lateral resistance is assumed to be provided by the tension diagonals only and brace design strength is based on the brace plastic capacity in tension. It is assumed that the compression diagonals will buckle and provide no lateral resistance. This is in contrast to the US provisions (AISC, 2010), where brace design strength is based on the buckling capacity in compression. This means that CBFs designed according to US provisions tend to have stockier, less slender bracing members than their European equivalents

as design is controlled by the buckling capacity rather than the tension capacity of the braces. Both of these design methods are simplifications necessary to allow the use of linear analysis tools and can introduce a significant level of overstrength in CBF designs (Elghazouli, 2010).

For regular structures, static analysis is performed, generally assuming pinned connections, and the design force to be resisted by each brace, N_{Ed} , is calculated. The bracing members are then selected such that the plastic resistance, $N_{Pl,Rd}$, calculated in accordance with Eurocode 3, exceeds the design force, i.e. $N_{Pl,Rd} > N_{Ed}$.

2.3.4.2 Brace Slenderness

Numerous studies (for example Tremblay (2002)) have shown that brace slenderness has a major influence on brace performance, both in terms of buckling resistance and ductility capacity. Compression resistance and energy dissipation capacity is larger for stockier, less slender braces, but they experience greater reductions in compressive strength in the post buckling range. More slender braces have higher ductility but buckle under lower compressive loads (Goggins et al., 2006). Member slenderness is expressed in the European codes by its non-dimensional value, $\bar{\lambda}$, defined as:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i} \frac{1}{\lambda_1} \quad 2.5$$

$$\lambda_1 = \pi \sqrt{\frac{E}{f_y}} \quad 2.6$$

Where A is the section area, f_y is the yield strength, N_{cr} is the elastic critical force for the relevant buckling mode, L_{cr} is the relevant buckling length, i is the relevant radius of gyration and E is the modulus of elasticity.

Eurocode 8 imposes limits on brace slenderness for buildings over 2 storeys tall. In order to prevent elastic buckling, which is undesirable as energy is not dissipated, the maximum allowable $\bar{\lambda}$ value of 2 is prescribed by Eurocode 8. For frames using X-diagonal braces a lower $\bar{\lambda}$ limit of 1.3 is also imposed. The purpose of this lower bound is to avoid overloading the columns in the pre-buckling stage.

2.3.4.3 Storey Mechanisms and Brace Overstrength Limitations

Concentrically braced frames are susceptible to the development of a weak storey mechanism, where the inter-storey drift and ductility demand is concentrated at a single floor level. Once yielding occurs in braces at a particular level, the frame resistance is mainly determined by the

capacity of this storey. Compared with moment resisting frames, post-yielding redistribution is typically very limited or non-existent in CBFs (Elghazouli, 2005). This behaviour characteristic can often lead to a concentration of inelasticity at one level in the building, leading to the formation of a storey mechanism.

Eurocode 8 attempts to address the problem by limiting the difference between brace overstrength at different levels. Brace overstrength, Ω_i , is defined for each storey as the ratio of plastic axial capacity, $N_{pl,Rd,i}$, to design axial force, $N_{Ed,i}$:

$$\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}} \quad 2.7$$

The difference between the maximum and minimum overstrength cannot exceed 25%. This is to ensure plastic demand is evenly distributed along the building height and reduce the risk of a soft storey developing. However, this requirement in isolation does not eliminate the problem and also imposes additional design effort and practical difficulties in the selection of brace sizes (Elghazouli, 2010). Indeed, often the combination of the overstrength and brace slenderness limitations can lead to inefficient design. At the upper floor levels in a structure the seismic forces are often relatively small and brace design is often governed by the slenderness limitations, resulting in large overstrength values. These overstrength values can in turn dictate the design for the lower portion of the structure, resulting in increased section sizes throughout. The inefficiencies arising from these restrictions have been discussed in detail elsewhere, with alternative methods proposed to improve design (for example Brandonisio et al. (2012)).

2.3.4.4 Design of Non-Dissipative Elements

To ensure yielding occurs in the braces before yielding or buckling of other components, the design axial load, $N_{Ed,m}$, in elements other than braces is determined by the expression:

$$N_{Ed,m} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \quad 2.8$$

Where $N_{Ed,G}$ and $N_{Ed,E}$ are the axial forces due to gravity loads and lateral seismic loads respectively, γ_{ov} is the overstrength factor used in design, typically 1.25, and Ω is the minimum brace overstrength over all the braces.

2.3.4.5 Drift Limitations

When capacity design is complete, inter-storey drifts under seismic actions are estimated. This is done using the product of the displacement obtained from an elastic analysis and a displacement amplification factor, q_d , which is the same as the behaviour factor, q , for steel structures. The use of the same value here means that in effect the code effectively employs the

well-known equal displacements rule (Veletsos and Newmark, 1960) to estimate post-yield displacement. The design must firstly be checked for susceptibility to second order effects, before a damage limitation check is performed at a reduced seismic intensity. Limitations on the maximum inter-storey drift are set, typically at 0.5%, 0.75% or 1% of storey height, depending on the nature of the non-structural elements. However, because of the relatively high lateral stiffness of CBFs, these design checks rarely lead to alterations (Elghazouli, 2008). In contrast, for moment resisting frames these drift limitations can often control the design.

2.4 Conclusion

The chapter introduces some of the key concepts of CBF seismic response and CBF design. Under infrequent, high intensity earthquakes most structures are designed to behave inelastically and dissipate energy. In CBFs, this is achieved by allowing the bracing members to yield in tension and buckle in compression.

The design of CBFs to Eurocode 8 is then discussed. Firstly, this involves establishing the seismic forces, which are obtained from a design spectrum. This is typically calculated using a behaviour factor, q , which reduces the design forces to account for nonlinear behaviour. For CBFs designed to EC8, the bracing members are selected to resist these forces through tension only. The code imposes limitations on the brace non-dimensional slenderness and overstrength which can often control design. In contrast, the drift limitations imposed are rarely influential in the design of CBFs.

3 Numerical Analysis of Seismic Response

3.1 Introduction

This Chapter discusses the numerical analysis of structures subjected to seismic loading. Firstly, a range of potential structural analysis methods of varying complexity are discussed. The focus then moves to the OpenSees computational platform, which is used extensively throughout this research. The layout of the OpenSees framework is discussed, before a review of modelling techniques used for CBFs is presented.

3.2 Numerical Analysis

There are various numerical methods of analysing a structure subjected to seismic loading. Broadly speaking, these can be broken down into static or dynamic methods and linear or nonlinear methods. Nonlinear numerical models account for the material and geometric nonlinearities that occur when structures are subjected to large earthquakes. They are more computationally expensive than linear analyses and often require the use of iterative techniques to reach converged solutions.

As mentioned in Chapter 2, under large earthquakes most structures in seismic zones are designed to behave nonlinearly. The simplest method of including nonlinear behaviour in analysis is to employ a global behaviour factor to modify the design spectrum and then perform linear analysis, as is done in the Eurocode 8 equivalent static force design method previously discussed. However, improved analysis can be performed by obtaining an accurate estimate of distributed yielding within the structure from nonlinear models rather than assuming an overall uniform ductility.

3.2.1 Nonlinear Static Analysis

In nonlinear static analysis, also known as pushover analysis, appropriate lateral load patterns are applied to the numerical model and their amplitude increased in a stepwise fashion. Static analysis is performed at each step until a collapse mechanism is developed in the structure. This allows a pushover curve, where base shear is plotted against top displacement, to be produced and gives the engineer a feel for the nonlinear behaviour of the structure under lateral load. However, pushover analysis has no theoretical basis and may be inaccurate if the assumed load distribution is incorrect (Williams, 2009).

3.2.2 Nonlinear Dynamic Analysis

The most rigorous, but most computationally expensive, analysis approach is nonlinear dynamic analysis. Also known as nonlinear time history analysis (NLTHA), this involves solving the equations of motion for a numerical model of the structure at a series of timesteps to obtain the structural response. The loading, typically a ground motion record, and the response are divided into a series of data points at time intervals in the order of 0.01s. The response during each step is calculated from the initial conditions at the beginning of the step.

Different ground motion records produce different structural responses; hence it is necessary to perform a number of analyses to obtain representative results. Ideally a very large number of analyses would be performed; however this is not practical, partly due to the limited number of applicable ground motion records for each design scenario but primarily because of the limited time that can be devoted to running analyses. The number that must be performed for a stable estimate of response is dependent on both the response quantity of interest and the method by which the ground motion records are selected and scaled, but reasonably accurate measures of drift and acceleration response can be achieved with as few as one or two appropriately selected records (Hancock et al., 2008). Eurocode 8, which requires non-linear time history analysis to be performed for the design for complex structures, stipulates at least three ground motion records be applied, with design to be based on the maximum, as opposed to the mean, response if less than seven analyses are performed. In literature however it is common to use a much greater number of records; often in excess of 40 time history analyses are performed to estimate structural response. Further detail about the selection and scaling of ground motion records for time history analysis is given in Chapter 4.

As a final point, it should be remembered that when considering nonlinear dynamic analysis it is important not to allow its complexity to create the impression of high accuracy, particularly in areas such as strain demand prediction that may be beyond its capabilities (Uang and Bruneau, 2018).

3.3 OpenSees

The Open System for Earthquake Engineering Simulation, or OpenSees (McKenna, 2011), is a computational framework designed for simulating the seismic response of structural systems. OpenSees is a finite element type, fibre based program with advanced capabilities for modelling and analysing the nonlinear response of systems using a wide range of material models, elements, and solution algorithms. Compared to typical commercial structural analysis packages, OpenSees provides a much greater level of control of the various modelling and analysis

parameters. This user controllability is the fundamental reason why many researchers use OpenSees rather than other finite element software packages in which much of the analysis parameters are predefined.

3.3.1 OpenSees Architecture

The OpenSees source code is an object oriented program written in C++. Although this is editable, in most cases the user doesn't need to alter this. Instead, a script, i.e. a list of instructions to build and then analyse a model, is written using an interpreter and is implemented by the OpenSees executable (Opensees.exe). Originally, this interpreter used tcl (Tool Command Language), however in various interpreters have been developed including a graphical, Matlab based interface called the OpenSees Navigator (Schellenberg et al., 2005) and more recently a Python interpreter (Zhu et al., 2018). However, most of the available support and examples have been developed for tcl. Hence tcl was used to create the scripts used in this research. However, given the greater functionality of Python compared to tcl, particularly in terms of mathematical applications, the Python interpreter could significantly improve the versatility of OpenSees for tasks outside its core use for finite element analysis.

Figure 3-1 shows the fundamental components, or objects, in the OpenSees framework. The Domain object represents the state of a finite element model, which changes as a result of the Analysis object advancing the state of the model at each analysis step. The Domain is created by the ModelBuilder object, while the Recorder objects report information from the Domain for post-processing and visualization of the simulation results.

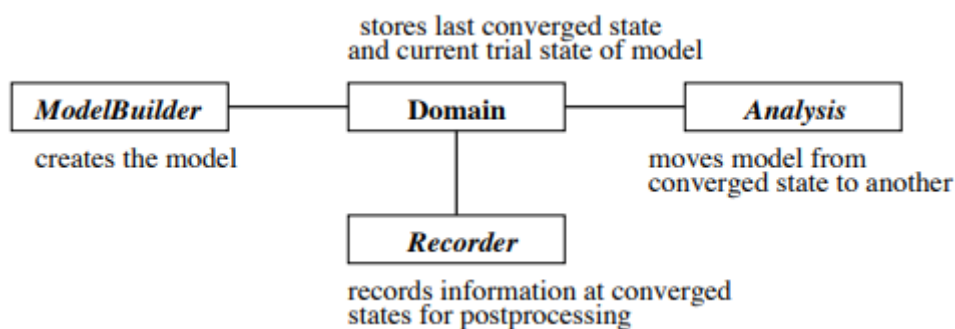


Figure 3-1 High level OpenSees objects

These high-level objects are each composed of further, more detailed, objects. Figure 3-2 shows the objects that make up the Domain. These are the fundamental components of any finite element model; namely the elements, nodes, loads, and constraints boundary conditions, which are imposed by single (SP) or multi-point (MP) constraints.

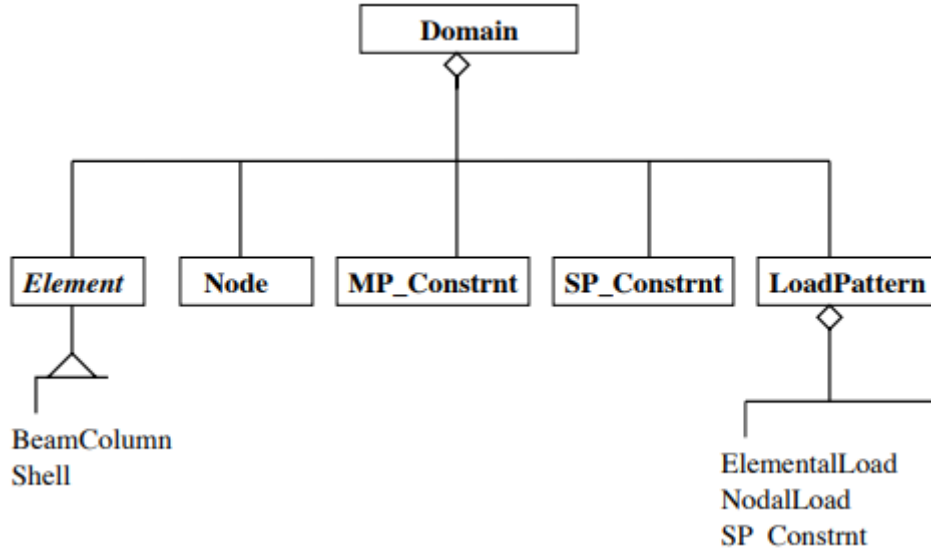


Figure 3-2 Components of the OpenSees Domain

3.3.2 Time History Analysis Using OpenSees

Time History Analysis involves solving the equations of motion at a series of time steps.

$$M\ddot{u} + C\dot{u} + Ku = P \quad 3.1$$

Where M , C and K are the mass, damping and stiffness matrices of the structure, u is a vector of nodal displacements and P is the force applied to the structure, typically, but not necessarily, from a ground motion.

This is a second order differential equation that for realistic loading cases has no analytical solution. Numerical methods can however be used to get (approximate) solutions. Put very simply, OpenSees reduces this differential equation to an equation of the form:

$$[A]\{x\} = \{b\} \quad 3.2$$

Which is sometimes referred to as:

$$[K_{eff}]\{\Delta u\} = \{P_{eff}\} \quad 3.3$$

Where x is displacement, K_{eff} can be referred to as the effective stiffness matrix and P_{eff} as the effective load vector (Paultre, 2013). How exactly these terms are formulated, and what exactly they represent, is dependent on a series of analysis control objects specified by the user in the tcl script. For example, for the Newmark integration algorithm the effective stiffness matrix and load vector can be calculated as:

$$K_{eff} = c_3M + c_2C + c_1K_T(u_{i+1}) \quad 3.4$$

$$P_{eff} = P_{i+1} - K_T(u_{i+1}) - M\ddot{u}_{i+1}(u_{i+1}) - C\dot{u}_{i+1}(u_{i+1}) \quad 3.5$$

Where K_T is the tangent stiffness matrix and c_3 , c_2 and c_1 are constants whose values are functions of the timestep and the constants, γ and β , that define the Newmark algorithm. For the Newmark scheme, it is possible to develop expressions for $\dot{u}_{i+1}$ and $\ddot{u}_{i+1}$ as a function of u_{i+1} :

$$\dot{u}_{i+1} = \frac{\gamma}{\Delta t \beta} (u_{i+1} - u_i) - \left(\frac{\gamma}{\beta} - 1 \right) \dot{u}_i - \Delta t \left(\frac{1}{2\beta} - 1 \right) \ddot{u}_i \quad 3.6$$

$$\ddot{u}_{i+1} = \frac{1}{\Delta t^2 \beta} (u_{i+1} - u_i) - \frac{1}{\Delta t \beta} \dot{u}_i - \left(\frac{1}{2\beta} - 1 \right) \ddot{u}_i \quad 3.7$$

This allows Equation 3.3 to be solved iteratively for Δu . OpenSees uses the displacement at the previous time-step, u_i , as the initial approximation, or ‘trial’, displacement u_{i+1} to begin the iteration process.

Figure 3-3 shows the components of the Analysis object in OpenSees. The following section discusses each of these and their role in NLTHA.

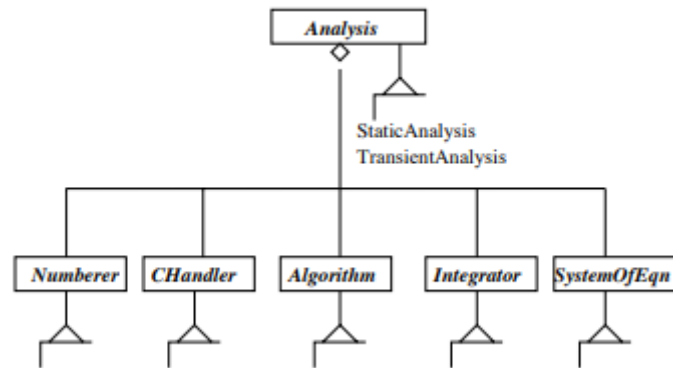


Figure 3-3 Analysis object in OpenSees

3.3.2.1 System

The *system* class defines how the terms in Equation 3.2 are stored. The simplest way is to store the matrices and vectors as they are written, this is done using the *fullGeneral* object, where no memory saving techniques are used. However, quicker computation can be achieved by storing matrices more efficiently; a number of different objects are available to do this depending on the nature of the matrices. For example, many stiffness matrices tend to be banded in nature, meaning all elements outside a diagonal band are equal to zero. The *BandGeneral* object stores these much more efficiently, as illustrated in Figure 3-4, which shows how a banded 6x6 matrix can be stored as a 6x3 matrix.

$$\begin{bmatrix} B_{11} & B_{12} & 0 & \cdots & \cdots & 0 \\ B_{21} & B_{22} & B_{23} & \ddots & \ddots & \vdots \\ 0 & B_{32} & B_{33} & B_{34} & \ddots & \vdots \\ \vdots & \ddots & B_{43} & B_{44} & B_{45} & 0 \\ \vdots & \ddots & \ddots & B_{54} & B_{55} & B_{56} \\ 0 & \cdots & \cdots & 0 & B_{65} & B_{66} \end{bmatrix}
\qquad
\begin{bmatrix} 0 & B_{11} & B_{12} \\ B_{21} & B_{22} & B_{23} \\ B_{32} & B_{33} & B_{34} \\ B_{43} & B_{44} & B_{45} \\ B_{54} & B_{55} & B_{56} \\ B_{65} & B_{66} & 0 \end{bmatrix}$$

Figure 3-4 Illustration of how the *BandGeneral* object efficiently stores a banded 6x6 matrix as a 6x3 matrix (http://opensees.berkeley.edu/wiki/index.php/BandGeneral_SOE)

3.3.2.2 *Numberer*

The role of the *numberer* class is to sort the degrees of freedom specified in the model for analysis. As with the system command, the *plain* object numbers the degrees of freedom as they are inputted by the modeller. However better performance can be achieved using other *numberer* objects, such as the Reverse Cuthill McKee algorithm or *RCM*, which reduces the bandwidth of the matrices, resulting in more efficient computation.

3.3.2.3 *Constraints Handler*

The *constraints* class determines how specified values for a degree of freedom, such as a fixed degree of freedom, or a relationship between degrees of freedom, for example where a pinned connection is modelled using the equalDOF command, are enforced.

The *plain* constraints handler can only deal with single point constraints, i.e. when a degree of freedom is fixed. Although both have more fundamental mathematical formulations, the *penalty* method and the *Lagrange* multiplier methods of imposing constraints can be viewed as adding a very stiff element or an equal and opposite reaction force pair between constrained degrees of freedom. The *transformation* constraint handler is recommended for use in dynamic analysis. It works by transforming the stiffness matrix by condensing out constrained degrees of freedom. However, the transformation method breaks down if degrees of freedom are constrained in series, for example if degree of freedom 3 is constrained to degree of freedom 2, which is in turn constrained to degree of freedom 1.

3.3.2.4 *Integrator*

The *integrator* class determines the meaning of the terms in Equation 3.2, as previously illustrated for the *Newmark* integrator, and is responsible for firstly calculating so-called ‘trial’ values to start iteration, and once convergence has been achieved, updating the response quantities at the nodes given the solution. Integrators can either be explicit or implicit. Explicit integrators do not require iteration but are generally only conditionally stable and often require very small timesteps and mass at each degree of freedom. In contrast, for implicit integrators

the solution at each timestep is dependent on the present state of the system, and hence iteration is generally required. However, such methods are generally unconditionally stable.

Some integrators, such as *HHT* (Hilber et al., 1977), introduce numerical damping into the equations of motion. The role of this is to damp out higher modes of vibration, which are often not well captured by finite element software.

3.3.2.5 Test

The *test* class defines the convergence test used and the associated tolerance. When implicit algorithms are employed, a test is required to see if convergence has occurred at the end of an iteration step. This is done by checking whether the specified parameter is less than the allowable tolerance. If not, the algorithm continues iterating until either the convergence is achieved or the maximum specified number of iterations is reached. Parameters that can be used to test for convergence include absolute value of the displacement increment (the *NormDisplIncr* object) or the energy increment (the *EnergyIncr* object) at an iteration.

If the convergence criterion is not met within an acceptable number of iterations, the model is said to have ‘failed to converge’. This is sometimes deemed to mean that the structure has collapsed under the earthquake.

3.3.2.6 Algorithm

The *algorithm* class determines the series of steps taken to solve the equations at a timestep. Within the *algorithm* class, the other previously defined classes (*integrator*, *test* etc.) are called. The exact order of how this is done depends on the *algorithm* chosen.

For linear analysis, or nonlinear analysis using an explicit integration scheme where iteration is not required, the *Linear* algorithm which takes only one iteration step to solve the system of equations can be used. However, in most cases, at least for CBFs, an iterative algorithm based on Newton Raphson techniques (the *Newton* object) is required. Developments of Newton Raphson, such as *NewtonLineSearch* or *KrylovNewton*, often give improved performance.

A combination of these commands are executed when the *analyze* command is invoked in a tcl script. Pseudo-code for the algorithm class is as follows:

Integrator -> form(b)	%Create effective force vector, b
Test -> while (residual > tolerance) (	%Iterate while residual is greater than
Integrator -> form[A]	%Create effective stiffness matrix, A
System -> solve[A](Δx)=(b)	%Solve for displacement increment, Δx
Integrator -> form(b)	%Update effective force vector, b

)

How exactly this is done depends on the *algorithm* used, but these are the basic steps that must be performed. A more rigorous description is provided by McKenna (1997).

The previous section gives a brief overview of the OpenSees computational platform and how it works during time history analysis. For further detail on the internal structure of OpenSees source code the reader is referred to (McKenna, 1997), while for discussion on the capabilities and implantation of the various classes the reader is referred to the OpenSeesWiki webpage (http://opensees.berkeley.edu/wiki/index.php/OpenSees_User).

3.4 Modelling CBFs in OpenSees

Research has been carried out into how best to model CBFs using OpenSees. There are a number of key areas that need to be modelled correctly in order to accurately reflect the behaviour of CBFs.

3.4.1.1 Material Model – Concentrated vs Distributed Plasticity

Structural components can be modelled as either elastic or inelastic elements. Elastic elements are incapable of capturing any material yielding and hence are not particularly suitable in most analyses. Inelastic structural component models can be classified by the manner in which plasticity is distributed through the member cross section and over its length.

The simplest nonlinear models concentrate inelastic deformations at the end of the element, in either a plastic hinge or through a hysteretic spring element (Figure 3-5 (a) and (b)). In models with finite length hinge zones (Figure 3-5(c)) plasticity is distributed over a certain length of the element. Integration of deformations along the hinge length captures the spread of yielding more realistically than concentrated hinges. More thorough again are fiber formulation models, where a uniaxial stress-strain relationship is assigned to each fibre. Plasticity is distributed through integration of the material response over the cross section and then integration of the section response along the length of the element (Figure 3-5(d)). The most detailed, but computationally intensive, element formulation involves discretising the continuum into small finite elements (Figure 3-5(e)). Due to the level of computational effort, continuum finite element models are generally only practical for simulating local details (D’Aniello et al., 2013).

Certain phenomena, such as material strength degradation, can be better modelled with phenomenological concentrated plasticity models (Deierlein et al., 2010). However, fibre and finite element type modes simulate certain behaviours more fundamentally than concentrated plasticity models. Concentrated plasticity models do not capture axial force-bending moment

interaction, with a single value for capacity assumed throughout the analysis, whereas in reality this varies as function of the axial force. Furthermore, for distributed plasticity elements it is not necessary to carry out a specific calibration of the inelastic material model (D’Aniello et al., 2013). This is in contrast to concentrated plasticity elements, where the material models employed are dependent on the cross-sectional properties of the element represented. For example the properties of a spring element using the modified Ibarra Medina Krawinkler material model (Lignos and Krawinkler, 2010) at the end of an S355 203x203x46 UKC and S355 305x305x97 UKC column would be different, despite the two elements being composed of the same material. A further important benefit to distributed plasticity elements is that the calculated response is not sensitive to the choice of damping model (Chopra and McKenna, 2016).

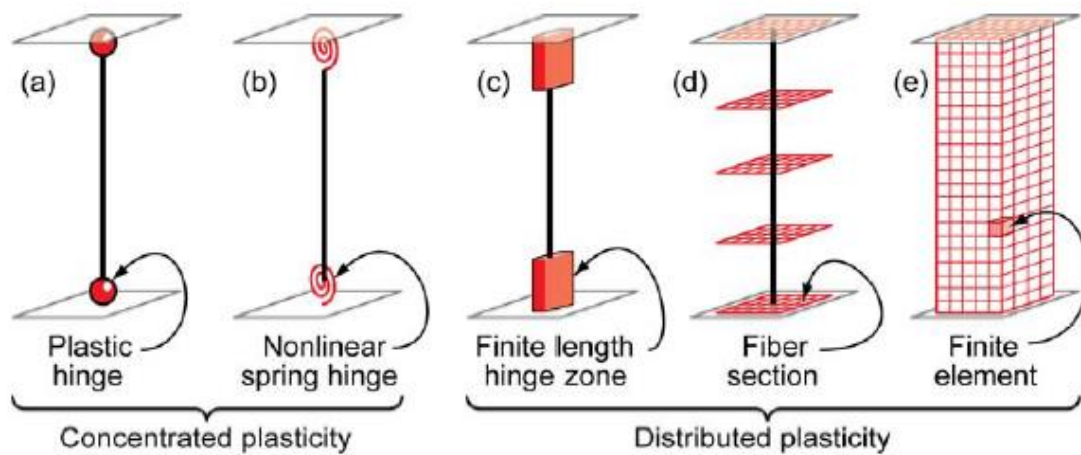


Figure 3-5 Concentrated and distributed plasticity models (Deierlein et al., 2010)

3.4.1.2 Force based vs Displacement based elements

Distributed plasticity frame elements in OpenSees can be formulated with either a displacement or force based approach. For full details on the exact mathematical formulation of the two element types the reader is referred to Hunt (2013). Displacement based shape functions, following standard finite element procedures based on the so-called ‘weak form’ of equilibrium, are used in the displacement based approach. In the force based formulation equilibrium between element and section forces is strictly imposed and hence the force based formulation can be considered exact with respect to the displacement based formulation (D’Aniello et al., 2013). The accuracy of the solution obtained using force based elements can be improved by increasing either the number of integration points or the number of elements, whereas for displacement based elements this can only be achieved by increasing the number of elements,

with larger computational effort required to achieve comparable accuracy to force based elements.

3.4.1.3 Bracing Members

Force based, fiber type distributed plasticity elements are typically used to model bracing members (D'Aniello et al., 2013). This means elements are discretized into sections, or integration points, and sections are further discretized into fibers, as illustrated in Figure 3-6. The Steel02 material, which represents the Giuffre-Menegotto-Pinto model shown in Figure 3-7, is generally used.

Uriz et al. (2008) showed that discretizing bracing members into two elements with three integration points per element and ten to fifteen layers across the depth of the brace cross section is sufficient to capture global behaviour. To capture local behaviour more integration points are required.

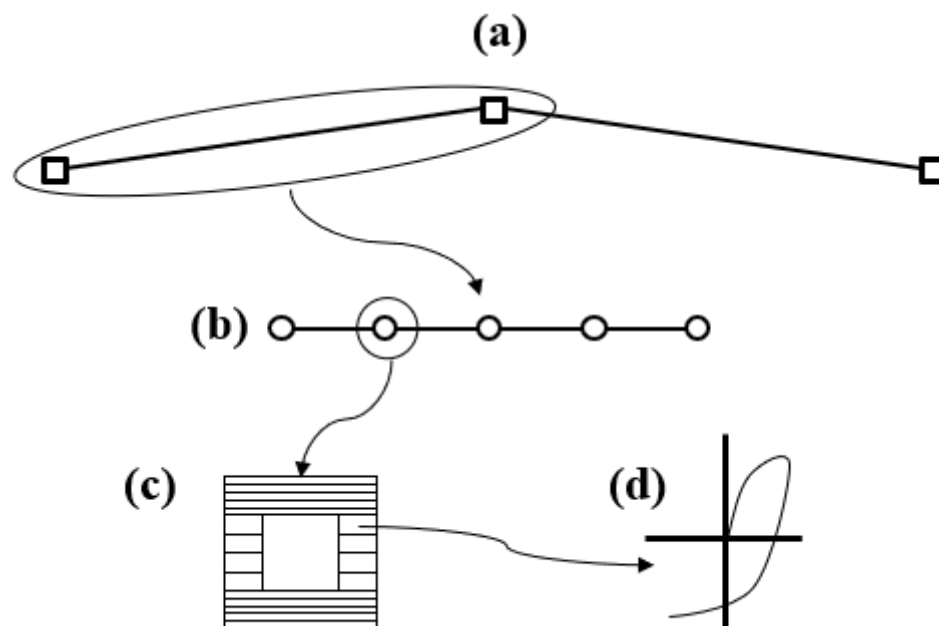


Figure 3-6 OpenSees element discretization showing: a) Brace finite element model with initial camber, elements and nodes; b) Integration points along an element; c) Fibre discretization of the cross section and d) Material stress-strain response associated with a fibre

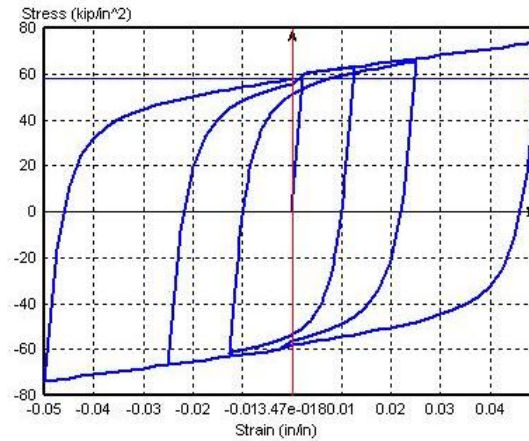


Figure 3-7 Example stress-strain relation for the Steel02 material

The bracing members must be modelled in a manner that allows them to buckle in compression and exhibit the hysteretic behaviour described in Chapter 2. This can be achieved by including an initial camber in the brace model. A series of different studies have investigated the ideal value of brace camber to correctly capture global buckling based on experimental work. Table 3-1 gives the values recommended by these studies. The value of 0.1% of the brace length proposed by Uriz et al. (2008) is often adopted in studies that examine the response of CBFs without particular focus on model calibration (for example in Hwang and Lignos (2017a)).

Table 3-1 Recommended initial brace camber to capture brace hysteretic behaviour in various studies

Study	Recommended Camber (% of brace length)
Uriz et al. (2008)	0.05 – 0.1
Wijesundara (2009)	0.3
Salawdeh and Goggins (2013)	0.1-1
Ryan et al. (2017)	0.8

Figure 3-8 shows the values recommended by various studies where the initial camber was calibrated based on experimental data. The recommended camber is plotted against the non-dimensional slenderness of the members from which the recommendations were derived. It can be seen that there is a general, albeit loosely defined, trend showing that recommended initial camber increases with non-dimensional slenderness.

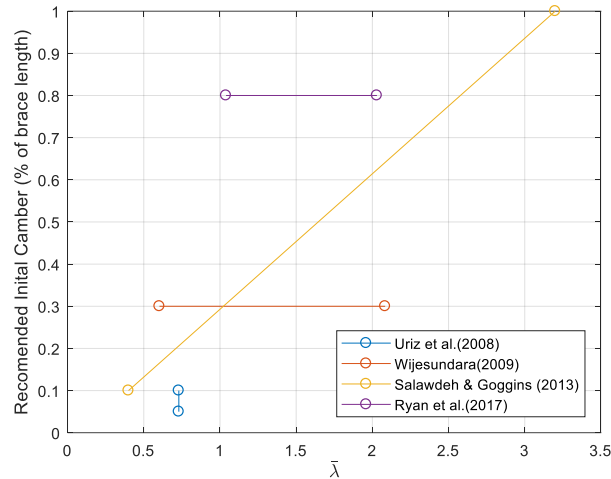


Figure 3-8 Variation in recommended initial brace camber calibrated from experimental data with the range of non-dimensional slenderness values of the bracing members used in the experimental investigation

OpenSees has the capability to capture low cycle fatigue and brace fracture (Uriz et al., 2008). This is done using a ‘wrapper’, which estimates the damage experienced by each fibre for each loading cycle using Miner’s rule. If the cumulative damage for a particular fiber exceeds a threshold value, the material resistance in the fiber drops to zero. Various studies (for example Salawdeh and Goggins (2013)) have attempted to use experimental results to calibrate the parameters that control the fatigue models. However, recommendations have been shown to be very dependent on the particular experimental results as well as the details of the OpenSees model itself (e.g. brace length, number of integration points, distribution of fibres) from which they are derived and no uniform values have been accepted across literature.

3.4.1.4 Modelling Gusset Plates

Gusset plate connections can be modelled as purely pinned, by using either a fibre connection model (Uriz and Mahin, 2008) or using a nonlinear rotational spring model (Hsiao et al., 2012). These modelling options are illustrated in Figure 3-9. The blue elements in the illustration represent the rigid elements which are used to capture the increased bending stiffness of members around the connection. Terzic (2013) showed that the fibre and spring approaches returned similar results, but that assuming fully pinned connections are only adequate for basic models.

(a)

(b)

(c)

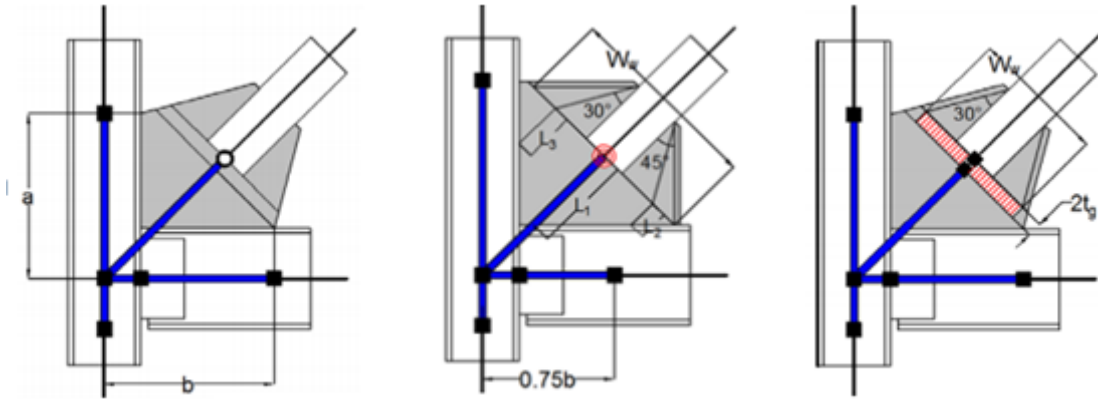


Figure 3-9 Illustration of gusset plate models in OpenSees; (a) Pinned Connection Model, (b) Rotational Spring Model (Hsiao et al., 2012), (c) Fibre Connection Model (Uriz et al., 2008)

3.4.1.5 Beams and Columns

Beams and columns can be modelled as either elastic elements or using either concentrated plasticity or distributed plasticity elements. Using elastic elements allows for quicker analysis but means no nonlinear material behaviour can be captured. Nonlinearity can be captured using either distributed plasticity elements, or by concentrating plasticity in rotational spring elements at the end of the members. Distributed plasticity, fibre based elements allow for behaviour to be simulated more fundamentally, with yielding possible throughout the element and avoid the need to calibrate the material model for each individual element. However, concentrated plasticity models, using spring or hinge elements at the ends of beams or columns, allow material models that capture degradation to be employed. This means such models can be more useful in collapse assessment.

3.4.1.6 Damping

Energy dissipation through hysteretic response of structural elements, primarily the bracing members in CBFS, is captured directly in OpenSees for components modelled as nonlinear elements. Damping in OpenSees, and in most NLTHAs, generally refers to the portion of energy dissipation that is not captured in the hysteretic response of components included in the model, and may be more accurately termed “unmodelled energy dissipation” (FEMA, 2009). This is typically applied as Rayleigh damping (Chopra and McKenna, 2016). A classical damping matrix can be assembled as a linear combination of the mass and stiffness matrices:

$$C = a_0 M + a_1 K \quad 3.8$$

Where C , M and K are the damping mass and stiffness matrices respectively, and a_0 and a_1 are mass and stiffness proportional coefficients. The values of these coefficients can be obtained from:

$$\begin{Bmatrix} \xi_k \\ \xi_n \end{Bmatrix} = \frac{1}{2} \begin{bmatrix} \frac{1}{\omega_k} & \omega_k \\ \frac{1}{\omega_n} & \omega_n \end{bmatrix} \begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} \quad 3.9$$

where ξ and ω are the damping ratios and frequencies of any two modes, k and n . In order to solve this equation for a_0 and a_1 the damping ratios at these two modes are specified, typically in the range of 2% to 5%. A more comprehensive discussion of the theory behind damping in general and Rayleigh damping in particular can be found in most textbooks, for example in Clough and Penzien (1995).

Charney (2008) discussed unintended consequences of applying Rayleigh damping for inelastic structural systems. It was shown that when the stiffness portion of the damping matrix is based on the initial stiffness matrix artificial damping is generated when the structure yields. This can lead to invalid response calculations. In order to avoid this, it was recommended to calculate damping based on the tangent stiffness matrix. When this done, and the Rayleigh proportionality constants are calculated continuously based on updated frequencies at each timestep, virtually no artificial damping is present. The influence of these artificial damping forces generated when the initial stiffness matrix is used on the response of CBFs was highlighted by Karamanci and Lignos (2014). However, Chopra and McKenna (2016) have shown that the problem of artificial damping forces is limited to models that employ concentrated plasticity elements, i.e. concentrated plastic hinges at the end of beams and columns. For force-based distributed plasticity models using fibre sections the structural response is shown to be insensitive to the damping model, with Rayleigh damping calculated using either the initial or tangent stiffness matrix giving similar results.

3.4.1.7 2 Dimensional and 3 Dimensional Models

In the vast majority of cases buildings are designed so that only certain parts of the structure resist seismic load; the remaining structural elements resist gravity loads only and are sometimes referred to as the 'gravity frame'. A typical example of such a scenario is presented in Figure 3-10, where the only the central bay of the perimeter frame is considered to be earthquake resistant, with the remaining beams and columns only supporting vertical load.

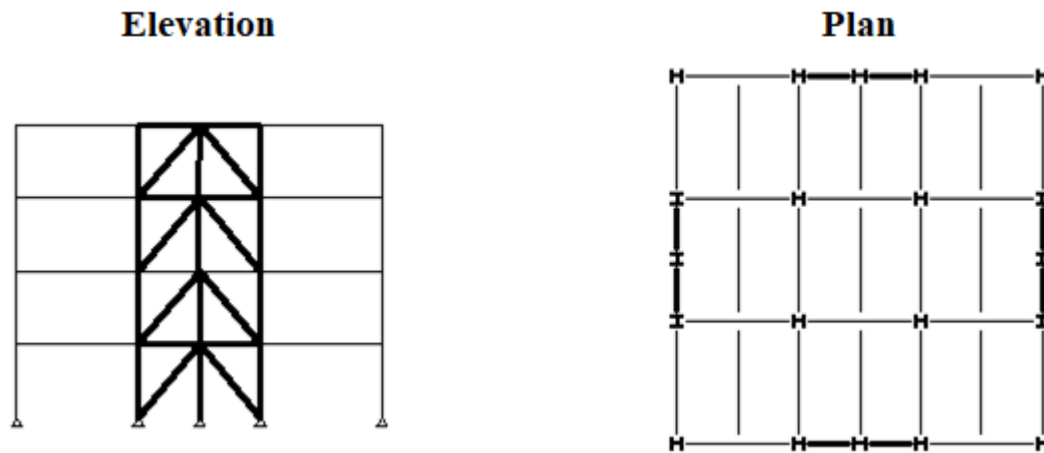


Figure 3-10 Illustration of a CBF structure where seismic resistance is provided by central bay of the perimeter frame only

Clearly, real structures exist in three dimensions, and the structure must be capable of resisting lateral load in 2 orthogonal directions on plan. However, in the modelling process it is typical to develop planar, i.e. two dimensional, models of the seismic resisting elements that provide lateral stiffness in one direction only. Two dimensional models are more computationally efficient than their 3D equivalent as each node only has 3 degrees of freedom, as opposed to 9 in 3D.

The validity of employing 2 dimensional models to model the response of regular three dimensional structures has been proven using a series of shaking table test on a 4 storey moment resisting frame structure (Lignos et al., 2012). From the results of a blind prediction numerical modelling contest with over 40 entries, it was concluded that there is no advantage clear to using 3D as opposed to 2D models.

However, when the system deforms laterally, P-Delta effects due to the vertical loads on the gravity columns can become important. Under large displacements, second order moments are excited at the base of gravity columns. Since these columns are not designed to resist these moments, they can only remain stable by the aid of horizontal forces exerted on the lateral system. In other words, these columns 'lean on' the lateral system to remain stable while displaced laterally.

As these gravity loads are not included in 2D models, it is common practice to capture these second order effects by including an extra fictitious pinned-base column with much greater stiffness than the columns of the seismic resisting frame, called a leaning column (Geschwindner, 2002). The total vertical load at each floor level imposed on the gravity columns is placed on the leaning column. Rigid elements are used to connect the seismic frame and

leaning columns. These links are connected to the columns using very low stiffness spring elements so that the columns do not attract significant moments.

Generally unless the ground motion intensity is very strong, the impact of leaning columns on results is negligible. However, under strong ground motions, where P-Delta effects become significant, gravity loads on the leaning column element can become influential, particularly when collapse probability is being assessed. The p-delta or co-rotational coordinate transformation must be employed to allow second order effects to be considered.

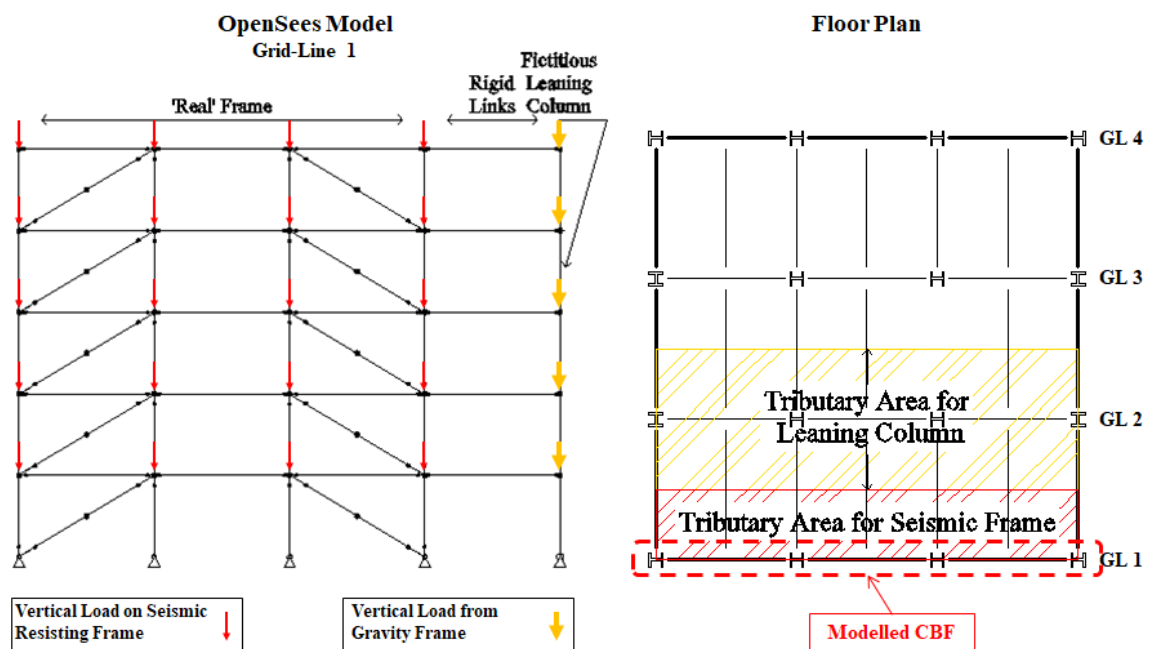


Figure 3-11 Illustration of gravity loads in an OpenSees model including fictitious leaning column element for a 5 storey structure with seismic resistance provided by perimeter CBFs

3.5 Conclusions

This Chapter discussed numerical methods for seismic analysis, with the focus primarily on time history analysis. The OpenSees computational platform was introduced and the analysis commands for solving the equations of motion at each timestep were discussed. An overview of modelling practices and research related to modelling CBFs in OpenSees was then provided.

4 Performance Based Earthquake Engineering

4.1 Introduction

The primary goal of traditional seismic design code provisions has been to ensure life safety and prevent structural collapse. Recent large earthquakes have shown that buildings designed in accordance with modern codes are generally capable of meeting these no-collapse requirements (Clifton et al., 2011, Saatcioglu et al., 2013, Okazaki et al., 2013).

However, whilst collapse is unlikely, even structures designed in accordance with modern codes do not necessarily provide sufficient resilience. This idea emerged initially in the aftermath of the 1994 Northridge and 1995 Kobe earthquakes where the amount of damage, downtime and economic loss were unacceptably high (Lee and Mosalam, 2006). Other examples of inadequate building performance emerged from the 2010 Chile earthquake, where significant economic losses were reported due to non-structural damage (Miranda et al., 2012), hospitals were evacuated due to damage to non-structural infill walls and some residents refused to return to their homes despite satisfactory performance according to available codes (Günay and Mosalam, 2013).



Figure 4-1 Non-structural damage after the 2011 Christchurch earthquake (Clifton and MacRae, 2013)



Figure 4-2 Damage windows and ceiling in a low-rise steel CBF in the Maule earthquake in Chile 2010 (Saatcioglu et al., 2013)

The Vision 2000 report (SEAOC, 1995) defined Performance Based Seismic Design (PBSD) as a design framework which results in desired system performance at various levels of seismic hazard. As summarised in Figure 4-3(a), system performance levels were defined as fully operational, operational, life safety and near collapse, while hazard levels were classified as frequent, occasional, rare and very rare. Depending on the nature of the structure being considered, acceptable performance levels were given for different hazard levels. Subsequent documents, such as FEMA 365 (2000), provided similar frameworks for assessing structural performance and incorporating it in structural design, as demonstrated in the flowchart in Figure 4-3 (b). The key step in this design philosophy is to evaluate a particular structural design and calculate performance measures that allow a judgement to be made as to whether the design fulfils performance targets. This process termed ‘performance assessment’.

Performance Based Earthquake Engineering (PBEE) is a more encompassing concept that involves construction, evaluation and monitoring as well as PBSD (Ghobarah, 2001, Bertero and Bertero, 2002). The Pacific Earthquake Engineering Research (PEER) Centre has developed a probabilistic framework for performance assessment (Miranda and Aslani, 2003) which has been further refined by the FEMA P-58 research project (FEMA, 2012a).

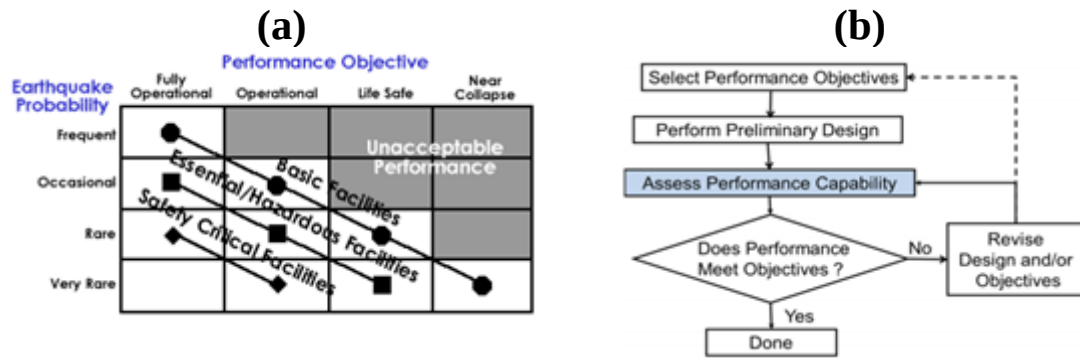


Figure 4-3 Summary of Performance Based Design introducing a) the basic idea of performance objectives (SEAOC, 1995) and b) an outline of the Performance Based Design procedure (FEMA 365, 2000)

This Chapter introduces these concepts of PBEE and performance assessment in more detail. It begins by briefly introducing some ideas about seismic hazard, which are fundamental to PBEE. The PEER loss assessment methodology and its development in FEMA P-58 are then discussed. A review of existing literature using this methodology to examine the seismic performance of various structures is presented. Structural analysis is an integral part of performance assessment, with most studies using NLTHA to obtain key response parameters. A brief overview of ground motion selection for NLTHA for use in performance assessment is given. Alternative methods of predicting response parameters are also explored. The chapter concludes by establishing several research objectives related to the seismic performance of CBFs.

4.1.1 Probabilistic Seismic Hazard Assessment (PSHA)

As can be seen from Figure 4-3, the fundamental idea of PBEE is to limit losses to acceptable levels under different of earthquakes. In order to do this it is necessary to quantify the intensity of future earthquakes that a building may experience. This is challenging as there is a great deal of uncertainty about the location, size and resulting shaking intensity of future earthquakes. However, Probabilistic Seismic Hazard Analysis (PSHA) techniques have been developed for this purpose.

The aim of PSHA is to quantify the rate, or probability, of exceeding various ground motion intensity levels at a site accounting for all possible future earthquakes and the associated uncertainty. For details regarding PSHA calculations the reader is referred to Baker (2008). The primary output from PSHA can be summarized using a hazard curve, like the one illustrated in Figure 4-4, which displays the Mean Annual Frequency of Exceedance (MAFE), or a related measure such as return period or probability of exceedance over a different time period, of a selected ground-motion parameter, such as peak ground acceleration, PGA , or spectral acceleration, S_a (Bommer and Abrahamson, 2006).

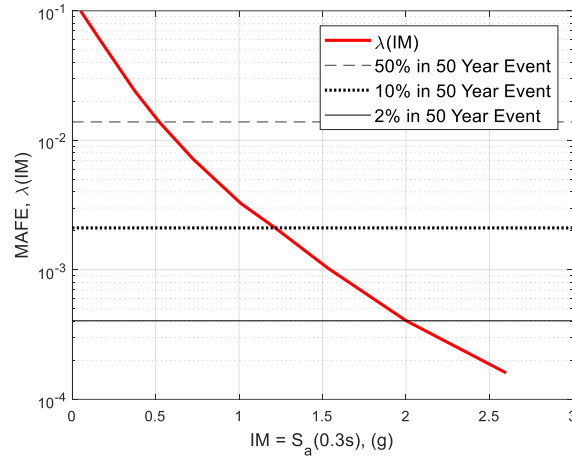


Figure 4-4 Example seismic hazard curve for a site in Oakland, California showing the Mean Annual Frequency of Exceedance (MAFE) of different ground motion intensities for structures with a fundamental period of 0.3s

4.1.1.1 Probabilities, Rates of Exceedance and Return Periods

PSHA results are generally presented in one of three different formats; annual rates of exceedance, probabilities of exceedance over a longer time period or return periods. For example the design seismic action in EC8 has a probability of exceedance of 10% in 50 years. This corresponds to a mean annual rate of exceedance of 0.0021 times and a return period of 475 years.

The relationship between annual rate of exceedance and probability of exceeding a given ground motion intensity within a period of time is controlled by the probability distribution of time between earthquakes. Poisson distributions are generally assumed as they result in simple mathematical expressions and have been shown to match observations in most cases (Baker, 2008). The Poisson model assumes that occurrences of earthquakes are independent in time; in other words the probability of an earthquake in a period of time is related only to the length of the period, and is independent of the time since the most recent occurrence. Furthermore, it is assumed that the probability of more than one occurrence in a very short interval is negligible. Under these assumptions the probability, P , of observing at least one event in a time period, t , can be written as:

$$P(t) = 1 - e^{-\lambda t} \quad 4.1$$

Where λ is the annual rate of occurrence of an event (the symbol ν is also used frequently). Similarly, the inverse relationship applies; the mean annual rate of exceedance can be calculated from the probability of exceedance in a time period using the expression:

$$\lambda = \frac{-\ln(1 - P)}{t} \quad 4.2$$

The return period is defined as the reciprocal of the rate of occurrence. Hence, taking the Eurocode 8 design level earthquake with a 10% in 50 year probability of exceedance, as an example:

$$P(50 \text{ years}) = 10\% = 0.1 \quad 4.3$$

$$\lambda = \frac{-\ln(1 - 0.1)}{50} = 0.0021 \text{ times} \quad 4.4$$

$$\text{Return Period} = \frac{1}{0.0021} = 475 \text{ years} \quad 4.5$$

This does not imply that the ground motion will be exceeded exactly once every 475 years, but rather that the average time between exceedances is 475 years.

It should be noted that at low frequencies of exceedance, such as those typically encountered in seismic design, the annual frequency of exceedance, λ , and probability of exceedance in one year, $P(1 \text{ year})$, are very similar and can be assumed to be equal (Aslani and Miranda, 2005b).

In PBEE it is common to examine building performance at earthquake intensities with 2%, 10% and 50% probabilities in 50 years. These are termed the Maximum Credible Earthquake (MCE), Design Level Earthquake (DLE) and the Service Level Earthquake (SLE) respectively. Eurocode 8 based design is carried out for two limit states; the Ultimate Limit State (ULS) where, as previously mentioned, no-collapse requirements are imposed for a seismic hazard with a 10% in 50 year probability of exceedance, while displacement limits are imposed at the Serviceability Limit State (SLS) which has a 10% in 10 year, or 41% in 50 year, probability of exceedance.

4.1.2 PEER Performance Assessment Methodology

The original performance assessment methods, such as those proposed in Vision 2000 (SEAOC, 1995) and FEMA-365 were based on deterministic, i.e. non-probabilistic, relations between structural response and performance. These are defined as first generation PBEE methods by Günay and Mosalam (2013).

Second generation PBEE methods have been developed by the Pacific Earthquake Engineering Research (PEER) Centre (Miranda and Aslani, 2003), with the main difference from first generation methods being the inclusion of a measure of uncertainty at all stages in the assessment procedure. Another development is the quantification of performance through 'decision variables' such as monetary losses, downtime and casualties, which are intended to be meaningful to all stakeholders in a development. This is as opposed to previous performance

measures, such as element deformation or force acceptability criteria (Günay and Mosalam, 2013), which were often only relevant to the structural engineer.

The PEER PBEE methodology consists of four analysis stages:

- Examining the seismic hazard at the site to obtain a seismic hazard curve describing the frequency of an earthquake intensity measure, IM , being exceeded, denoted $\lambda(IM)$.
 - This is done through the PSHA techniques discussed previously.
- Performing structural analysis corresponding to a particular earthquake intensity to calculate the probability of a structural analysis output, known as Engineering Demand Parameter or EDP , being exceeded at this intensity measure, $P(EDP/IM)$.
 - This typically involves performing a number of nonlinear time history analyses (NLTHAs) at various ground motion intensities. Peak inter-storey drift and peak floor acceleration are generally the primary $EDPs$ of interest.
 - It should be noted here that in general the notation $P(A/B)$ means the probability of event A occurring given event B has already occurred; hence $P(EDP/IM)$ refers to the probability of an EDP occurring given that an earthquake of intensity IM occurs.
- Calculating the probability of a damage measure (DM) being exceeded in various components of the facility given the occurrence of a certain EDP , $P(DM/EDP)$.
 - This is done using relations known as fragility functions, which quantify $P(DM/EDP)$ for different components, elements or systems. As an example, Figure 4-5 shows fragility functions for HSS braces (Karamanci and Lignos, 2014) and for a suspended ceiling (FEMA, 2012a). Damage measures represent the various states of damage a component can experience; for example a bracing member may be undamaged, buckled globally, buckled locally or fractured.
 - Generally, but not always, the probability of damage to structural elements is calculated based on peak inter-storey drift ($PISD$) while damage to most non-structural elements is generally dependent on peak floor acceleration (PFA).

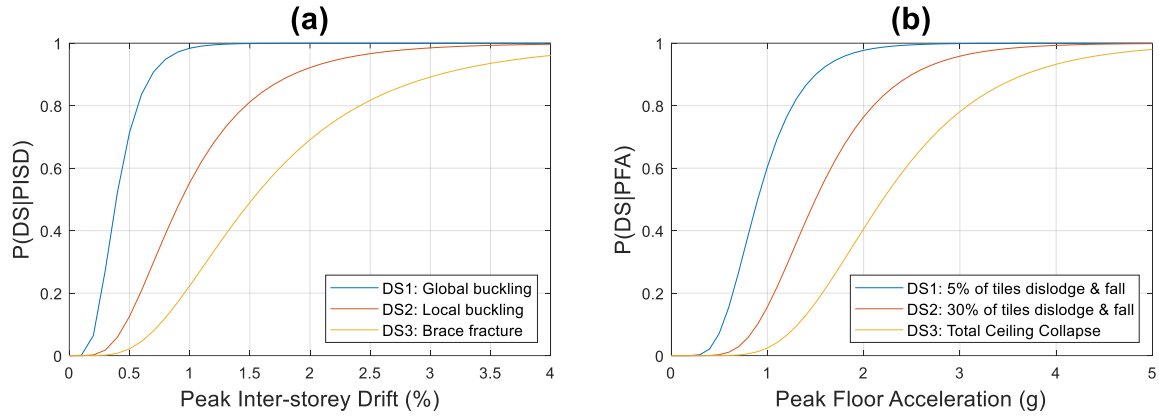


Figure 4-5 Examples of fragility functions for (a) HSS bracing members, $f_y = 355 \text{ N/m}^2$ (Karamanci and Lignos, 2014) and (b) suspended ceiling (FEMA, 2012a)

- Estimating the probability of exceedance of a decision variable (DV) of interest, such as financial loss, downtime or casualties, given the occurrence of a certain level of damage, $P(DV|DM)$.
 - The relations that quantify $P(DV|DM)$ for various components, elements or systems are known as consequence functions.

The so-called PEER framework equation (Krawinkler and Miranda, 2004), Equation 4.6, combines the four analysis stages using the total probability theorem in order to calculate the probability of a decision variable being exceeded in a specified time period:

$$P(DV) = \int \int \int P(DV|DM) dP(DM|EDP) dP(EDP|IM) d\lambda(IM) \quad 4.6$$

It should be noted that each term in the equation represents a probability distribution, i.e. has an associated median and distribution value, and hence the result obtained is also a probability distribution. While the decision variable can in theory be any measure of performance, the majority of studies use expected financial losses, denoted $E(L)$, to quantify performance.

In performance-based design, the decision variables calculated through equation 4.6 can then be used to decide whether a design meets the target performance objectives. It is possible to execute performance-based design outside of the PEER framework; seismic performance can be quantified by looking at $EDPs$, for example peak drift or floor acceleration, without calculating component damage or financial losses. Judgement on the performance of a particular design can be made using limits on peak drift or peak acceleration as a performance measure, in the way that EC8 already does for Serviceability Limit State through the drift limitations. However, the benefit of employing financial losses as a performance measure is that they are meaningful to all

stakeholders in a way that parameters of interest to the structural engineer, like peak drift, are not.

Equation 4.6 has been further developed by Ramirez and Miranda (2012) to account for the impact of possible collapse and building condemnation due to excessive residual drift. After an earthquake a building can be in one of three mutually exclusive states: i) collapsed, ii) not collapsed but not repairable meaning it needs to be demolished or iii) not collapsed and repairable. Therefore, the total expected losses, $E(L_T)$, can be computed as the sum of the losses due to collapse $E(L_C)$, losses due to demolition, $E(L_D)$, and losses due to repair, $E(L_R)$:

$$E(L_T) = E(L_R) + E(L_D) + E(L_C) \quad 4.7$$

In order to compute the expected losses for a given intensity level, each of these terms is multiplied by the probability of occurrence of the particular damage state.

$$E(L|IM) = E(L_R)P(R|IM) + E(L_D)P(D|IM) + E(L_C)P(C|IM) \quad 4.8$$

Where $P(R|IM)$ is the probability of the building needing to be repaired, $P(D|IM)$ is the probability of the building needing to be demolished and $P(C|IM)$ is the probability of collapse at the intensity level IM .

The probability of collapse, $P(C|IM)$, is obtained from a collapse fragility function developed from structural analysis, like the one illustrated in Figure 4-6.

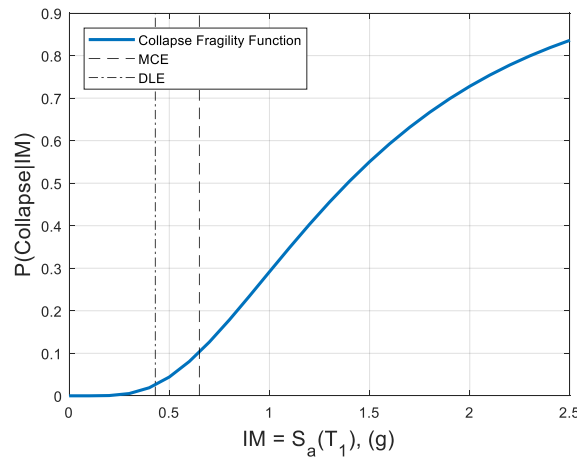


Figure 4-6 Example of a collapse fragility function for a 10 storey CBF in Seattle; with some intensity levels of interest marked

The probability of the building needing to be demolished, $P(D|IM)$, due to residual drift is computed by firstly integrating the probability of a building needing to be demolished at a

particular residual drift, $P(D/RISD)$ as illustrated in Figure 4-7(a), with the probability of occurrence of this residual drift, as illustrated in Figure 4-7(b).

$$P(D|NC, IM) = \int_0^{\infty} P(D|RISD) dP(RISD|NC, IM) \quad 4.9$$

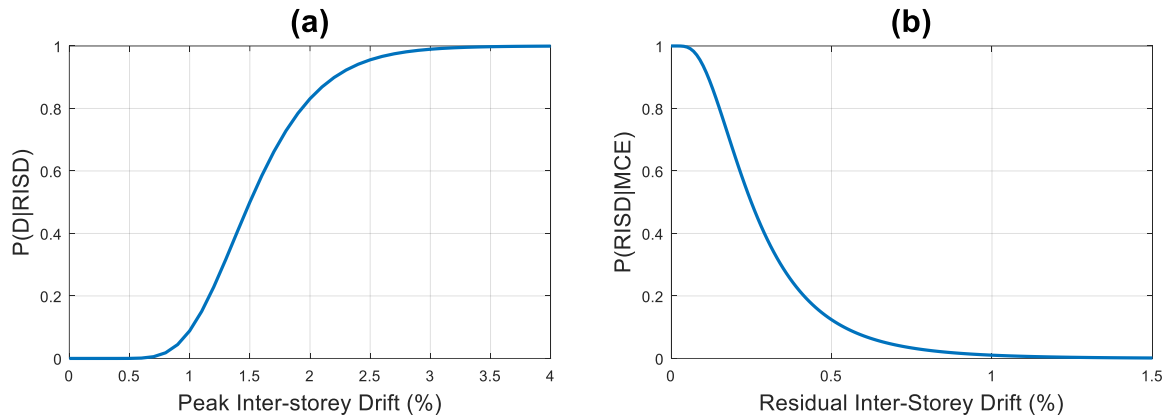


Figure 4-7 Examples of probability distributions describing (a) the probability of demolition given residual drift, $P(D|RISD)$ and (b) the probability of residual drift for a given earthquake intensity $P(RISD|MCE)$ for a case study building

Figure 4-7(a) shows an example the probability of demolition for given a residual inter-storey drift, $P(D/RISD)$, and the probability of residual drift exceeding a given value for the maximum credible earthquake, $P(RISD/MCE)$, for a case study building. These functions are integrated in accordance with Equation 4.9 to give $P(D|NC, IM)$.

This is then multiplied by the probability of non-collapse at the intensity level, $P(1-P(C|IM))$, in order to obtain the probability of the building needing to be demolished, $P(D|IM)$:

$$P(D|IM) = P(D|NC, IM)\{1 - P(C|IM)\} \quad 4.10$$

The probability of the building needing to be repaired, $P(R|IM)$, is calculated as the product of the probability of non-collapse and the probability of the non-collapsed building not needing to be demolished:

$$P(R|IM) = \{1 - P(D|NC, IM)\}\{1 - P(C|IM)\} \quad 4.11$$

Thus, using Equations 4.9 - 4.11, Equation 4.8 can be rewritten as:

$$\begin{aligned} E(L_T|IM) &= E(L_R)\{1 - P(D|NC, IM)\}\{1 - P(C|IM)\} \\ &\quad + E(L_D)P(D|NC, IM)\{1 - P(C|IM)\} \end{aligned} \quad 4.12$$

$$+E(L_C)P(C|IM)$$

Once the expected losses at a given intensity level are computed, the expected annual losses can be calculated by integrating over all possible earthquake intensities as follows:

$$EAL = \int_0^{\infty} E(L_T|IM)d\lambda(IM) \quad 4.13$$

Where $\lambda(IM)$ is the mean annual frequency of exceedance of the ground motion intensity.

Equation 4.13 gives a value for expected annual losses, sometimes termed *EAL*. However, in order to provide greater context, it is often beneficial to express losses over a building's lifetime as opposed to on an annualized basis. Lifetime losses are generally calculated using a discount factor, which converts the costs due to earthquakes in the future into present value, *PV*, accounting for inflation. This is done using Equation 4.14 proposed by Wen and Kang (2001a):

$$PV = EAL \times \frac{1 - e^{-rt}}{r} \quad 4.14$$

where r is the discount factor, *EAL* is the expected annual loss and t is the expected remaining life of the building. Building lifetimes of 50 years are generally assumed, while the values adopted for the discount factor used in literature are in the range of 3% (Hwang and Lignos, 2017a) to 5% (Macedo and Castro, 2017).

4.1.3 FEMA P-58 & PACT

The FEMA P-58 (FEMA, 2012a) project proposed a performance estimation methodology to assess the probable seismic performance of individual buildings based on their unique site, structural, non-structural, and occupancy characteristics. The technical basis of the methodology is the PEER framework outlined above. Structural analysis, generally NLTHA, is performed at various ground motion intensity levels allowing *EDPs* at each storey level to be calculated. These *EDPs* are then used to calculate probable damage and subsequent consequences. The Performance Assessment Calculation Tool, or PACT (FEMA, 2012b, ATC, 2012), has been developed to allow practical application of the methodology. A building performance model, created in PACT, lists the building assets at risk and their exposure to seismic hazard. The *EDPs* recorded in structural analysis, namely maximum and residual inter-storey drift and peak floor acceleration at each level, as well as the collapse fragility functions, are also inputted. Using this information PACT carries out the performance assessment calculation to evaluate various performance measures, like expected repair costs or expected downtime.

In order to do this, PACT uses a Monte Carlo procedure to develop a large number of potential earthquake responses, known as realizations, from a limited number of NLTHA results. Each realization represents one possible building performance outcome given a particular earthquake scenario. This means that the number of potential earthquakes considered in the performance calculation is much greater than the number of NLTHAs performed.

Fragility and consequence data for most common structural systems and building occupancies are included in the tool. Using this information, PACT can estimate performance metrics, such as casualties, repair costs and repair time for either a single earthquake scenario or over a building's lifetime. The flow chart in Figure 4-8 (FEMA, 2012a) shows the process adopted by PACT to assess performance for a single earthquake realization. Through examining this it can be appreciated how Equation 4.12, based around the three potential damage states identified by Ramirez and Miranda (2012), is implemented.

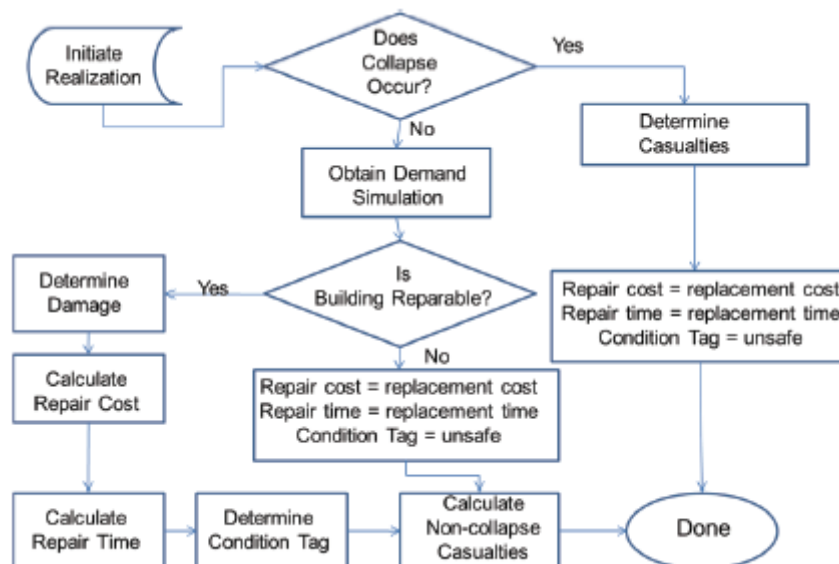


Figure 4-8 Flowchart for assessing the performance outcome in each realization in PACT (FEMA, 2012a)

In order assist with estimating the quantities of various non-structural building components for the building performance model, the Normative Quantity Estimation Tool is provided alongside the software. This estimates the type and quantity and of components present for the specified building use. The user inputs a list of floor areas and their usages, and the tool produces a floor-by-floor list of suggested components for input into the building performance model. Every suggested component has associated fragility and consequence functions in PACT.

4.1.4 Lifetime Performance Assessment of Various Building Types – Numerical Studies

As discussed, expected financial losses are often used as a metric to quantify the performance of a building. Studies that have attempted to calculate expected losses over the lifetime of a building can be divided into first and second generation studies based on the nature of the performance calculation employed (Günay and Mosalam, 2013), with second generation performance assessments using probabilistic methods to define the relationship between *EDPs* and consequences.

4.1.4.1 First Generation Studies

Many of the earliest studies that attempted to calculate lifetime performance metrics were optimizations, where lifetime losses were a parameter to be minimised. Often the primary focus in these was on the application of the optimization algorithms rather than on the accurate calculation of overall lifetime losses. In earlier studies the combination of initial construction costs and predicted seismic losses were referred to as ‘life cycle costs’, however this term is rarely used in more recent studies, possibly because a ‘life cycle cost’ is now generally understood to have an environmental component.

Examples of studies employing this methodology include those by Wen and Kang (2001b), Liu et al. (2004) and Fragiadakis et al. (2006), who all attempted to minimise the life cycle costs of steel moment resisting frames, while the studies by Fragiadakis and Papadrakakis (2008) and Gencturk (2013) looked to optimize life cycle costs of reinforced concrete structures. All of these studies employed a variation of the lifetime performance assessment framework developed by Wen and Kang (2001a), with non-probabilistic relations used to estimate damage and losses from the calculated structural response.

Using a similar performance assessment methodology, Mitropoulou et al. (2010) examined the impact of the Eurocode 8 behaviour factor, q , on the performance of reinforced concrete buildings. The study is notable as the focus was on comparing the influence of structural design decisions on life cycle costs as opposed to using life cycle costs in an optimization. Case study reinforced concrete frames were Eurocode 8 using behaviour factors of 1 to 4. It was demonstrated that, as expected, the initial construction costs were lowest for the frames designed with $q=4$ and highest for $q=1$. However, the overall life cost of frames designed with $q=4$ was the highest, due the increased losses from seismic damage over the life of the building, with the overall cost of the $q=1$ building being the lowest. This shows that while it is fair to assume increasing q reduces initial costs and hence gives more economical construction; doing

so may not necessarily result in the most economical or best solution over the building's lifetime.

4.1.4.2 Second Generation Studies

As previously stated, probabilistic relationships between *EDPs* and consequences are the main characteristic of second generation performance assessments. One of the first applications of second generation performance assessment methods was by Aslani and Miranda (2005a), who performed a detailed loss assessment of an existing seven storey reinforced concrete structure using the PEER equation. The work included the development of fragility functions for specific components of the structure. Direct economic losses were computed as a combination of losses resulting from repair and replacement of both structural and non-structural components in the non-collapse case and losses associated with demolition and replacement in the collapse case. It was shown that for the reinforced concrete building in question, probable losses primarily resulted from damage to non-structural components, with drift sensitive components contributing significantly more than acceleration sensitive components.

One of the main obstacles to application of this performance assessment methodology was the need to develop fragility and consequence functions for each element of the building. Aslani and Miranda (2005a) devoted significant attention to quantifying the contents of their case study building and developing fragility functions. Expected repair and replacement costs were calculated based on figures from RS Means (Waier, 2002) and detailed consideration of the necessary repair work for different damage states. In general, this type of work is unattractive for structural engineering researchers as it is primarily a quantity surveying type problem. This is particularly true when the facility being assessed has yet to be constructed, as opposed to an already existing building like the one examined by Aslani and Miranda (2005a). One solution to this problem has been to use the fragility and consequence functions from the HAZUS tool (FEMA, 2003). HAZUS was initially developed to calculate expected losses on a city-wide, as opposed to building specific, level. It contains coarse fragility functions for various building types. These functions simply grouped all components of a building into three damageable groups, namely drift sensitive structural components, drift sensitive non-structural components and acceleration sensitive non-structural components.

Rojas et al. (2011) and Saadat et al. (2014) both used the HAZUS fragility and consequence functions to propose an optimisation algorithm for steel moment resisting frames based on minimising total loss over the lifetime of the structure. In both studies as expected, heavier (i.e. stiffer) structures resulted in lower expected losses. When expected losses were examined by

damageable group, Rojas et al. (2011) concluded that in non-structural acceleration sensitive components contributed most to expected losses, with losses incurred by drift-sensitive structural components, i.e. the structural framing system, contributing the least. In contrast Saadat et al. (2014) showed that non-structural drift sensitive components were the largest contributor to overall losses.

Macedo and Castro (2017) also used the HAZUS fragility and consequence functions in a study that examined the lifetime losses incurred by steel moment resisting frames designed to Eurocode 8. Frames were designed using behaviour factors of 6.5, 4 and a behaviour factor calculated using the so-called Improved Force Based Design Procedure (Villani et al., 2009). Expected annual losses were shown to generally increase with the behaviour factor and decrease with building height, with losses to non-structural components contributing most to overall losses. The present value of lifetime losses obtained ranged from about 1.5% to 4% of total building replacement cost.

As discussed previously, PACT (FEMA, 2012a) was developed as part of the FEMA P-58 programme. The performance calculation is more detailed than the HAZUS based calculations as it relates *EDPs* to individual element performance and, in turn, global building performance. For example in a CBF structure, the probable damage, and subsequent losses, suffered by each bracing member is calculated based on the inter-storey drift at each floor level, as opposed to the lumping of all structural components together in the 'structural drift sensitive' performance group as in HAZUS.

One of the first studies to make use of the FEMA P-58 performance assessment methodology was conducted by Terzic et al. (2014). It compared the performance of a number of possible structural designs, including a SCBF, SMF and structures using base isolation and damping devices, for a case study building in Oakland, California. Expected financial losses were then calculated using PACT as the sum of business interruption costs and facility repair or replacement costs. It was shown that both initial construction costs and expected losses for a SCBF design were less than for a comparable special moment resisting frames (SMRFs), with a base isolated braced frame shown to give the lowest overall losses. Similarly, Karavasilis et al. (2015) used PACT methodology to examine the influence of incorporating self-centring systems and damping devices on expected losses in steel MRFs. It was shown that such devices can significantly reduce losses at higher intensity earthquakes.

Hwang and Lignos (2017b) used the FEMA P-58 performance assessment methodology to assess the financial losses suffered by SMRFs designed to US seismic provisions (ASCE/SEI, 2010, AISC, 2010) for a site in Los Angeles. The study showed that for frequently occurring seismic events losses are dominated by non-structural content repairs, however for design level and maximum credible earthquake events, losses due to collapse and demolition as a result of excessive residual drift are the dominant contributor to overall losses. The extent to which this is the case was shown to be dependent on the structural modelling assumptions. In contrast, expected annual or lifetime losses were not sensitive to the structural model employed and were dominated by repair costs of acceleration sensitive components, followed by repair costs of drift sensitive elements. The contribution of collapse and particularly demolition to annual losses became more important for taller frames. The present value of lifetime losses of the case study buildings varied from about 10% to 20% of total replacement cost. It is interesting to note that these values are approximately an order of magnitude greater than those obtained by Macedo and Castro (2017) in their study of MRFs designed to EC8, where losses were calculated using the HAZUS fragility and consequence functions. This disparity is attributable to a combination of the differences between HAZUS and FEMA P-58, the extent of the seismic hazard considered in the two studies and possibly differences between EC8 and US design provisions.

Hwang and Lignos (2017b) also investigated the impact of the strong-column weak-ratio (SCWB) used in design of the SMRFs on performance. The impact of this on annualized losses was minimal as the SCWB ratio mostly affects the formation of collapse mechanisms, and due the low frequency of occurrence, differences are not captured in expected annual loss calculations. However, collapse and demolition losses at the MCE were shown to reduce with increasing SCWB ratio. Like the work by Terzic et al. (2014), Karavasilis et al. (2015) and Macedo and Castro (2017), this component of the study demonstrates how performance assessment methodologies can be used to compare different structural designs and subsequently inform design decisions.

Another study by Hwang and Lignos (2017a) investigated the effect of modelling assumptions on the losses suffered by a series of SCBF structures designed according to the US provisions (AISC, 2005), (ASCE/SEI, 2006) at both low and high seismicity sites. Again, performance was quantified by expected losses calculated using the FEMA P-58 methodology. In terms of lifetime losses the study showed losses ranging from about 15% to 25% of the building replacement cost, which are consistent with, but slightly larger than, values obtained for MRFs. Lifetime losses were dominated by losses due to damage to acceleration sensitive components. Repair costs incurred by drift-sensitive structural components were the second most important contributor to overall

losses and became more important with increased frame height. Losses due to collapse and demolition due to residual drifts were very minor contributors to overall losses. The study also examined the expected losses at the design level and maximum credible earthquake. It was shown for taller frames at these intensities for highly seismic sites the contributions of demolition and collapse losses become very important and can dominate overall losses at the MCE intensity.

In terms of the sensitivity to modelling parameters, the study examined the influence of gravity framing and the type of brace fragility functions employed in performance assessment. These parameters were shown to have little impact on expected annual or lifetime losses. However, the modelling assumptions investigated did impact on losses at higher intensity earthquakes, particularly at the maximum credible earthquake. Collapse and particularly demolition losses at these intensities were shown to be significantly overestimated when the lateral resistance from gravity framing elements were ignored in structural modelling. Likewise, at higher intensity levels it was recommended to use dual parameter fragility curves that relate damage to braces to both slenderness and inter-storey drift (Lignos and Karamanci, 2013), as opposed to the more commonly employed fragility functions that predict damage based solely on inter-storey drift and tend to result in an overestimation of losses due to brace damage.

Del Gobbo et al. (2018) examined losses incurred at the ultimate limit state (ULS, with 10% in 50 year probability of exceedance) and serviceability limit state (SLS, which is calculated as half the EC8 elastic spectrum and nominally has a 10% in 10 years, or 41% in 50 years, probability of exceedance) earthquake intensity levels for ten case study CBFs designed to Eurocode 8. This was a development of a previous paper where the same authors had provided a detailed examination of the losses suffered at these intensities by one of the case study frames (Del Gobbo et al., 2017). The influence on repair costs of the inter-storey drift limitations imposed at the design stage by Eurocode 8, which are intended to limit non-structural damage, was investigated. It was demonstrated that applying a design inter-storey drift limitation of 0.5% at ULS, as opposed to at SLS as stipulated by EC8, and thereby increasing frame stiffness, reduced median repair costs at ULS by between 12 and 15%. However, reductions in losses at SLS were minimal.

More generally, the study showed that damage to non-structural components, particularly acceleration sensitive components, was the majority contributor to losses at both intensity levels. It was concluded that the EC8 drift limitations are inadequate for preventing damage to drift sensitive non-structural systems and don't limit accelerations sensitive damage. Another

general trend of interest to be noted from the study is that repair costs were shown to increase with frame height, something that wasn't evident in other previously discussed studies.

Further examples of research using second generation performance assessment methodologies to calculate expected financial losses include studies by Ramirez and Miranda (2012), Baradaran Shoraka et al. (2013) and Liel and Deierlein (2013) all of whom examined reinforced concrete buildings and those by Porter et al. (2006) and Pei and Van De Lindt (2009) looking at timber frame buildings. However, the body of research relating to steel CBFs is quite limited.

A more general point that arises from the studies discussed above is the importance of examining both time based loss metrics, like expected annual losses or present value of expected losses, and scenario dependant metrics, like expected losses at the design level earthquake. Time based loss metrics weight the more frequent seismic events through integration over the hazard curve and hence give a more representative metric to evaluate seismic losses. For this reason both Macedo and Castro (2017) and Hwang and Lignos (2017a) recommend using time based performance metrics. However, it should be noted that in some cases, the impacts of design decisions or modelling assumptions on performance are only noticeable at infrequent, high intensity earthquakes. This is particularly true in terms of design alterations such as self-centring mechanisms or the SCWB ratio in MRFs that only become important at high intensity earthquakes. As these earthquakes are infrequent, the benefits of reducing losses during these events are not always apparent in expected annual or lifetime losses. Hence it is often necessary to examine both time based and scenario based performance measures in order to get a complete picture of seismic performance (Hwang and Lignos, 2017b).

4.2 Selection of Ground Motion Records for Performance Assessment

Performance assessment procedures typically require a number of NLTHAs to be performed at various different ground motion intensity levels. In order to do this it is necessary to select and possibly modify a set, or sets, of ground motion records. This process is termed Ground Motion Selection and Modification (GMSM).

There are two main ways to select records for performance assessment. The most common in literature is the so-called Incremental Dynamic Analysis (IDA) approach (Vamvatsikos and Cornell, 2002). This is where the same set of ground motion records, or record, is used at all intensities and progressively scaled to higher intensities. From a GSM view, IDA is relatively simple, as only one set of records needs to be selected.

In contrast to IDA, with a Multiple Stripe Analysis (MSA) approach different sets of ground motion records are selected at each ground motion intensity level (Baker, 2013). The reason for doing this is that low intensity ground motions generally have different properties than high intensity motions. Baker (2013) identifies spectral shape, the presence of velocity pulses and earthquake duration as intensity-dependant ground motion record parameters that can influence the results of structural analysis. This means that, compared to an IDA approach, the selected ground motions are more representative of the ground motions a building may actually experience. As records are selected to match hazard-consistent targets at each intensity level, the calculated structural response can be deemed hazard consistent. The drawback to an MSA type approach is that it requires detailed knowledge of the seismic hazard at the site of interest, which is often not readily available for sites outside the USA, while greater effort is required from the analyst in the GSM procedure.

It is debatable how much of an impact such procedures have at lower intensity levels where structures generally behave elastically, however for assessment of structural response at higher intensity levels, e.g. collapse assessment, capturing the full characteristics of ground motions is important, as these dictate the shape of the spectrum at points other than the from the fundamental period, which become important once the structure yields and the period of vibration changes (Baker, 2013).

4.2.1 Techniques for Selecting Ground Motion Records – UHS, CMS & CS

The easiest way to select ground motion records for performance assessment is to use existing standardized sets of records that have been developed previously. Examples of such sets include those developed by Somerville (1997) and Baker et al. (2011) or as part of the FEMA P-695 project (FEMA, 2009). However these are only appropriate for an IDA approach. More rigorous GSM can be performed by selecting records to match properties of the seismic hazard at a site of interest.

One way of doing this is to select records to match the design spectrum specified by a design code for a site. For example, this method of selecting records was adopted by Macedo and Castro (2017) in their study assessing the performance of MRFs and by Del Gobbo et al. (2018) in their study of CBFs. However there are a number of limitations to this, primarily that design code spectra are envelopes of maximum possible earthquakes and not representative of spectral shape of real ground motion records.

More complete site specific GMSM can be performed using the results of hazard deaggregation. Hazard deaggregation, also sometimes referred to as hazard disaggregation, is essentially the process of reversing a PSHA calculation to find the relative contribution of source-to-site distance, R , and earthquake magnitude, M , combinations to the rate of exceedance of a given ground motion intensity (McGuire, 1995). The USGS Unified Hazard Tool (United States Geological Survey, 2017), or formerly the Interactive Deaggregation Tool (United States Geological Survey, 2008), provides deaggregation results for sites in the United States. Figure 4-9 gives an example of such a deaggregation.

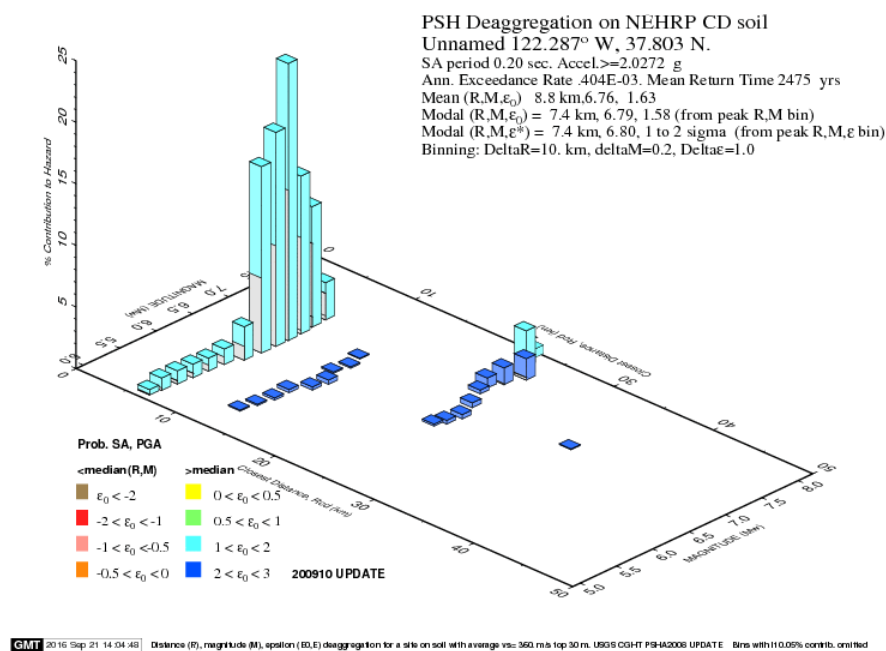


Figure 4-9 Hazard deaggregation for 2% in 50 year probability of exceedance for the Oakland site (United States Geological Survey, 2008)

Once the hazard is deaggregated, records with similar R and M combinations to those obtained can be selected. However, a more thorough process involves selecting records to match a target spectrum developed using the results of deaggregation. The Uniform Hazard Spectrum and the Conditional Mean Spectrum are two of the most common site specific target spectra employed for GMSM in literature.

Uniform Hazard Spectra (UHS) are computed by enveloping the spectral amplitudes at all periods that are exceeded at a given rate, as computed using PSHA. Hence, every point on a UHS has an equal probability of exceedance. This makes the UHS a potentially attractive target spectrum for GMSM procedures, as the probability of exceedance is independent of the period

of the structure being examined. Figure 4-10 shows UHS for 2%, 5%, 10% and 20% probability of exceedance in 50 years at a site in Oakland, California (United States Geological Survey, 2014).

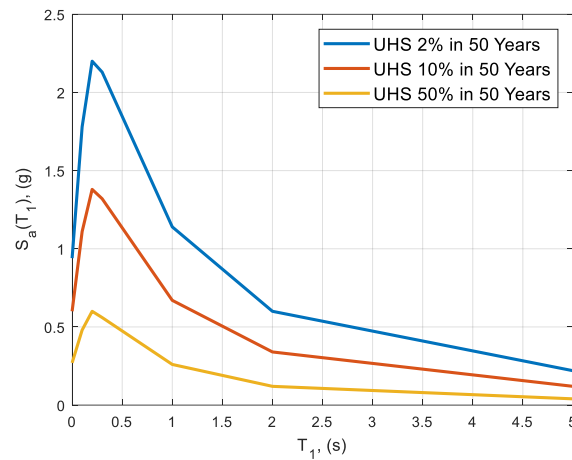


Figure 4-10 Uniform hazard spectra with probabilities of exceedance of 2%, 10% and 50% in 50 years at a site in Oakland, California

Given they are calculated from an envelope, uniform hazard spectra are generally conservative, as they imply that large amplitude spectral values will occur at all periods within a single ground motion, whereas in reality individual spectra are unlikely to be above average at all periods. Hence, it is not possible to select a single set of ground motion records that represent an equivalent hazard level for all periods while also maintaining a spectral shape representative of spectra from real ground motions. Baker (2010) discusses this issue in detail and suggests the Conditional Mean Spectrum (CMS) as a less conservative alternative method for selecting ground motion records. CMS provides the expected, i.e. mean, response spectrum conditioned on occurrence of a target spectral acceleration value at a period of interest, known as the conditioning period, T^* . This target spectral ordinate, $S_a(T^*)$, has a known probability of exceedance. Spectral acceleration values at the remaining periods are then calculated based on $S_a(T^*)$. This is done using empirical equations developed from large sets of ground motion records (Baker and Jayaram, 2008). The values obtained typically have a lower probability of exceedance than the target value, but more accurately represent the spectral shape of real ground motions. A further improvement on the CMS is the Conditional Spectrum, CS, which accounts for variation by considering the conditional standard deviation as well as the mean spectrum. For details on calculation of the CMS and CS the reader is referred to Baker and Jayaram (2008), Baker (2010) or Lin et al. (2013a).

The relationship between a UHS and CMS conditioned on various periods is shown in Figure 4-11 (Lin et al., 2013b) for 2% in 50 year probability of exceedance at a site in Palo Alto, California. It

can be clearly appreciated that the UHS is an envelope of all the potential CMS. The $S_a(T^*)$ values of the CMS at their conditioning periods equals those of the UHS.

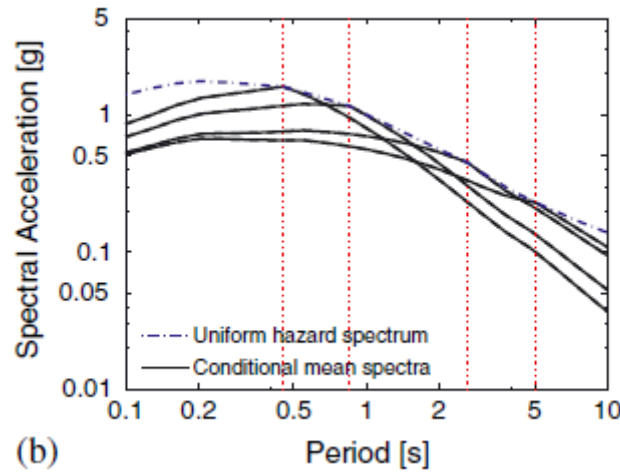


Figure 4-11 Relation between UHS and CMS conditioned at different periods, (Lin et al., 2013b)

The CMS is derived from a single period, T^* , which is generally taken to equal the fundamental structural period, T_1 . However, an obvious practical problem arises when selecting ground motions prior to the completion of structural design or when using a fixed set of ground motions for analysing several sets of structures with different periods. Employing the Uniform Hazard Spectrum may be more desirable such cases because it is invariant to the periods being considered, with the trade-off that it is conservative in enveloping S_a values at all periods (Baker, 2010).

4.2.2 Selecting Records to Match a Spectrum

Jayaram et al. (2011a) proposed an algorithm for selecting a series of ground motion records based on a target mean spectrum and variance. When matching to both mean and variance, as is the case in the CS, records cannot be treated individually; instead the suitability of a particular ground motion record can only be determined in the context of the complete ground motion set in which it may be included. This makes the selection procedure a computationally expensive problem.

Put simply, the method proposed by Jayaram et al. (2011a) works by using Monte Carlo simulation to probabilistically generate multiple response spectra from the target spectrum and variance and then selecting ground motions whose response spectra individually match the simulated spectra. Prior to the implementation of the selection algorithm, each record in a large database of potentially applicable records is scaled so that $S_a(T^*)$ matches the target $S_a(T^*)$ value. The similarity between a scaled ground motion response spectrum and a simulated

spectrum is evaluated using the sum of squared errors at a series of periods across the period range of interest. This is computed for each ground motion in the database of records and the one with the smallest value is chosen as best matching the simulated spectrum. Since the simulated response spectra have approximately the desired mean and variance, the response spectra selected using this approach will also have approximately the desired mean and variance.

MATLAB code is available online (https://web.stanford.edu/~bakerjw/gm_selection.html) (Jayaram et al., 2011b) which implements the algorithm described above using ground motion records from the PEER NGA database (Chiou et al., 2008). It should be noted here that an improved method for selecting records to match a target spectrum and variance has been recently proposed by Baker and Lee (2017), however the older algorithm was used in this study.

4.2.2.1 Impact of Record Selection Methods on Structural Analysis Results

Lin et al. (2013c) compared the structural response of 20 storey reinforced concrete frames subjected to records selected using the Conditional Spectrum (CS), Conditional Mean Spectrum (CMS) and the Uniform Hazard Spectrum (UHS). CS were calculated based on conditioning periods of 0.45s (the third natural period of the case study building, T_3), 0.85s (T_2), 2.6s (T_1) and 5s ($2T_1$). It was shown that for assessments such as those used in performance assessment type analyses, termed risk based analyses in the study, using the UHS resulted in overestimation of structural response while the CMS results are dependent on the conditioning period. Furthermore, it was shown that when using the CS, results are relatively insensitive to the choice of conditioning period. This is demonstrated in Figure 4-12 (Lin et al., 2013c).

From this analysis it was recommended to use the CS as the target spectrum for PBEE type calculations. The choice of T^* is not crucial, but one closely related to the structural response of interest is recommended. Hence, it is typical to use the fundamental structural period, T_1 .

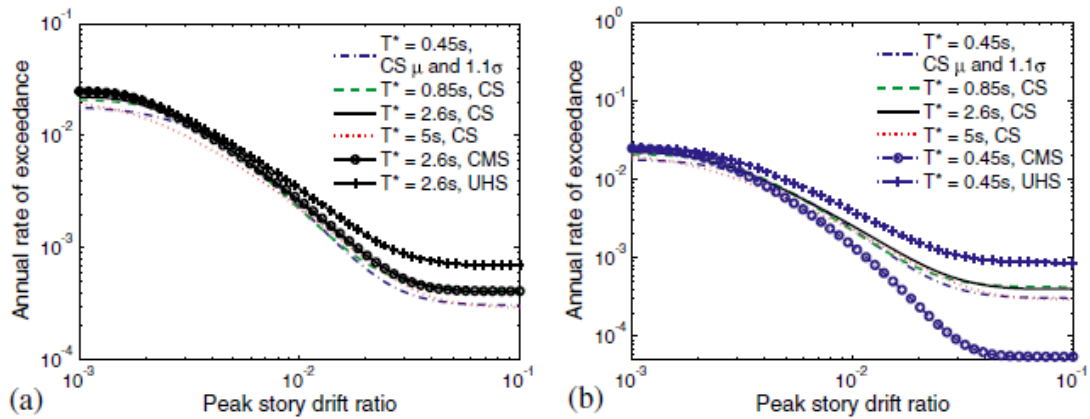


Figure 4-12 Comparing the impact of different target spectra on results of risk-based assessment for peak inter-storey drift for a 20 storey frame Lin et al. (2013c). It can be seen that: 1) results using CS as the target spectrum are insensitive to the conditioning period, 2) CMS based results are sensitive to the conditioning period 3) UHS based results are conservative and overestimate probability of exceedance of engineering demand parameters.

A final point that should be noted about GMSM is that the online hazard information available from USGS was changed during the course of this research work. Originally, two separate online tools were available; the Hazard Curve Application (United States Geological Survey, 2014) and the Interactive Deaggregation tool (United States Geological Survey, 2008). However, these were replaced by the Unified Hazard Tool in March 2017 (United States Geological Survey, 2017). The older, now decommissioned, tools were used for the work carried out in this study.

4.3 Estimation of EDPs without NLTHA

Performance assessment procedures such as FEMA P-58 and performance based design methods in general are mainly confined to academia and are not commonly used in industry (Günay and Mosalam, 2013). This is because they required accurate predictions of structural response, particularly in the post-elastic range. These are possible to obtain using sophisticated nonlinear time history analysis techniques. However, because of the considerable time, costs and expertise involved, this is often not practical (Malaga-Chuquitaype and Elghazouli, 2012). Consequently, attempts have been made to develop alternative methods to quickly estimate EDPs in different earthquake scenarios, without the need for detailed computer analysis.

4.3.1 Drift Prediction

The traditional method of estimating peak drift response has been based on the so-called equal displacements and equal energies rules proposed by Veletsos and Newmark (1960). The equal displacements rule is an empirical observation stating that for medium and long period structures the displacement of elastic and inelastic systems are approximately the same. Similarly, the equal energies rule states that for low period structures, peak displacement can be

calculated by equating the energy corresponding to maximum deformation of an inelastic system with the maximum strain energy in a corresponding elastic system.

In Eurocode 8 (CEN, 2004), design drift values are determined from elastic lateral load analysis results increased by a displacement amplification factor, q_d . By specifying the same values for this factor and the behaviour force reduction factor, q , for all steel structures, the equal displacements rule is effectively employed by the code to predict both global and inter-storey drift.

However, a series of studies on various frame types (Miranda, 2000, Ruiz-Garcia and Miranda, 2004, Medina and Krawinkler, 2005, Málaga-Chuquitaype and Elghazouli, 2011) have consistently shown that for low period structures inelastic drift response can exceed the values proposed by the equal displacements rule. Málaga-Chuquitaype and Elghazouli (2011) looked at the applicability of the equal displacements rule for CBFs in particular. Using idealized Single Degree of Freedom (SDOF) structures, it was shown that the equal displacements rule is only adequate for periods greater than 1s for stiff soils and 1.5s for soft soils, although these values were found to depend on the earthquake intensity and brace slenderness.

A number of alternative methods have been proposed to predict peak drift in CBF structures based on the examination of structures idealized as SDOF systems. These include an equivalent linearization model (Málaga-Chuquitaype and Elghazouli, 2012) and a model based on dimensional analysis (Málaga-Chuquitaype, 2015), both of which were validated reasonably successfully using finite element analysis for a limited number of multi-storey CBFs. However, while the SDOF idealization offers a simple approach to examine overall inelastic response, the most realistic drift prediction methods are those developed from models of multi-storey frames designed to a specific code or design philosophy (Kumar et al., 2013). A small number of such studies of this type have been performed for CBFs and steel MRFs.

Karavasilis et al. (2007) performed an examination of a series X-braced CBFs designed to possess specific brace slenderness values and column to brace stiffness ratios. The study proposed an expression for global drift based on brace slenderness, λ , the ratio of column to brace stiffness, α , the number of storeys, NS , the behaviour factor, q , and the ratio of fundamental structural period, T , to the corner period T_c , which represents ground motion frequency content. It was shown that the equal displacements rule overestimated drift for the majority of the frames analysed. However, for the two lowest period frames examined, global drifts were shown to exceed the value predicted by this rule.

Kumar et al. (2013) performed a large number of dynamic nonlinear time history analyses on MRFs designed to Eurocode 8 and proposed multi-variable regression models to predict global and inter-storey drift. The global drift modification factor, δ_{mod} , was used to assess roof drift and was calculated using the expressions:

$$\delta_{mod} = \frac{\Delta_{max}}{q \times \Delta_1} \quad 4.15$$

Where Δ_{max} represents the maximum roof drift recorded in a time history analysis, Δ_1 is the roof displacement at first yield, obtained from a pushover analysis, and q is the Eurocode 8 behaviour factor. As discussed, Eurocode 8 estimates drift in cases of nonlinear structural response using a drift amplification factor q_d . For steel structures the same value is proposed for both q and q_d , meaning the equal displacements rule is effectively adopted. This means that the code effectively assumes a value of 1 for δ_{mod} .

Similarly, the maximum drift modification factor, ϑ_{mod} , was used to assess inter-storey drift:

$$\vartheta_{mod} = \frac{\theta_{max}}{q \times \theta_1} \quad 4.16$$

Where ϑ_{max} is the maximum inter-storey drift obtained from time history analysis and θ_1 represents the maximum inter-storey drift at first yield, obtained from pushover analysis. Again, EC8 assumes a constant value of 1 for ϑ_{mod} based on the equal displacements rule.

Kumar et al. (2013) showed global drift to be a function of earthquake intensity, represented by the Eurocode 8 behaviour factor, q , and ground motion frequency content, represented by the ratio of fundamental structural period to mean earthquake period, T_1/T_m . Inter-storey drift was also shown to be influenced by these parameters as well as relative storey stiffness and the first mode participation factor. Figure 4-13 summarises the key findings of the study. For T_1/T_m values below 1, both δ_{mod} and ϑ_{mod} increase nonlinearly as T_1/T_m decreases. These large drifts were attributed to the elongated post-elastic fundamental period of the structure being close to the period of the ground motion and subsequent resonant effects. Similarly, for higher T_1/T_m values both global and inter-storey drifts were shown to increase due to resonance between the ground motion and higher order natural frequencies. A region of relatively constant drift, with δ_{mod} and ϑ_{mod} values lower than unity, was shown to exist for medium T_1/T_m values.

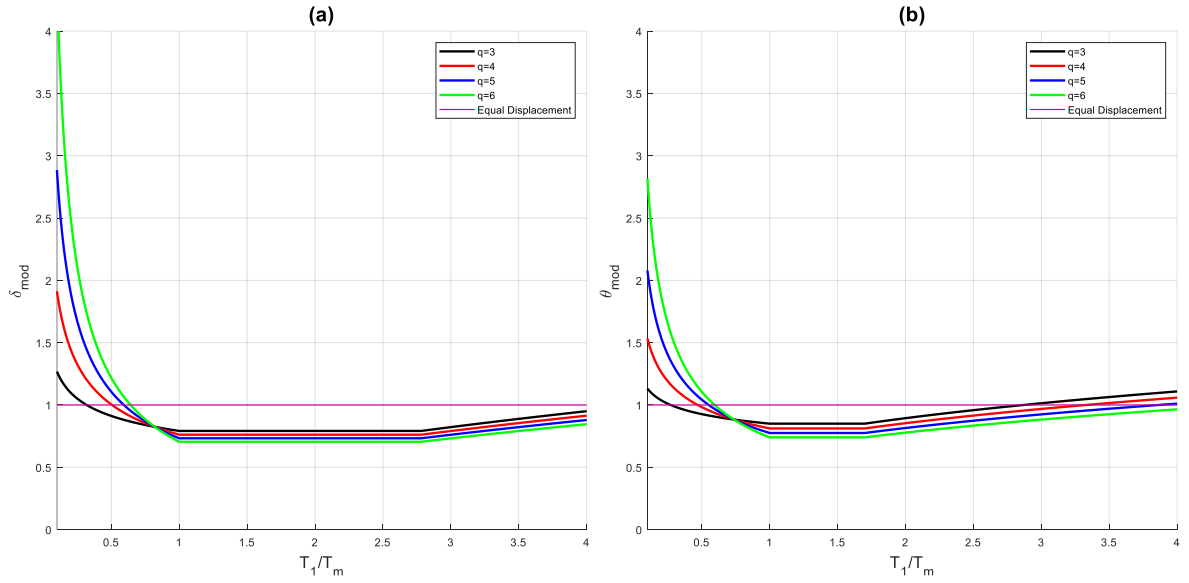


Figure 4-13 Relationships between (a) δ_{mod} and T_1/T_m and (b) θ_{mod} and T_1/T_m in MRFs predicted by Kumar, et al. (2013)

In contrast, in a similar study Karavasilis et al. (2008), concluded that, for MRFs, global drift is independent of structural and ground motion period, and influenced only by relative ground motion intensity. Kumar et al. (2013) attribute the difference in the conclusions of the two studies to the nature of the ground motion records used by Karavasilis et al. (2008).

FEMA P-58 (FEMA, 2012a) also contains a method to estimate inter-storey without the need to perform time history analysis. These can be used as the inputs for the so-called simplified analysis in PACT, which allows performance to be assessed whilst avoiding NLTHA. Instead linear analyses are performed at each intensity level of interest, with load distributed at each storey level based on the storey mass and height, in a similar manner to the Eurocode 8 equivalent static load design method. A nonlinear static analysis is also required to obtain the yield strength of the structure, which allows the extent of the nonlinearity at each intensity level to be calculated. Based on this, equations accounting for nonlinear behaviour and higher mode effects are proposed to obtain the predicted Peak Inter-Storey Drift at each floor level, $PISD_i$, under seismic excitation:

$$PISD_i = \Delta_{static,i} H_i \quad 4.17$$

Where $\Delta_{static,i}$ is the inter-storey drift at the i^{th} level obtained from a static analysis and H_i is a correction factor that accounts for inelastic behaviour and higher mode effects. Separate expressions are proposed for H_i for moment resisting, braced and shear wall structures; for braced frames under 10 storeys this is given by:

$$\ln(H_i) = 0.9 - 0.12(T_1) + 0.012S - 2.65 \frac{h_i}{H} + 2.09 \left(\frac{h_i}{H}\right)^2 \quad 4.18$$

and for braced frames between 10 and 15 storeys:

$$\ln(H_i) = 1.91 - 0.12(T_1) + 0.077S - 3.78 \frac{h_i}{H} + 6.43 \left(\frac{h_i}{H}\right)^2 - 3.42 \left(\frac{h_i}{H}\right)^3 \quad 4.19$$

here T_1 is the fundamental structural period, h_i is the height of the particular storey in question and H is the overall building height. The factor S accounts for the degree of nonlinearity experienced during the earthquake under consideration. It is given by the expression:

$$S = \frac{S_a(T_1)m_1}{V_{BS,yield}} \quad 4.20$$

Where $S_a(T_1)$ is the spectral acceleration at the fundamental structural period, m_1 is the seismic mass of the first mode and $V_{BS,yield}$ is the estimated yield strength of the structure obtained from a nonlinear static analysis.

Equations 4.18 and 4.19 were developed from the results of NLTHA using 3, 5, 9 and 15 storey archetype frames subjected to 25 near-field and 25 far-field ground motions (Huang and Whittaker, 2012), however the analyses used to develop the expressions proposed for braced frames were carried out for eccentrically rather than concentrically braced frames. In FEMA P-58 (FEMA, 2012a) the equations are presented as being applicable for 'Braced' frames.

4.3.2 Peak Floor Acceleration Response

The second primary *EDP* of interest for performance assessment is peak floor acceleration. Peak floor acceleration (*PFA*) is often normalized by peak ground acceleration (*PGA*), i.e. PFA/PGA , which is sometimes referred to as the 'floor acceleration magnification' (Rodriguez et al., 2002).

For linear behaviour, peak acceleration response, which is known to be sensitive to higher mode effects, can be predicted using modal analysis techniques. By the principles of modal superposition, elastic acceleration response can be estimated by combining the acceleration response of the first r modes by an appropriate method, such as the square root of sums squared (SRSS), or more thoroughly through complete quadratic combination (CQC). The acceleration response at a given location due to a particular mode can be obtained by multiplying the spectral acceleration for that mode, $S_a(T_n)$, by the modal participation factor, Γ_n , and the mode shape amplitude at the location of interest, φ_n . Thus peak floor acceleration at level i can be calculated as:

$$A_i = \sqrt{\sum_{n=1}^r [\Gamma_n \phi_i^n S_a(T_n)]^2} \quad 4.21$$

For nonlinear response, Rodriguez et al. (2002) proposed a method based on a simple adaption of elastic modal analysis techniques to predict inelastic roof acceleration. Equation 4.21 is altered to account for ductility by dividing the first mode contribution by an effective behaviour factor, R . Therefore, the method is termed the 'First Mode Reduced' method. Thus the peak roof acceleration is given as:

$$A_{roof} = \sqrt{\left[\Gamma_1 \phi_{roof}^1 \frac{S_a(T_1)}{R} \right]^2 + \sum_{n=2}^r [\Gamma_n \phi_{roof}^n S_a(T_n)]^2} \quad 4.22$$

The peak floor acceleration is assumed to equal the roof acceleration at all floor levels above 0.2 times the roof height, with a linear interpolation between this value and the peak ground acceleration at lower levels, as illustrated in Figure 4-14 (Rodriguez et al., 2002). Values predicted by the method were shown to compare reasonably well with time history analysis, for deriving floor accelerations at individual floor levels.

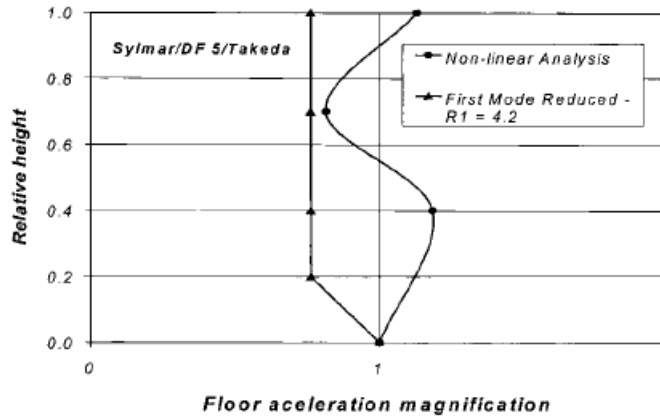


Figure 4-14 Comparison between the floor acceleration magnification obtained from NLTHA and using the first mode reduced method (Rodriguez et al., 2002)

Through NLTHA, it has been shown that as the degree of inelasticity is increased, the peak floor accelerations are generally reduced, as highlighted in Figures 4-14 and 4-15 taken from Flores et al. (2015). This trend has been shown in a variety of different studies looking at different structures, for example by Medina et al. (2006) for MRFs in general, for steel MRFs by Wieser et al. (2013) and Flores et al. (2015), and for concrete frames by Surana et al. (2017) amongst other studies. Wieser et al. (2013) attribute this to the elongation of the period of vibration that occurs

with yielding. Taghavi and Miranda (2012) concluded that there is a limit beyond which the peak floor acceleration cannot increase regardless of the ground motion intensity, a phenomenon which they term saturation of floor acceleration.

As well as the degree of inelasticity, building height has also repeatedly been shown to influence the distribution of floor accelerations over the various stories for both elastic and inelastic response. Taller buildings tend to experience peak floor acceleration values that are reasonably similar to the peak ground acceleration, except at the uppermost storeys where a sharp increase is often observed (Reinoso and Miranda, 2005, Medina et al., 2006, Flores et al., 2015). This was termed the ‘whiplash effect’ by Singh et al. (2006). In contrast, low rise buildings tend to see a more gradual increase in floor acceleration magnification over the frame height. This trend can be clearly seen by comparing Figures 4-14 and 4-15, which show the responses of various models of two and eight storey steel MRFs (Flores et al., 2015).

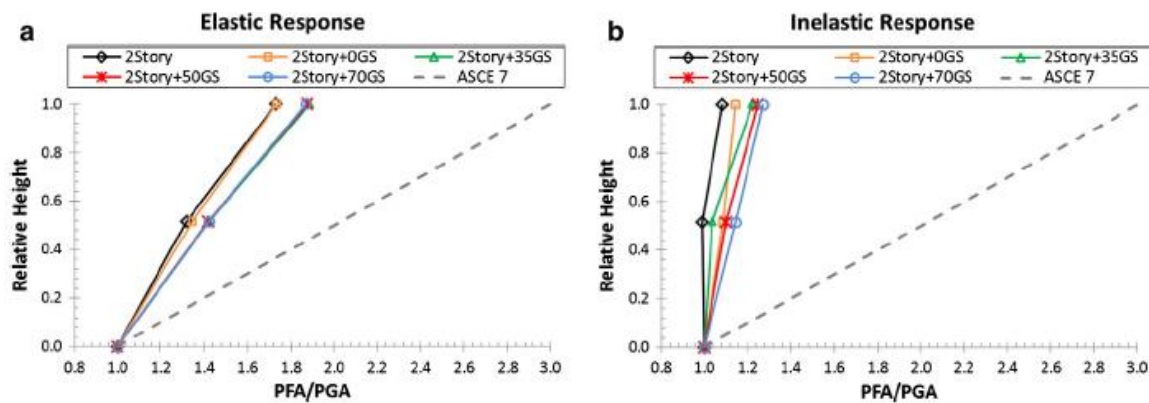


Figure 4-15 PFA/PGA for various models of a 2 storey steel MRF for elastic and inelastic response (Flores et al., 2015)

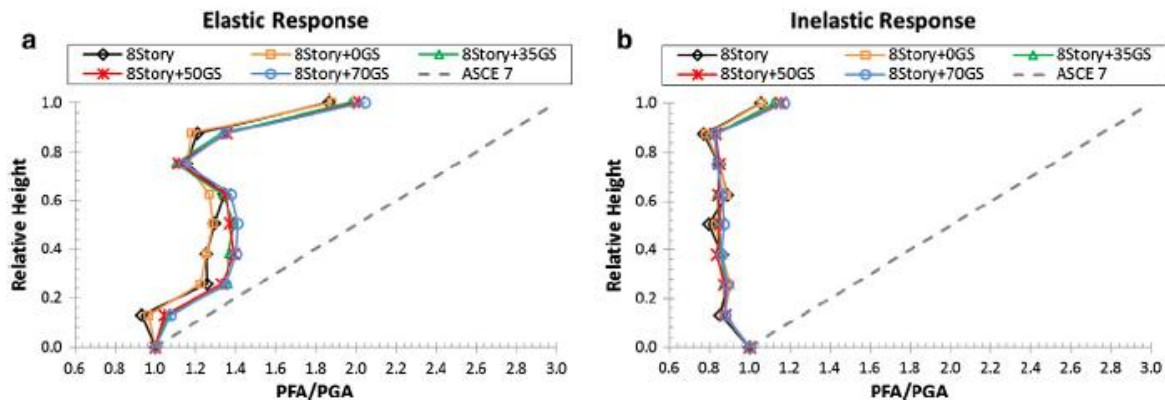


Figure 4-16 PFA/PGA for various models of an 8 storey steel MRF for elastic and inelastic response (Flores et al., 2015)

As with drift response, FEMA P-58 (FEMA, 2012a) contains expressions to represent floor acceleration at each storey level over the frame height as a function of the peak ground

acceleration. For braced frames, peak floor acceleration at the ground level is taken as equal to the PGA and then at higher floor levels as:

$$\ln\left(\frac{PFA}{PGA}\right) = 0.66 - 0.27T_1 - 0.089S + 0.075\frac{h_i}{H} \quad 4.23$$

or for between 10 to 15 stories as:

$$\ln\left(\frac{PFA}{PGA}\right) = 0.44 - 0.27T_1 - 0.052S + 3.24\frac{h_i}{H} - 9.71\left(\frac{h_i}{H}\right)^2 + 6.83\left(\frac{h_i}{H}\right)^3 \quad 4.24$$

where T_1 is the fundamental period, q_{eff} is the effective behaviour factor accounting for the extent of the nonlinearity experienced and h_i/H is the relative height of the floor level under examination. As with the predictions of drift demands, these equations can be used in simplified analysis procedure in PACT to calculate performance. As before, it should be borne in mind that these equations were developed through analysis of eccentrically rather than concentrically braced frames.

Floor acceleration magnifications in design codes are generally proposed for the design of non-structural components. Eurocode 8 contains an expression to calculate floor response spectra for this purpose:

$$S_a = a_g S \left[\frac{3 \left(1 + \frac{h_i}{H}\right)}{1 + \left(1 + \frac{T_a}{T_1}\right)^2} - 0.5 \right] \quad 4.25$$

Where a_g is the design ground acceleration, S is a factor representing the soil conditions, T_a is the period of the non-structural component and T_1 is the fundamental period of the structure. By assuming a rigid non-structural element, i.e. $T_a = 0$, the design peak floor acceleration can be calculated. Effectively, this formulation implicitly assumes that the floor acceleration magnification factor ranges linearly from a value of 1 at the base, i.e. the PFA is equal to the PGA, to 2.5 at the top of the structure. Likewise, the US provisions (ASCE/SEI, 2010) use a linear variation, except with a value of 3 at roof level. Various studies (for example Wieser et al. (2013) or Flores et al. (2015)) have shown that these values tend to be very conservative in many cases

4.4 Conclusions & Areas for Further Research

The unacceptably high economic and social losses incurred in the Northridge and Kobe earthquakes, and the insufficient resilience of modern buildings in subsequent seismic events, have led to the earthquake engineering community looking beyond collapse prevention and moving towards performance-based design approaches. A key part of this work has been the development of performance assessment methodologies, such as FEMA P-58, that allow

performance, which is generally represented expected financial losses, to be calculated probabilistically. Once performance is calculated, the ability of a design to fulfil target performance objectives can be judged.

The body of research available on the lifetime seismic performance of CBFs is relatively limited, particularly in comparison to steel MRFs. While the study by Hwang and Lignos (2017a) did examine the performance and expected lifetime losses incurred by CBFs, the focus was on how these were affected by structural modelling assumptions as opposed to how they can be influenced by the designer. Furthermore, the CBFs examined were designed in accordance with the US provisions, which can be significantly different to those designed according to Eurocode 8. Del Gobbo et al. (2018) examined how choices in design affect performance of CBFs designed to EC8. However, losses were only calculated at two intensity levels, rather than over the lifetime of the buildings examined. Beyond this, there is to the best of the author's knowledge, very little research has been performed examining the lifetime performance of CBFs.

CBFs are inherently stiff structures, which reduces drifts but can lead to relatively high acceleration demands, even under more frequently occurring earthquakes. As a result of this drift-acceleration trade off, for CBFs it is particularly important to understand performance across the entire range of possible earthquakes that may be experienced. Furthermore it is necessary to understand how the choices made in design influence this, particularly as design moves towards performance based methods. Therefore, there is a need to comprehensively examine the lifetime seismic performance of CBFs, particularly those designed to EC8, and to understand how best to limit these losses through informed decisions at the design stage.

A brief review of the literature regarding the selection and modification of ground motion records for NLTHA in performance assessment was presented. From this it can be concluded that the most thorough procedures for selecting records involves using a site and intensity level specific Multiple Stripe Analysis approach. The conditional spectrum appears to be the best target spectrum for selecting ground motion records. The majority of previous performance assessment studies have paid little attention to ground motion record selection and have used IDA type approaches. Therefore, a study that adopts best practice in GSM for performance assessment may also be valuable, as it is possible that the conclusions reached may be different, particularly for higher intensity earthquakes.

Table 4-1 Summary of recent studies examining earthquake induced lifetime losses for steel frame structures; showing how the proposed study fits in with and furthers existing work

Study	Aim	Loss Assessment	Ground Motion Selection
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		Methodology	Method
Terzic et al. (2014)	Compare losses for different types of structures (CBF, MRF, damping devices and base isolated structures)	FEMA-P58 (PACT); consider losses due to repair cost and downtime	Standard set (Baker et al., 2011); analysis only performed at 2%, 10% and 50% in 50 year intensities
Macedo and Castro (2017)	Compare losses for different of MRF structures designed to EC8	HAZUS; consider losses due to repair cost only	IDA – records selected to match EC8 design spectrum
Hwang and Lignos (2017a)	Examine influence of modelling assumptions on losses for CBF structures designed to US provisions	FEMA-P58; consider losses due to repair cost only	IDA – standard set (FEMA, 2009)
Hwang and Lignos (2017b)	Examine influence of modelling assumptions and compare losses for different MRF structures designed to US provisions	FEMA-P58; consider losses due to repair cost only	IDA - standard set (FEMA, 2009)
Del Gobbo et al. (2018)	Examine the influence of drift limitations on ULS and SLS losses for CBFs designed to EC8	FEMA-P58; consider losses due to repair cost only	Records selected to match EC8 design spectrum (ULS) and 0.5 times EC8 design spectrum (SLS)
This study aim	Examine the influence of design decisions on life costs of CBFs designed to EC8	FEMA – P58 (PACT); consider losses due to repair time and downtime	Site specific MSA – Select records to match conditional spectrum at a case study site

On top of this, in the context of practical implementation of performance assessment and performance based design, there is a need to further develop and refine methods for predicting *EDPs* without recourse to computationally expensive and time consuming NLTHA. Various studies have proposed regression equations to predict drift response for different frame types, including CBFs. However, to the best of the author’s knowledge, no study has attempted to do so for CBFs designed to Eurocode 8. Likewise, while studies have examined the general trends in peak floor acceleration response, no study has done so for CBFs and no predictive model is available specifically for CBFs designed to Eurocode 8. As drift and acceleration response are dependent on the particular details of the design methodology employed, the unique design rules imposed for CBFs in EC8, primarily the brace slenderness and overstrength limitations, mean that methods to predict *EDPs* for such structures need to be developed to enable practical implementation of PBEE and PBSO.

Hence, in summary, from a review of the relevant literature two research tasks have been identified to improve PBEE for CBFs designed to Eurocode 8. Firstly, to use the FEMA P-58 methodology to carry out performance assessment for a series of case study structures in order to develop a greater understanding of how the designer can affect and potentially improve performance and secondly, to develop methods to predict nonlinear structural response without the need for NLTHA in order to aid practical implementation of PBEE and PBSO.

5 Hybrid Simulation

5.1 Introduction

Performance Based Seismic Design involves designing buildings with an understanding of the losses that may occur as a result of different earthquakes. In order to do implement this design philosophy, it is necessary to predict the structural response for various earthquake scenarios, something which is generally attempted through numerical analysis. However, accurate numerical modelling of CBFs is challenging, and further research is needed to improve modelling capabilities.

The experimental testing facilities at Trinity College Dublin (TCD) provide an opportunity to do this through hybrid simulation. Thus, the Chapter examines hybrid simulation as experimental testing method. A review of the development of the test method and key steps in its implementation are provided. The concept of model updating is then introduced before research objectives for of an experimental testing programme are established.

5.2 Experimental Testing Procedures

Although computational methods for dynamic analysis have substantially increased their capabilities in recent years, the need for experimental testing still exists. The computational models and constitutive relationships employed in numerical simulations are developed and evaluated through experimental testing. Hence experimental testing is an essential tool for understanding the response to seismic loading, this allowing the design and construction of safer structures (Carrion and Spencer, 2007).

The need for experimental investigation is particularly acute when the response of structural elements are not well understood or difficult to model numerically, as with the highly nonlinear behaviour that occurs in bracing members. While much work has been done to develop physical theory models that can accurately capture response of CBF structures, this still remains challenging. For example, Ryan et al. (2017) attempted to validate OpenSees models from results of shake table tests on a full scale CBF carried out as part of the BRACED test programme (Salawdeh et al., 2017).

The results of this study highlight the difficulty of accurately capturing key response parameters of interest, as shown in Figure 5-1 for 3 different earthquake intensity levels termed the Operation Basis (OBE, corresponding to a 50% in 50 year intensity), Contingency Level (CLE, 10% in 50 year intensity) and Maximum Credible Earthquake (MCE, 2% in 50 year intensity). Peak

drifts were underestimated while peak acceleration response was overestimated, particularly for frequently occurring earthquakes.

In particular, it is notable that peak accelerations at low intensity levels appear particularly challenging to predict accurately. This is important in the context of lifetime loss assessment because of the high weighting of losses at these frequently occurring intensity levels in the lifetime loss assessment procedure, particularly so for CBFs where previous studies have highlighted the importance of acceleration dependant losses.

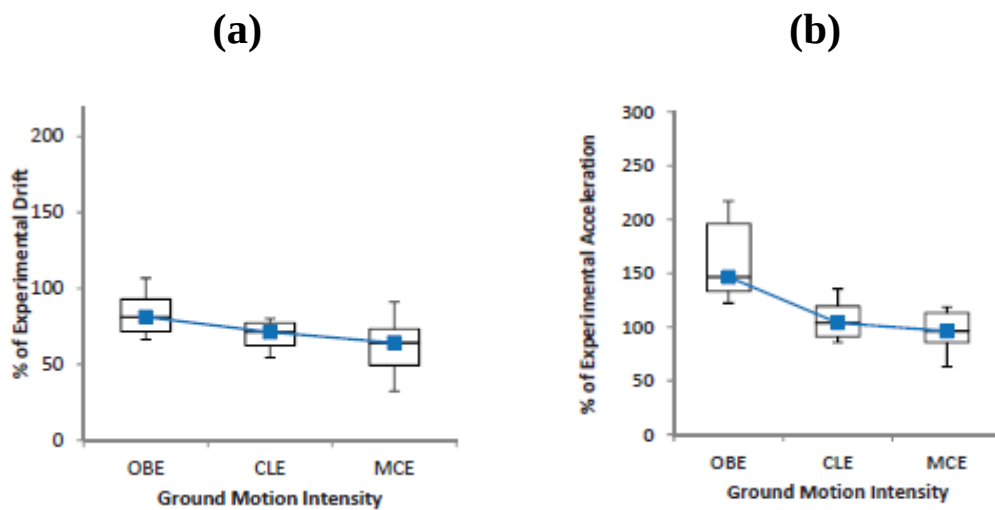


Figure 5-1 Peak drift and acceleration response calculated through NLTHA shown as a percentage of experimentally recorded values (Ryan et al., 2017)

The extent of the error in *EDP* prediction illustrates the ongoing need for experimental testing of CBF structures in order to further refine improve modelling capability. This is the motivation for exploiting the experimental facilities at TCD to further investigate the response of CBFs.

At present, there are essentially three well-established methods to conduct laboratory testing of structural components and systems; namely quasi-static testing, shake table testing and pseudo dynamic, or hybrid, testing.

Of these, quasi-static test methods, such as the static cyclic test, are the most common technique employed. While such tests are relatively easy and economical to perform, the demands imposed on the test specimen cannot be directly related to those experienced in an actual earthquake.

Shake table tests provide the most realistic method of subjecting a model structure to simulated earthquake loading, and give important insight into seismic behaviour. However, shake tables place limitations on the size, weight and strength of the specimens being tested making it very

difficult to test full scale systems. Models can be reduced in size to overcome this drawback, however scaling difficulties then arise and it can be problematic to relate results from a scaled test to a full sized model.

Hybrid simulation, formally known as pseudo-dynamic testing, offers an alternative cost-effective solution for large scale laboratory testing of structures under simulated earthquake loading. The term ‘hybrid test’ covers a broad family of test methods where part of the structural behaviour is tested physically whilst other terms are modelled numerically (McCrum and Williams, 2016). Typically, the components tested experimentally are critical or highly nonlinear portions of the structure that are challenging to model numerically, while elements with predictable behaviour are modelled on the computer. This concept, known as substructuring, avoids the need to test large, heavy structures. This allows full scale specimens to be tested, avoiding the scaling issues associated with shake table testing. Consequently, the hybrid test method is extremely useful for modelling structures exhibiting complex nonlinear behaviour, particularly if this is concentrated in specific regions of the structure.

5.3 Hybrid Simulation

The concept of hybrid simulation is presented graphically in Figure 5-2. During the test, the dynamic response of the structure due to a particular loading is calculated numerically on a computer using time step-integration of the equations of motion. The calculated displacements are then applied to the test specimen using actuators. The forces required to produce these displacements are measured and fed back to the computer to calculate the displacements corresponding to the next time step. Effectively therefore, the test method is a computational dynamic analysis in which the stiffness term is measured physically.

Hybrid testing is based on solving the equations of motion at various time intervals throughout the test. The equation of motion for a dynamic system can be expressed as:

$$M\ddot{u}(t_{i+1}) + C\dot{u}(t_{i+1}) + R_{i+1} = P(t_{i+1}) \quad 5.1$$

Where M and C are the mass and damping matrices, $\ddot{u}(t_{i+1})$ and $\dot{u}(t_{i+1})$ are the time dependant acceleration and velocity response vectors, R_{i+1} is the vector of nodal restoring forces and $P(t_{i+1})$ is the applied load. In a linear analysis the problem can be simplified by letting $R = K(u(t_{i+1}))$, where K is the initial stiffness matrix and $u(t_{i+1})$ is the displacement vector. In a nonlinear hybrid simulation, measured forces from the physical test specimen can be used to determine the value of R_{i+1} .

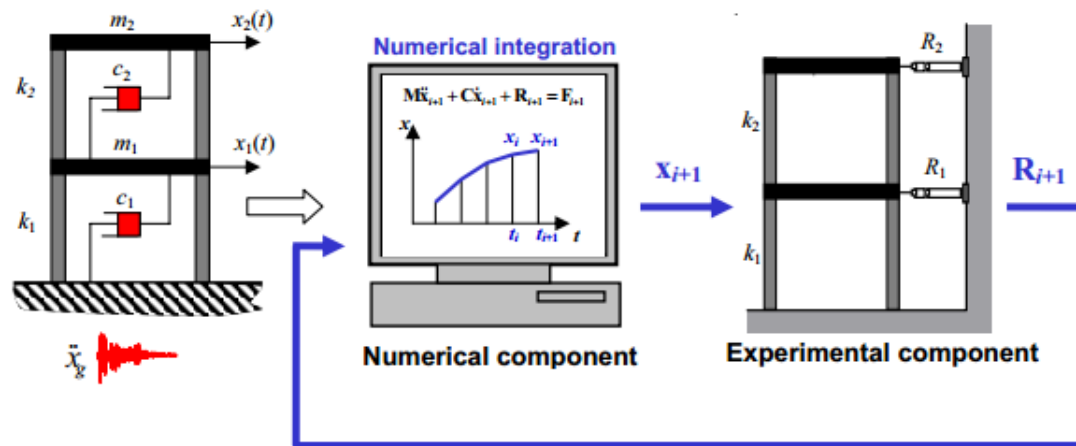


Figure 5-2 Summary of Hybrid Simulation (Carrion and Spencer, 2007)

5.4 Development of the Hybrid Test Method

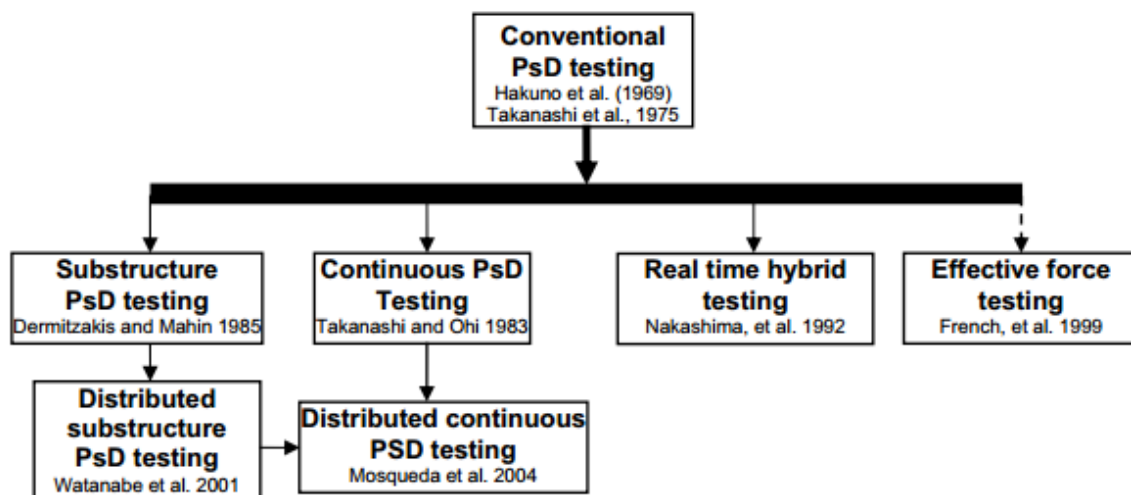


Figure 5-3 Development of the Hybrid Test Method (Carrion and Spencer, 2007)

Figure 5-3 (Carrion and Spencer, 2007) shows the development of the hybrid test method over time. The earliest hybrid tests, now termed Conventional Pseudo Dynamic (PsD) Tests, combined an analogue computer that solved the equations of motion and an actuator which imposed the calculated displacements on a SDOF structure in a quasi-static manner. In the PsD test method it is assumed that the structure can be represented by lumped masses at the location of the actuators and the equation of motion is solved in a stepwise manner throughout the test. The inertia and viscous damping properties of the structure are modelled analytically while the structural stiffness input into the computer model is obtained from the test specimen.

A quasi-static loading rate is required so as not to induce inertia or damping response as these are already accounted for numerically. In these Conventional PsD tests the actuator movement

is performed in a ramp and hold manner, meaning the method contains a pause phase in the order of some seconds at each time step to allow the numerical integration of the equations of motion to be completed, the displacements to be imposed and the restoring forces to be measured. These pause periods prevent smooth movement of the structure and can introduce spurious load relaxation (Magonette, 2001).

A development of the Conventional PsD method is the Continuous PsD test which sees loads applied smoothly, without the starts and stops associated with conventional PsD testing. This involves the use of a dynamic actuator and the improvement of servo-control so that the actuator velocity can be continuously adjusted. These tests still have the disadvantage of not being fully real time and consequently not accounting for strain-rate dependant effects. Figure 5-4 (Magonette, 2001) demonstrate the difference between the conventional and continuous PsD test methods.

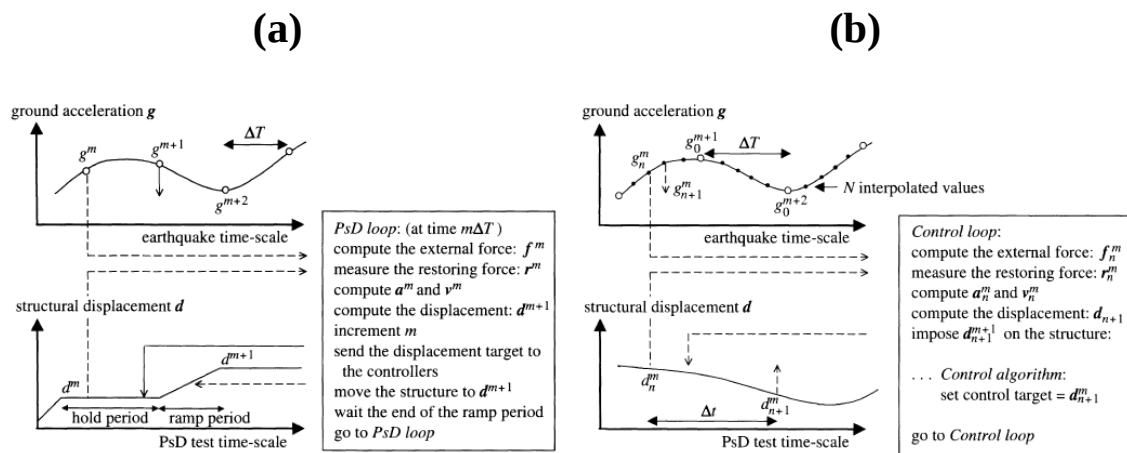


Figure 5-4 (a) Conventional and (b) continuous PsD tests, as illustrated by (Magonette, 2001)

The concept of substructuring in PsD testing was first introduced by Dermitzakis and Mahin (1985). This involved experimentally testing the critical parts of the structure where response is difficult to predict while simultaneously numerically modelling the remainder of the structure. The equation of motion for a substructured PsD test can be written with the restoring force split into two components as follows:

$$M\ddot{u}(t_{i+1}) + C\dot{u}(t_{i+1}) + R_{E,i+1} + R_{N,i+1} = P(t_{i+1}) \quad 5.2$$

Where $R_{E,i+1}$ is the restoring force vector for the physical substructure, which is measured experimentally, and $R_{N,i+1}$ is the restoring force vector for numerical substructure, which is calculated analytically.

Advances in computing power and testing hardware have allowed tests to be run in real time, where they are termed Real Time Hybrid Tests (RHT). A Real time hybrid test was first performed by Nakashima et al. (1992). These tests differ from the PsD method as the inertia and damping response of the physical substructure, as well as its stiffness, are measured during the test. RHTs are particularly useful for testing components whose response is velocity dependant, such as viscous dampers. Running a RHT requires the calculation of displacements from the numerical model to be performed rapidly. The equation describing a substructured RHT can be written as:

$$M\ddot{x}(t_{i+1}) + C\dot{x}(t_{i+1}) + R'_{E,i+1} + R_{N,i+1} = P(t_{i+1}) \quad 5.3$$

Where the term $R'_{E,i+1}$ differs from $R_{E,i+1}$ in Equation 5.2 as it includes the damping and velocity response as well as the stiffness of the experimental substructure.

Other recent advances in RHTs include running tests in combination with shake table tests, which allows some of the scaling issues associated with shake tables to be overcome and employing geographically distributed experimental substructures linked through a numerical model over the internet. The effective force testing (EFT) method is a slightly different concept developed to overcome issues with the PsD test that avoids the need for calculating real time structural response by physically modelling the full structural mass. For further details on these developments, and on the history of the hybrid test method in general, the reader is referred to McCrum and Williams (2016).

Much of the previous work in the field of hybrid testing focussed on developing the hybrid test method itself, however in recent years the technique is becoming more popular purely as a research tool in its own right (McCrumb and Williams, 2016). However, considerable expertise is still required to overcome some of the associated challenges in implementation.

5.5 OpenFresco

As explained, hybrid simulation involves the combination of a physical and numerical simulation. The Open Framework for Experimental Setup and Control, known as OpenFresco (Schellenberg et al., 2009a), is software developed to provide a link between a finite element model and laboratory test equipment, as illustrated in Figure 5-5. It allows a wide variety of hybrid simulation algorithms, laboratory and control systems, experimental setups, and computational simulation models to be combined for a specific hybrid simulation. Originally, the development of OpenFresco was strongly linked to the OpenSees computational framework, but newer versions can be combined with many different finite element packages.

For a local test, within OpenFresco, the user must define the experimental control, setup and element. The experimental control commands are used to communicate with different control and data acquisition systems in the laboratory. The experimental element object defines how the finite element software ‘views’ the experimental element(s) being tested and where it fits in to the overall structure in the finite element model. The experimental setup defines the actuator setup and how the coordinates are transformed between the experimental and numerical system. The experimental site also needs to be defined as some hybrid simulations are carried out in different locations over the internet, however this is of less importance for local tests. For a detailed description of the structure of the software the reader is referred to Schellenberg et al. (2009c).

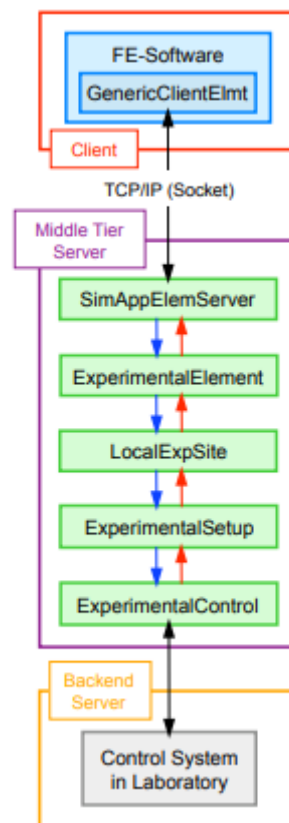


Figure 5-5 OpenFresco architecture (Schellenberg et al., 2009b)

5.6 Integration Schemes in Hybrid Simulation

The equations of motion that arise in hybrid simulation, and in structural dynamics in general, are generally solved through the application of numerical time-stepping integration algorithms. A vast number of methods have been developed for this purpose, however many of these are not particularly well suited to hybrid simulation. Thus, the following section provides a brief discussion on integration methods, focussing on their applicability for hybrid simulation.

Integration algorithms can be classified as either explicit, where the solution is dependent only on response values at previous steps, or implicit, where the new response value is based on current as well as previous response values. Explicit algorithms were favoured in early implementations of hybrid simulation for computational efficiency as they do not require iteration. However, traditionally employed explicit algorithms, such as the central difference method or explicit Newmark algorithms (Newmark, 1959), are only conditionally stable, meaning that the timestep has to be smaller than a critical value in order to yield a stable solution. As the maximum timestep is proportional to the highest natural frequency of the system, explicit methods are impractical for structures with very high natural frequencies, as the time step would become too small, and cannot be applied for structures with mass-less degrees of freedom, as this leads to infinitely stiff structures (Schellenberg et al., 2009c). More recently, some unconditionally stable explicit methods have been developed (Chang, 2007).

In contrast, implicit algorithms are generally unconditionally stable, making them ideal for stiff structures. However, they often require iteration to solve nonlinear equations of motion. This is undesirable for hybrid simulation because of the computational effort and the introduction of spurious loading cycles as part of the iteration which can damage the physical substructure (Schellenberg et al., 2009c). There are also issues with uncertainties in convergence of iterative schemes in nonlinear simulations involving numerical and experimental errors (Mosqueda and Ahmadizadeh, 2011).

Some of these issues with implicit algorithms can be alleviated by using a fixed number of iterations at each timestep. In implicit integration algorithms employed in regular numerical simulation the number of iterations at a step is dictated by a convergence test, with iteration continuing until the convergence criterion is satisfied, for example through the energy or displacement increment being less than a specified tolerance. However, with the fixed number of iterations modification the user specifies the number of iterations at each step, thus limiting the computational burden. Furthermore, the displacement increments are modified to make them more uniform. Lagrange interpolation, depending on the ratio of the current iteration number to the total number of iterations and using the trial displacements of the current iteration step and the committed displacements at number of previous timesteps, is used to calculate the applied displacement increment (Zhong, 2005). This gives displacement increments that are almost uniform and avoids the problems caused by spurious vibrations in regular iterative schemes.

Operator splitting techniques are unconditionally stable algorithms that do not require iteration. They are implicit for the linear part of the response and explicit for the nonlinear part, using an estimated tangent stiffness of the experimental substructure that negates the need for iteration. However, these methods do not deal well with geometric nonlinearities (Schellenberg et al., 2009c).

It is difficult to calculate the tangent stiffness matrix of the experimental substructure from the measurements at the controlled degrees of freedom, although it should be pointed out that algorithms do exist to do so (Thewalt and Roman, 1994). Hence, a so-called mixed stiffness matrix (K_m) approach is often taken in the solution of the equations of motion (Schellenberg et al., 2009c). As the structure begins to yield, the tangent stiffness matrix of the numerical substructure is used, as would normally be the case in a numerical analysis. However, the initial stiffness matrix of the experimental substructure is employed throughout the response calculation.

For further information on integration schemes and their applicability for different hybrid simulation scenarios, the reader is referred to Mosalam (2013) for a brief introduction to integration schemes for hybrid simulation in the OpenSees – OpenFresco environment, or Schellenberg et al. (2009c) who proved a detailed discussion of numerical integration in hybrid simulation.

5.7 Errors in Hybrid Testing

McCrum and Williams (2016) cite three main sources of error in hybrid testing; structural idealisations, approximations in numerical integration and experimental errors.

The main structural idealisations occur due to the use of lumped masses at the position of the actuators. This is necessary for the implementation of hybrid testing. Also, laboratory constraints typically mean testing is typically limited to SDOF or low MDOF systems. However, a model of sufficient accuracy can very often be obtained by selecting the dynamic degrees of freedom at locations where structural mass is actually concentrated. Therefore, the method is efficient and reliable for load carrying structures, such as multi-story buildings, with heavy floor systems at story levels (Mahin and Shing, 1985). McCrum and Williams (2016) also highlight the importance of realistically simulating the boundary conditions in substructured testing, particularly at the interface between the numerical and physical substructures.

Irrespective of the scheme used, numerical integration is, by its nature, only capable of providing an approximate solution. While the possible errors at each time step may be small, when

accumulated the overall error can be significant. Hence, it is necessary to insure that the dynamic response of the structural model can be accurately captured by the integration method and the integration time step selected. Similarly, errors can arise from the method by which structural damping is modelled within the solution can also introduce errors.

Experimental errors result from the displacement control of hydraulic actuators, force relaxation or strain rate effects due to the slow rates of testing, calibration errors in the instrumentation, and noise generated in the instrumentation and analogue to digital converters (Mosqueda et al., 2007). Experimental errors can be classified as either random or systematic. Systematic errors can have a greater influence on the simulation results compared to random errors (Mosqueda et al., 2005). Due to the cumulative nature of numerical integration, these can result in significant error growth. Sources of systematic errors include load-history effects on the experimental substructures and displacement control errors in the servo hydraulic actuators. Proper tuning of the actuator system is important to prevent such errors (McCrum and Williams, 2016).

Within the experimental control system a time lag in the order of 5 to 10ms exists between the sending the command to the actuator and achieving the desired displacement of the test specimen. This results in the restoring force being measured before the actuator has reached the target displacement. This is a significant issue for RTHT, where it has been the subject of much research (for example (Horiuchi et al., 1999, Darby et al., 2001)).

Shao and Griffith (2013) discussed the methods used to assess the accuracy of hybrid simulations. Ideally, the results of a hybrid simulation would be directly compared with those from a shake table test as was done by Elkhoraibi (2007); although, clearly, this is rarely possible as hybrid testing is often undertaken to avoid shake table tests. In general the most common verification method is to compare the results with those from numerical simulations. For steel structures the comparison can be either time history or force-displacement hysteresis response. It is stated that if the actuator can accurately apply the command displacement calculated through the numerical model, then the restoring force from the experimental substructure can be considered precise and can provide information for developing, calibrating and validating analytical models.

Other measures of error, including methods based on actuator tracking errors or input energy (Mosqueda et al., 2007) have also been used to evaluate the efficiency of actuator control strategies, particularly in RTHTs, and the reliability of hybrid test results (for example in Yang et al. (2009)).

5.8 Process Synchronization– Predictor Corrector

With modern testing equipment it is not untypical for the finite element model performing the time history analysis and the servo-controller to run at two different speeds. The speed of the finite element model is variable and depends on many factors, including the ground motion timestep, ground motion intensity, model complexity and available computational power. However, on the other hand modern servo-controllers run at constant sampling rates in the order of 1kHz and above. The problem can become particularly acute for geographically distributed tests where random delays due to latencies in the network also have to be considered. The solution to this discrepancy is the use of an intermediate predictor-corrector algorithm between the two processes.

These algorithms work by calculating a series of intermediate actuator target displacements which are applied to the experimental substructure while the equations of motion are being solved by the finite element software. Prior to receiving the next target displacement, extrapolation is used to predict the next displacement. Once the target displacement command is received interpolation begins and the actuator moves smoothly to reach the target displacement at the end of the timestep. This method ensures constant, smooth movement of the actuator.

Predictor-corrector techniques were first developed to allow the development of continuous PsD testing from conventional PsD testing, avoiding the ‘hold and ramp’ procedure as discussed previously. For example Nakashima and Masaoka (1999), implemented a RTHT by using Lagrangian polynomials for extrapolation and interpolation based on previous command displacements, as illustrated in Figure 5-6 (b) by Mosqueda et al. (2005). This algorithm was performed on a single digital processor, with computation time shared between solving the equations of motion and the predictor-corrector. Mosqueda et al. (2005) developed this by separating the two tasks onto two machines instead of one. Hence a so-called three-loop hardware architecture is employed, with the first loop running the time history analysis, the second running the predictor-corrector and the third running the servo-controller.

Adaptive, event driven strategies have been developed to further improve predictor-corrector algorithms. These use finite state models, where the system can exist in a number of different states with different events controlling the transition between states. As illustrated in Figure 5-6 (a) (Stojadinovic et al., 2006), the system moves between an extrapolate and interpolate state, as described previously. Delays are accounted for by transitioning to a ‘slow’ state, where the actuator movement is slowed down if no target displacement is received within an allotted time.

Longer delays result in a hold, and ultimately an unloading state, if no target displacement is received, due for example to non-convergence in the numerical model.

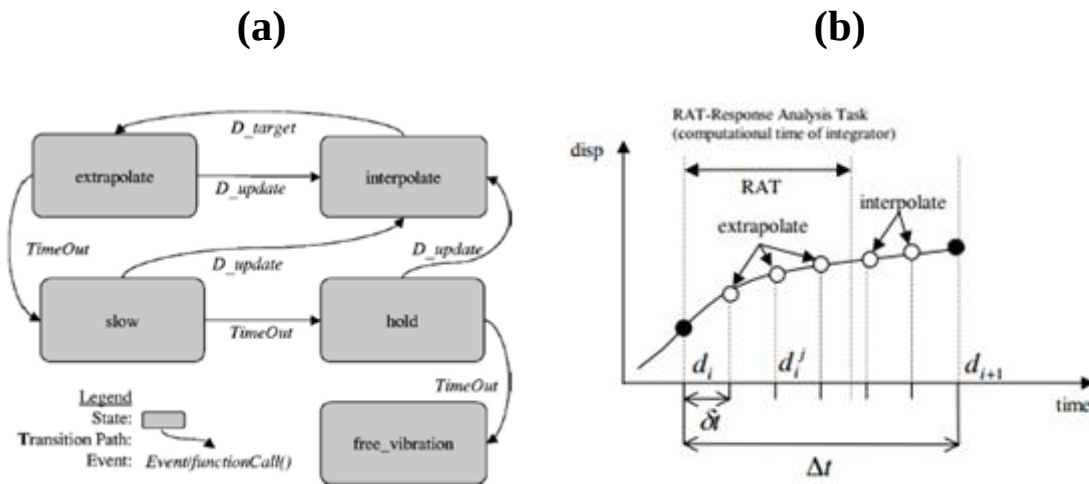


Figure 5-6 a) Event driven strategy for dealing with delays in hybrid simulation (Stojadinovic et al., 2006); b) Illustration (Mosqueda et al., 2005) of the extrapolation and interpolation of the command displacement signal during one integration step as proposed by Nakashima and Masaoka (1999)

Schellenberg et al. (2009c) developed the finite state event driven strategy to include a continuous change of the actuator velocity in the 'Slow Down' state. This avoids the sudden change of velocity that occurred in older systems. As the slow down state reduces the actuator velocity smoothly all the way to zero, the hold state is superfluous and therefore removed.

5.9 Online Model Updating in Hybrid Simulation

Efforts have been made to improve the accuracy of hybrid simulations through the use of online model updating techniques. Online model updating involves constantly optimizing parameters in the numerical model based on data measured during the course of experiment.

The concept of sub-structuring in hybrid simulation is based on the assumption that the numerical model accurately captures the response of the numerical substructure. However, as with all models, this is not necessarily true and during hybrid simulation a wealth of data becomes available from measured experimental response that can be used to improve the numerical model. This is particularly the case if components with similar structural and material properties are being tested experimentally and modelled numerically. Hence, braced frames,

where often a just single bracing member out of many is physically tested, are ideally suited to model updating algorithms.

One of the earliest attempts at model updating was made by Yang and Nakano (2005) who employed a neural network for the numerical substructure and updated the network based on online experimental data.

Using more conventional structural modelling techniques, Wang et al. (2011) adopted the least-square method to optimize the parameters of a bilinear hysteretic material model for a buckling-restrained brace. Kwon and Kammula (2013) developed an updating strategy, illustrated in Figures 5-7 and 5-8, in which the resisting force of the numerical substructure was defined by a weighted average of several numerical models with different parameters, referred to as numerical counterparts. The weighting factors assigned to each counterpart were calculated based on the resisting force measured from the experimental test structure, in such a way that the counterpart whose properties are closest to the experimental specimen carries the highest weight. The main disadvantage of the method is that if many parameters affect the hysteretic response, many alternative numerical counterparts need to be employed which increases the computational burden. The algorithm was implemented in UI SimCor (Kwon, 2011) and applied successfully to a hybrid simulation of a braced frame structure.

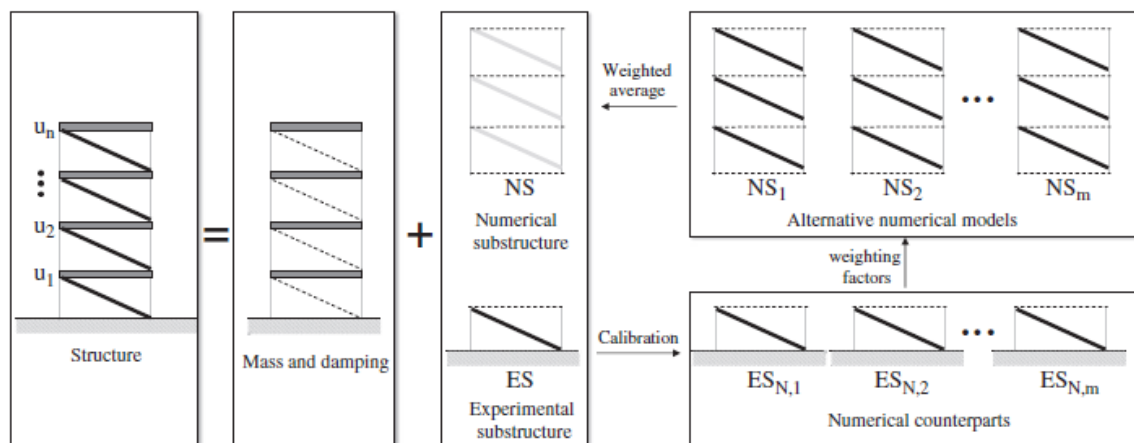
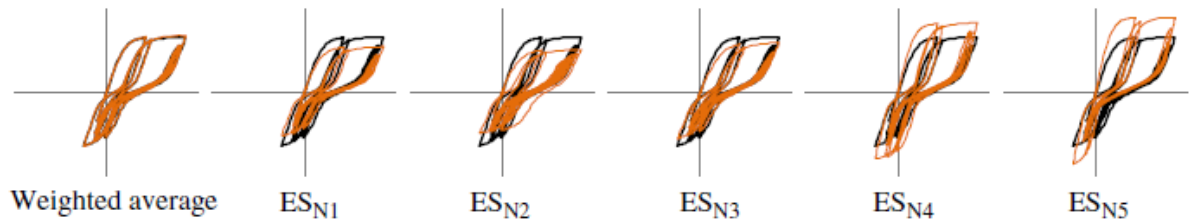


Figure 5-7 Setup for model updating using weighted average of numerical counterparts as implemented by Kwon and Kammula (2013)



Note: Thick line is the response of experimental substructure and light line is from numerical counterpart.

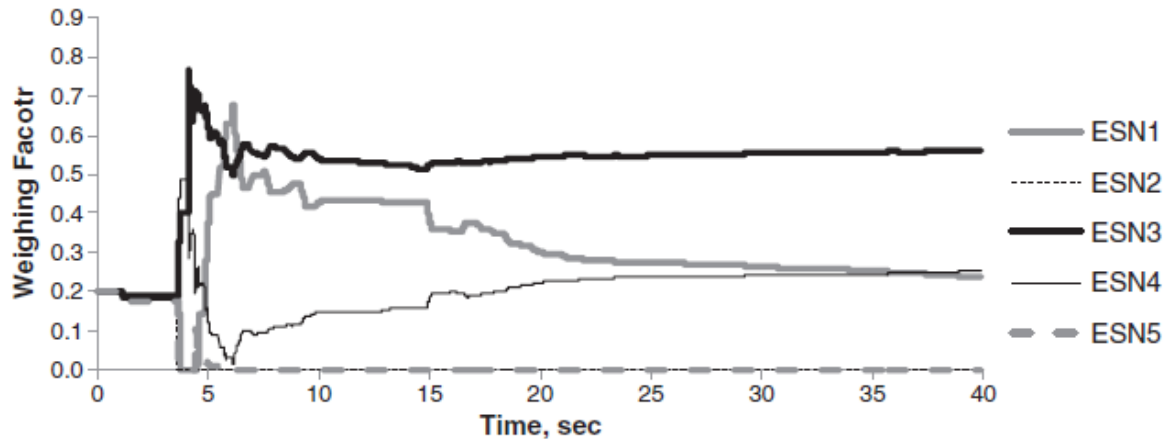


Figure 5-8 Illustration of how a weighted average of numerical counterpart responses can be used to match experimental response (Kwon and Kammula, 2013). For the majority of the simulation the highest weighting is given to counterpart 3 followed by counterpart 1, which generally have the closest match to the numerical response

Other studies (Yang et al., 2012, Elanwar and Elnashai, 2014) have used the simplex method and genetic algorithm approaches for parameter optimization in bridge hybrid simulation. In recent years it has become common to use the unscented Kalman filter (UKF) for updating. The basic idea of a Kalman filter is to estimate the true state of a system by comparing predicted and measured values. The UKF is a development of the Kalman filter for highly nonlinear applications. Song and Dyke (2013) and Hashemi et al. (2014) both provide detailed explanations of the theory behind the UKF and how it can be used for optimization in online model updating. These studies, as well as those by Wu and Wang (2014) and Wu et al. (2016), examined single-storey single-bay frames and used MATLAB for both structural modelling and model updating. Recently, Wu et al. (2018) have carried out model updating for a hybrid simulation using OpenSees as the computational driver with a UKF updating procedure implemented in Matlab, as illustrated in Figure 5-9.

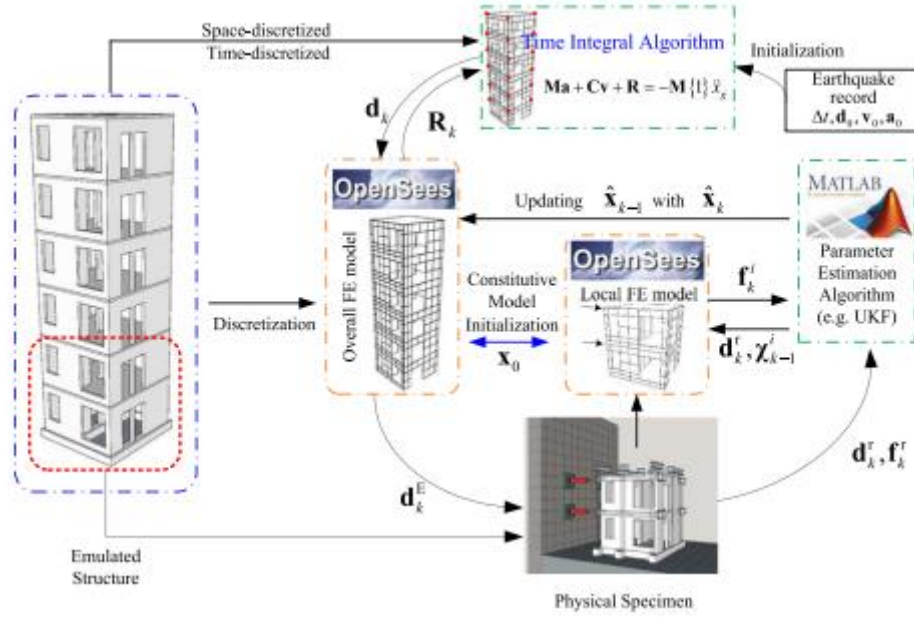


Figure 5-9 Illustration of the model updating algorithm implemented by Wu et al. (2018)

5.10 Hybrid Simulation Facility at Trinity College Dublin

The hybrid testing facility at Trinity College Dublin comprises of an MTS real-time hybrid test system. The hardware consists of a Series 111 MTS Accumulator and a 150 kN capacity high speed linear hydraulic actuator with a 250 mm (± 125 mm) stroke. The setup is illustrated in Figure 5-10 (McCrum, 2012). The numerical model is run by the Simulation and Target PCs, while the actuator and servo-controller, i.e. the Structural Test System (STS), are controlled via the Test PC. Communication between the experimental and numerical systems is achieved using SCRAMNet (Shared Common Random Access Memory Network) shared reflective memory, which allows information to be shared rapidly in both directions.

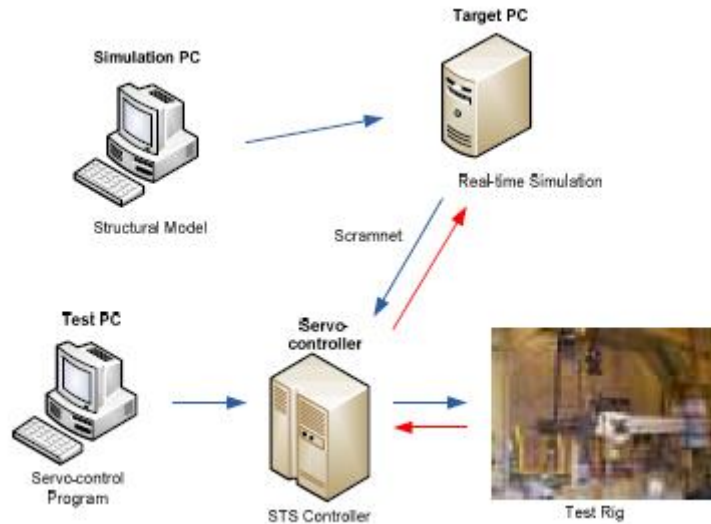


Figure 5-10 Hybrid test set up at Trinity College Dublin (McCrum, 2012)

Previously, so-called soft-RTHTs have been carried out using the facility (McCrum, 2012). These are essentially PsD tests with an extended timescale. They were carried out by experimentally testing a single storey X-braced frame and numerically modelling the remainder of a three storey frame, as illustrated in Figure 5-11 (McCrum and Broderick, 2013).

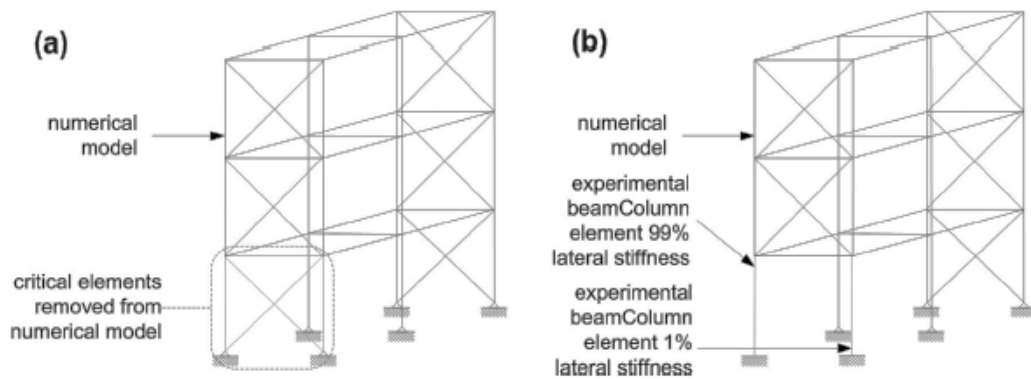


Figure 5-11 Previous hybrid simulation at Trinity College Dublin (McCrum and Broderick, 2013)

These tests were performed using OpenSees as the finite element software and OpenFresco as the middleware between the numerical and experimental models. The predictor-corrector algorithm developed by Schellenberg et al. (2009c) was used to synchronise the numerical integration with the servo-controller. The X-braced frame experimental substructure was represented in OpenFresco using two experimental beamColumn elements. 99% of the lateral stiffness of the frame was allocated to one of the elements with the remaining 1% allocated to the other. This approach allowed a complex physical substructure to be simplified in a manner that permitted testing with a single actuator. Comparisons between the experimental results

and numerical simulations showed that this substructuring method worked successfully (McCrum and Broderick, 2013). A similar test setup was employed for testing of a Moment Resisting Frame structure in an industry partnership.

Other examples of previous hybrid simulations of braced frame structures elsewhere include those by Yang et al. (2009), Lin et al. (2012), and Tsai et al. (2013). Yang et al. (2009) tested a zipper braced frame, with the experimental substructure tested at 1/3 scale. As can be seen in Figure 5-12, 3 actuators were used to control displacements in the two translational and rotational degrees of freedom. This configuration is now included as one of the standard experimental setups in OpenFresco, called InvertedVBrace. OpenSees was used as the finite element software, OpenFresco as the middleware and the event driven strategy for synchronization of the computational driver and experimental control system, as in the tests in TCD. This was the first recorded test where nonlinear behaviour was encountered in both the numerical, as well as the experimental, substructure. Good agreement with computer only numerical models was achieved and it was concluded that the system produced reliable results and that the system was suitable to study the response of other complex structural systems.

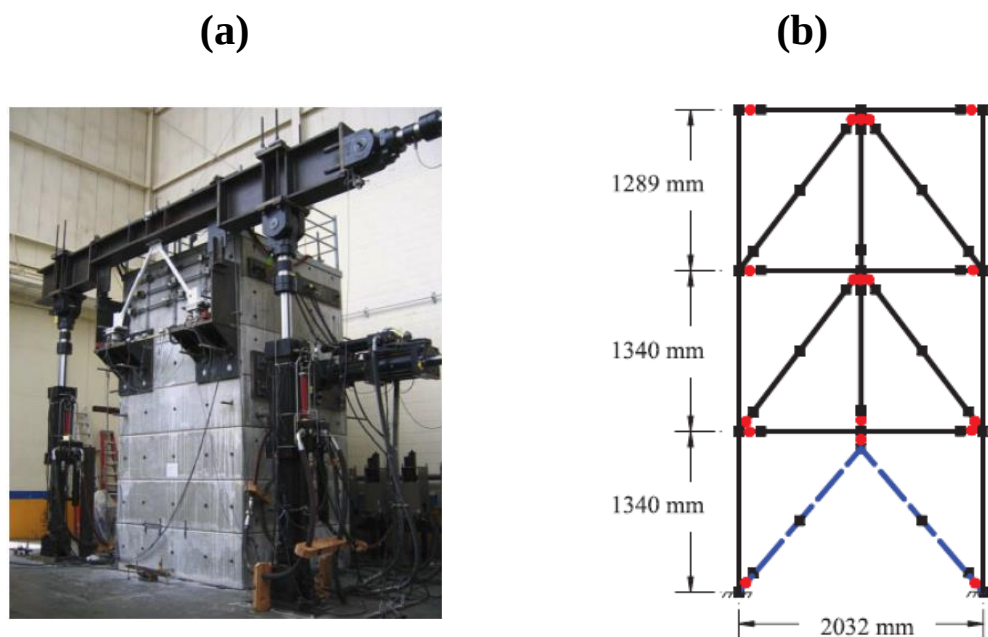


Figure 5-12 Hybrid simulation setup employed by Yang et al. (2009) showing (a) the experimental substructure and (b) the numerical model

A series of tests on braced frames using either buckling restrained braces (Lin et al., 2012) or in-plane buckling braces (Tsai et al., 2013) were performed as part of a Taiwan-US collaborative project entitled the “NEES-SR SG International Hybrid Simulation of Tomorrow’s Braced Frames”. The same basic setup was used in both studies, with actuators used to apply horizontal

displacements to a three storey experimental substructure, as illustrated in Figure 5-13. Again, OpenSees was used as the computational driver and OpenFresco as the middleware between it and the experimental setup. A generic element was used to represent the experimental substructure in OpenFresco which was then connected to a numerical leaning column element to represent the gravity loads on non-seismic elements not included in the model, as illustrated in Figure 5-13(c).

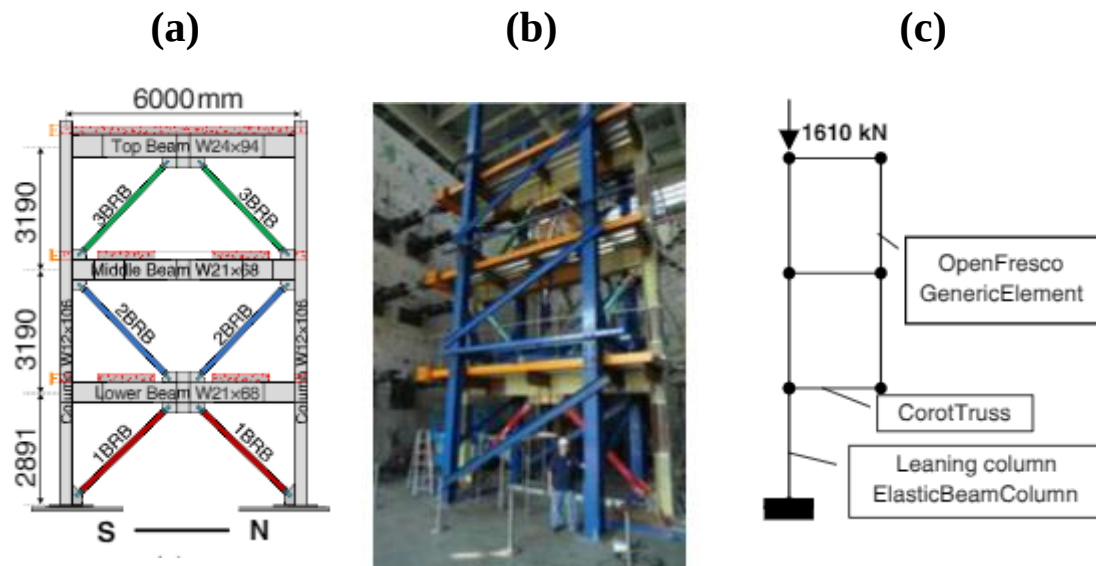


Figure 5-13 (a) Schematic and (b) photograph of experimental substructure and (c) numerical model using the GenericElement approach employed by Lin et al. (2012) for hybrid simulation of a 3 storey frame with buckling restrained braces

5.11 Conclusions & Areas for Further Research

Numerical models that can accurately predict response to seismic excitation are central in the move towards performance based earthquake engineering, discussed in Chapter 4. However, The response of CBFs to large earthquakes is highly nonlinear and challenging to capture correctly in a purely numerical model (for example see Ryan et al. (2017)). By alleviating the need to model some of the more complex components, substructured hybrid simulation can be a valuable tool for investigating the seismic performance of CBFs, while the experimental data obtained can be used to refine numerical models and improve the ability to predict structural response. Therefore, the experimental test facilities available at TCD provide the opportunity to use hybrid simulation to develop, validate and improve analytical models of CBFs.

A brief review of literature detailing the development and implementation of hybrid simulation as an experimental testing method was presented. A large amount of work has been carried out to develop hybrid simulation as a testing method to the point that it is now being employed as a

research tool in its own right. Recently, researchers have focussed on the implementation of model updating techniques that use measured experimental data to improve the numerical models used as part of hybrid simulations.

Therefore, a goal of this research is to perform a series of hybrid simulations that allow the response of CBFs to earthquakes of various intensities to be examined. This work will involve developing the previous implementations of hybrid simulation at TCD. The application of model updating techniques will allow more robust hybrid simulations to be performed and provide further opportunity to refine numerical models and learn about optimum modelling parameters. A further challenge within this is to implement model updating solely within the OpenSees-OpenFresco environment. This will make the full nonlinear analysis capabilities of OpenSees available for use in online model updating and, on a more practical level, will make model updating more accessible for research engineers accustomed to performing numerical analysis in OpenSees. The results of hybrid simulations, with and without updating, will allow numerical models used elsewhere in this work to be evaluated and potentially improved.

6 Lifetime Performance Assessment of CBF Structures

6.1 Introduction

As discussed in Chapter 4, there is a need to develop greater understanding how decisions taken at the design stage impact upon the performance of CBF structures, and as design moves towards performance based methodologies, to examine whether the structural engineer can select design parameters to improve lifetime performance, within the constraints of conventional seismic design codes.

An important trend highlighted in many recent performance assessment studies is that losses resulting from damage to acceleration-sensitive non-structural components tend to be the majority contributor to lifetime repair costs. CBFs are inherently stiff structures, a characteristic that reduces drifts but can lead to relatively high acceleration demands, even during low intensity earthquakes. Hence, the prominence of acceleration related costs in lifetime losses highlighted by these studies is particularly relevant for CBFs, which because of their tendency to experience higher accelerations, may be prone to excessive life costs. However, it is possible that by altering the frame stiffness, the trade-off between drift and acceleration can be improved to reduce lifetime losses. Within the confines of the Eurocode 8 design rules for CBFs, stiffness can be systematically controlled by changing the design behaviour factor, once the material strength is kept constant. Lower q values lead to stiffer frames which may be vulnerable to excessive acceleration demands. In contrast, designs using higher q values lead to more flexible structures, where large drifts are of greater concern. Therefore by altering q the structural engineer can attempt to coordinate drift and acceleration damage control. Thus, the aim of this Chapter is to examine how the choice of behaviour factor affects the lifetime seismic performance of CBF structures designed to Eurocode 8.

Performance is assessed by using *EDPs* obtained from NLTHA to evaluate the expected lifetime losses, calculated using the FEMA P-58 methodology, for a set of case study CBFs. Two, five and ten storey structures are examined, while a number of moment resisting frames designed to Eurocode 8 are also assessed for comparative purposes. The frames are designed using behaviour factors ranging from 1 to 5, meaning designs ranging from non-dissipative to highly-dissipative, and high to low stiffness, are investigated. Performance is evaluated for sites with both medium and high seismic hazards, using ground motion intensities with probabilities of exceedance between 95% in 50 years and 2% in 50 years, meaning structural response is examined for a wide range of potential earthquake scenarios. The calculated performance metrics are then analysed to assess the contribution of damage to different building

components to overall losses. Thus a comprehensive picture of low and mid-rise CBF lifetime performance is obtained. This allows the key trends to be examined, and provides potentially useful information for design to allow lifetime losses to be minimised.

The chapter begins by presenting the steps within the performance assessment procedure employed, before the design and structural modelling of case study buildings is discussed. The *EDPs* obtained from NLTHA, and the resulting losses, are then shown. The implications of these results are then discussed, with a focus on how the trends in the calculated performance metrics impact design, particularly as design moves towards performance based design methods.

6.2 Procedure

Figure 6-1 outlines the procedure adopted to assess the performance of the case study buildings. In short, case study buildings are designed to EC8, NLTHA is carried out in OpenSees and the *EDPs* obtained are used to calculate performance metrics in PACT. Appendix A gives a detailed step-by-step breakdown of all the steps followed in the performance assessment procedure employed.

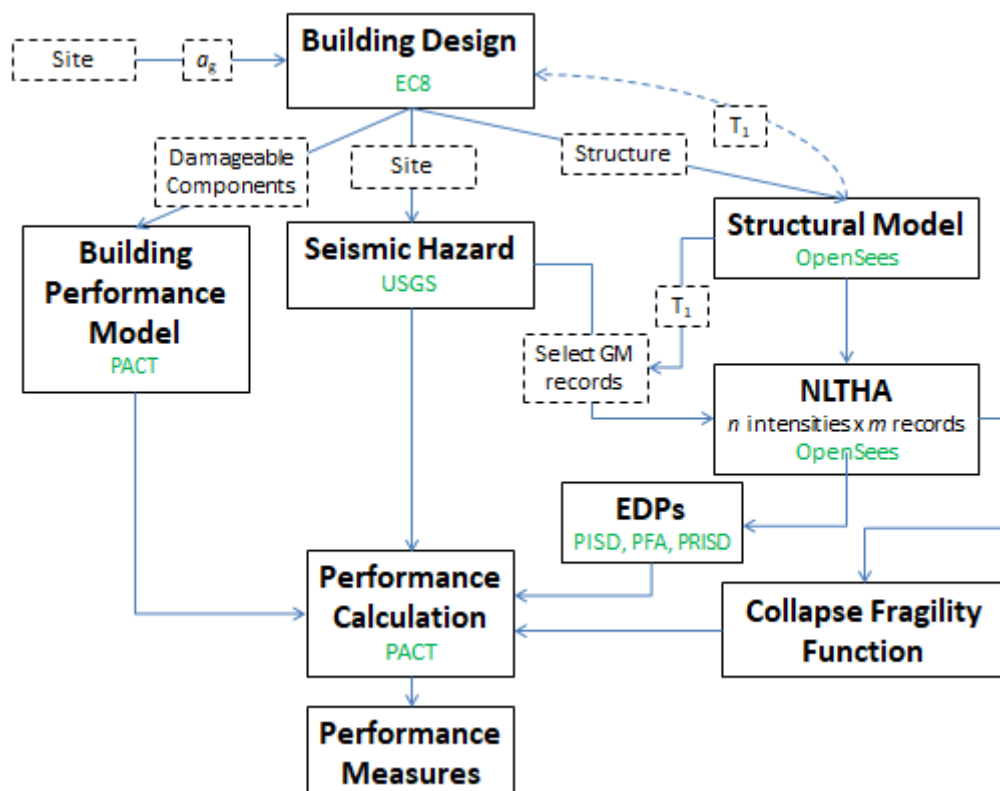


Figure 6-1 Illustration of the procedure followed for performance assessment

6.2.1 Case Study Buildings

For this study, two sites, one in Oakland, California (37.803°, -122.287°) and one in Seattle, Washington (47.609, -122.334°) were selected as case study sites. The site in Oakland, which has been used in a number of previous studies (Baker et al., 2011, Terzic et al., 2014), has a severe seismic hazard, whereas the design hazard at the Seattle is in line with more moderate values found in Europe.

A set of two, five and ten storey CBFs as illustrated in Figure 6-2 and detailed in Table 6-1, were designed in accordance with Eurocode 3 (CEN, 2005) and Eurocode 8 using behaviour factors between 1 and 5. Buildings were designed to be square in plan with seismic resistance provided by perimeter braced frames, as illustrated in Figure 6-3 for the 5 storey CBFs. For each frame, bay widths and storey heights were kept constant at 6m and 3.6m respectively, except in the 10 storey frames where braced bay widths were halved, as can be seen in Figure 6-2. The frames were initially designed for gravity loading according to Eurocode 3, with unfactored dead and live loads of 3kN/m² and 3.3kN/m² applied at each floor level and 2.6kN/m² and 0.6kN/m² at the roof level.

Subsequently, seismic design was carried out in accordance with Eurocode 8, assuming Type C ground conditions at both sites. The USGS hazard application (United States Geological Survey, 2014) was used to obtain the 10% in 50 year peak ground acceleration for Type A ground at the sites, 0.5g in Oakland and 0.3g in Seattle, and these values were used as the reference peak ground acceleration, a_g , values to compute the Eurocode 8 design spectrum. The Type 1 design spectrum, illustrated in Figure 6-4, was used for both sites given the magnitudes of the earthquakes that contribute most to the seismic hazard.

Selecting a case study site in the USA allowed state-of-the-art ground motion selection procedures based around the Conditional Spectrum (CS) to be employed. As explained in Chapter 4, the CS is calculated using the site-specific hazard deaggregation information. Such information is not freely available for European sites. Therefore, case study sites were chosen in the USA, where this information can be obtained using the USGS online tool (United States Geological Survey, 2008). However, given that the designs were carried out using the Eurocode 8, the use of sites in the USA introduces questions about the applicability of the design spectrum. Figure 6-4 compares the Eurocode 8 elastic spectrum and the 10% in 50-year Uniform Hazard Spectrum (UHS) obtained from the USGS hazard application (United States Geological Survey, 2014) for the two case-study sites. Clearly, there is some incompatibility between the spectra, particularly for mid-range periods of about 0.5 to 1s. However, this tendency of the

Eurocode 8 Type 1 elastic spectrum to diverge from the 10% in 50-year UHS is also be seen at European sites. The Type 1 spectrum is anchored to earthquakes of approximately 7; once the causal magnitude moves away from this, differences with the UHS become apparent (Bommer and Pinho, 2006, Bommer and Stafford, 2009). Therefore, it can be concluded that even though the Eurocode 8 spectrum appears to give a poor representation of the seismic hazard at a site in the USA, an equally poor match could also be obtained for European sites and hence the scenarios considered are not unrealistic.

The design actions to be resisted by the structural members were calculated through elastic analyses using the Eurocode 8 lateral force method for regular structures. For the CBFs the lateral resistance of the structure was assumed to be provided by the tension diagonals only. The bracing members, which were either rectangular or square hollow sections, were designed to resist axial forces due to the seismic design actions. The Eurocode 8 non-dimensional slenderness limitations and overstrength differential criteria were obeyed in the brace design procedure.

The purpose of halving the bay widths for the 10-storey frames is to reduce the brace length, which in turn reduces brace slenderness. This allows braces with lower cross-sectional areas to fulfil the Eurocode 8 non-dimensional slenderness limitations, meaning braces with lower overstrength values can be selected. This helps prevent the situation where, in order to fulfil the slenderness limitations, excessively large braces with high overstrength values are required at the upper storeys where seismic design forces are low. The overstrength differential criteria require the use of braces with similarly large overstrength values at all levels, meaning that towards the base of the structure, where design forces are high, unfeasibly large brace sections are needed to achieve such high overstrength values. The potential for the slenderness and overstrength limitations to combine in such a way and lead to inefficient design has been highlighted before by Elghazouli (2008) and Brandanissio et. al (2012) amongst others. In this study, such effects were found to be particularly important for the 10 storey frames designed with low behaviour factors, therefore the bay width was reduced for these frames.

Based on the selected brace properties, columns and beams were capacity designed, using UK steel profiles, to resist the combination of gravity and seismic actions stipulated by Eurocode 8, with member sizes kept constant at each floor level.

The majority of structural members were designed with characteristic yield strength of 355 N/mm², however in a number of cases with lower behaviour factors it was necessary to employ

a yield strength of 460 N/mm² in non-dissipative members to fulfil capacity design criteria whilst maintaining a constant structural configuration. The EC8 deformation checks for second order effects and damage limitation requirements were also performed as part of the design process, but for CBFs, as is often the case due to their high stiffness, these did not impact any of the designs. Details of the structural members selected in each design are given in Appendix B.

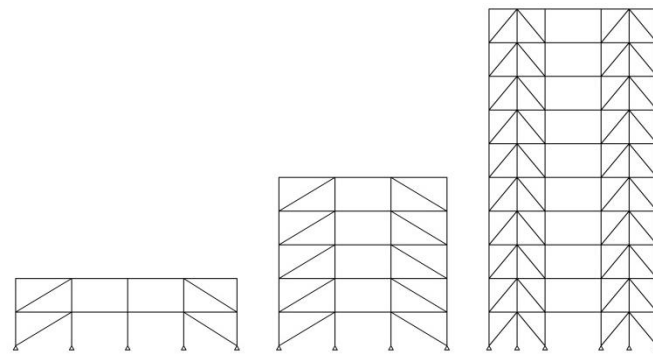


Figure 6-2 Elevations of case study CBFs

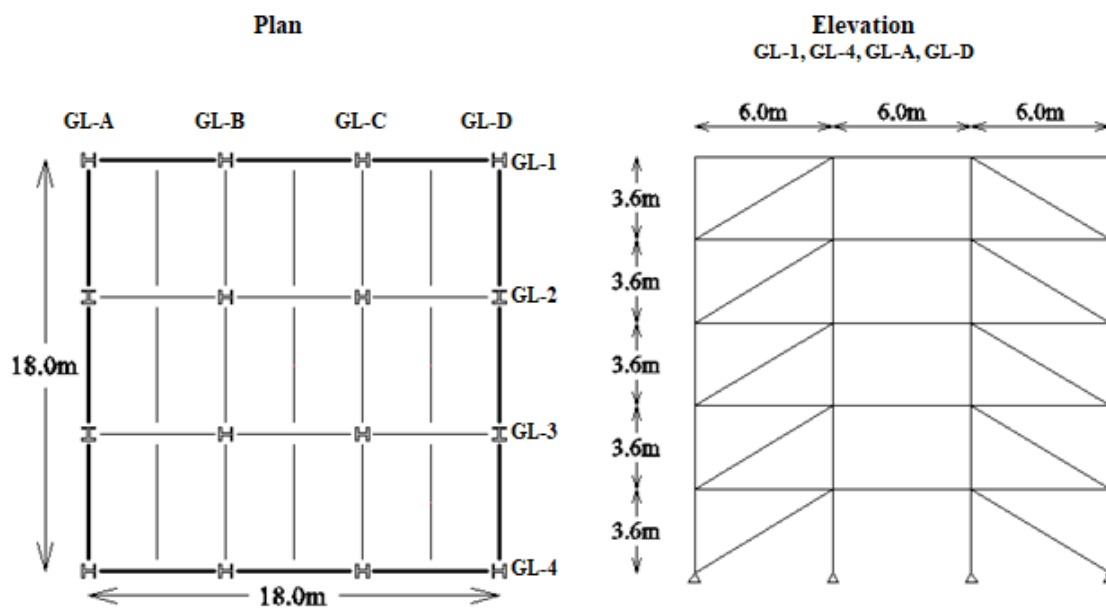


Figure 6-3 Plan and Elevation of braced gridlines (GL-1, GL-4, GL-A and GL-D) for the 5 storey case study CBFs

Table 6-1 Details of case study frames; showing the influence of the behaviour factor, q , on the mass of steel required and fundamental structural period, T_1

No. of Stories	q	Oakland Site, $a_g = 0.5g$		Seattle Site, $a_g = 0.3g$	
		Mass of Steel (Mg)	T_1 (s)	Mass of Steel (Mg)	T_1 (s)
2	1	91.4	0.12	70.2	0.15
	2	70.6	0.16	41.2	0.22
	3	58.8	0.20	38.0	0.25
	4	44.8	0.23	35.8	0.28
	5	41.2	0.24	35.5	0.29
5	1	223.7	0.22	131.9	0.27
	2	170.5	0.26	113.5	0.32
	3	138.5	0.31	87.0	0.38
	4	114.3	0.35	64.9	0.44
	5	98.4	0.36	64.7	0.45
10	2	376.4	0.51	295.1	0.59
	3	308.0	0.60	222.2	0.71
	4	241.2	0.70	178.5	0.82

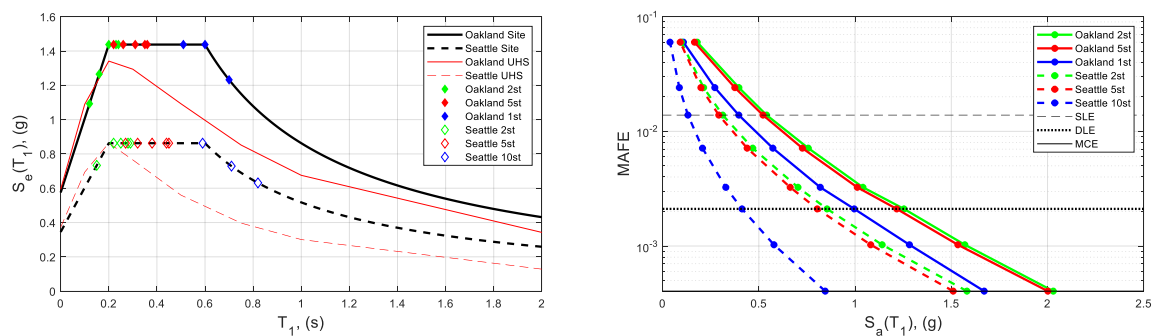


Figure 6-4 a) EC8 Type 1 elastic spectrum, S_e , and Uniform Hazard Spectrum for the two sites with fundamental period of case study CBFs marked and b) the seismic hazard curve showing Mean Annual Frequency of Exceedance (MAFE) of ground motion intensity, measured by spectral acceleration at the fundamental period $S_a(T_1)$, for each of the sets of CBFs. Intensity levels where analysis is performed are marked with dots, showing a reasonably even distribution across the hazard curve.

6.2.2 Structural Modelling

To carry out NLTHA-based response simulations the frames were modelled in OpenSees (McKenna, 2011). Individual planar models were created for both primary response directions, with one component of the ground motion pair applied to the model representing one direction and the orthogonal component applied to the other model, in accordance with the FEMA P-58 recommendations (FEMA, 2012b). This meant two separate two-dimensional OpenSees models were developed in order to predict $EDPs$ in the two primary response directions. However, as the frames are square on plan these two models were very similar, with the orientation of the corner columns being the only difference between the two. One of the two components of the earthquake ground motion was applied to the first model, with the orthogonal component being

applied to the second in a separate NLTHA. No rotation of ground motion records was performed. The potential for increased demand in a direction other than the two primary response directions is considered by including a 'non-dimensional conversion factor' of 1.2 in the loss analysis procedure, as recommended by FEMA-P58 (FEMA, 2012b). This factor sets the value of an *EDP* used in loss analysis as a multiple of the maximum value in either of the two directions analysed.

As discussed in Chapter 3, the use of line-element physical theory models to capture the hysteretic behaviour of CBFs in OpenSees has been the subject of much research (for example Ryan et al. (2017)). Following the recommendations of Uriz et al. (2008), the bracing elements in the CBFs were modelled using two force-based nonlinearBeamColumn elements with three integration points per element. An initial camber displacement of 0.1% of the brace length was employed at the midpoint in order to simulate buckling behaviour. This value for initial camber has been employed in a number of different studies examining CBF response (Karamanci and Lignos, 2014, Hwang and Lignos, 2017a, Del Gobbo et al., 2018). The brace elements were discretised as illustrated in Figure 6-6(a), using 12 layers across the depth of the cross section. Nonlinear rotational spring elements, with initial rotational stiffness and yield moment calculated according to the expressions proposed by Hsiao et al. (2012), were employed at the brace end point to simulate the end restraint imposed on the braces by the gusset plates. Rigid elements were used to model the remainder of the gusset plates and the sections of the beams and columns in the region of the connection, as illustrated in Figure 6-6(b). For both frame types the beams and columns were also modelled using force-based nonlinearBeamColumn elements with 3 integration points and discretized fiber sections. Distributed plasticity elements, as opposed to concentrated plasticity elements which can capture cyclic degradation, were used to model the beams and columns as they are able to capture some behaviours, particularly axial force bending moment interaction, more fundamentally than concentrated plasticity elements. The Steel02 material, which represents the Giuffre-Menegotto-Pinto model, was used for all nonlinear components. Seismic masses were lumped at the nodes. Rigid truss elements were used to ensure the nodes at each floor level moved together horizontally, modelling the impact of the floor diaphragm on response. As illustrated in Figure 6-5, a fictitious leaning column element (Geschwindner, 2002) was included in order to account for second order effects due to the gravity loads from parts of the building not included in the OpenSees models. This was connected to the model using rigid truss elements. The co-rotational coordinate transformation was used to allow second order effects to be considered. For all analysis 5% Rayleigh damping at the first and third modes was applied. The value of the Rayleigh coefficients were

calculated using the mass and tangent stiffness matrices (Charney, 2008), however as discussed by Chopra and McKenna (2016) the method of calculating the coefficients is not particularly influential for structural models employing distributed plasticity elements, unlike models using concentrated plasticity elements. Appendix D provides an example of the OpenSees code used for a typical model.

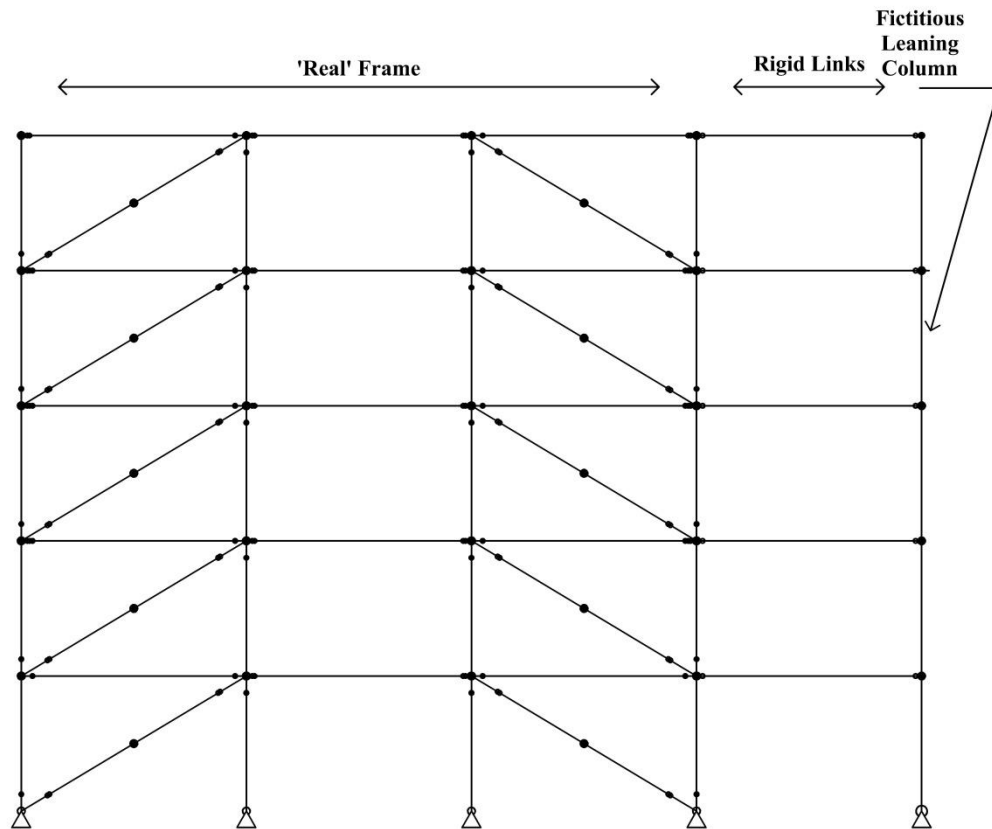


Figure 6-5 OpenSees Model of 5 storey CBF

(a)

(b)

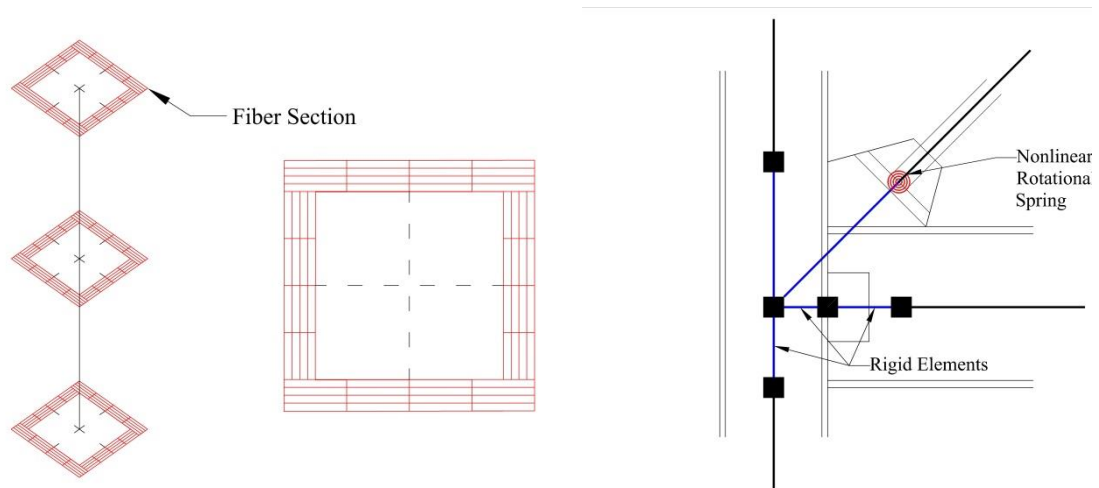


Figure 6-6 Discretized fiber section distributed plasticity bracing element and gusset plate model

6.2.3 Selection of Ground Motion Records for NLTHA

NLTHA was performed at 8 intensity levels using 10 two-component ground motion records. To select ground motion records for NLTHA a Multiple Stripe Analysis (MSA) approach was adopted. As discussed in Chapter 4, MSA involves selecting different sets of ground motion records at each ground motion intensity level considered (Baker, 2013). This is in contrast to more frequently employed Incremental Dynamic Analysis (IDA) procedures (Vamvatsikos and Cornell, 2002), where the same set of progressively scaled records is used for all intensities. The MSA approach captures the variations in expected characteristics of ground motions at different intensities ensuring that records selected, and consequently the calculated structural response, are hazard consistent.

In order to employ MSA for this study, ground motion records for NLTHA were selected to match the conditional spectrum (CS) (Baker, 2010) based on the recommendations of Lin et al. (2013c) discussed in Chapter 4. Ideally, each frame would be analysed using ground motion records selected to match a spectrum conditioned on its own fundamental period, implying the use of different sets of records for each frame. However, as mentioned in Chapter 4, it has been shown (Lin et al., 2013c) that when using the CS, probabilistically weighted structural analysis results are relatively insensitive to the choice of conditioning period. It was therefore considered acceptable to use the mean fundamental period for each frame height at each site as the conditioning period for developing the target spectrum. This ensured that frames designed with different behaviour factors were subjected to the same set of ground motion records, allowing the influence of the behaviour factor to be fairly assessed.

Alternatively, the recently proposed average spectral acceleration method could have been used to develop the CS. Generally, as discussed, conditional spectra are calculated using just a single fundamental period, T^* . However, a method for calculating the CS based on multiple periods, as opposed to a single period, has recently been proposed by Kohrangi et al (2017). This is based on the average of spectral acceleration values at a number of periods of interest and could have been employed here to develop the conditional spectrum considering the spectral ordinates at the fundamental periods of each structure under examination. This differs from the approach adopted here, where the mean of the fundamental periods of the group of structures being considered was chosen as a suitable T^* value for a traditional single value based CS.

The procedure for selecting ground motion records first involved seismic hazard deaggregation, which was performed for the case study sites using the USGS tool (United States Geological Survey, 2008). Then, using the results from this, MATLAB code, available online (https://web.stanford.edu/~bakerjw/gm_selection.html) (Jayaram et al., 2011a) was used to calculate the appropriate CS and scale and select appropriate two-component ground motion records from the PEER NGA database (Chiou et al., 2008). Near-field records often contain pulses which are well known to impose higher demands on structures compared to ordinary records. No pulse-like records were included in the analysis. As an example, Figure 6-7 shows the ground motion records selected for the 2% and 10% in 50 year probability of exceedance intensity levels for the 5 storey CBFs at the Oakland site. Appendix C provides details of the ground motion records selected for each analysis.

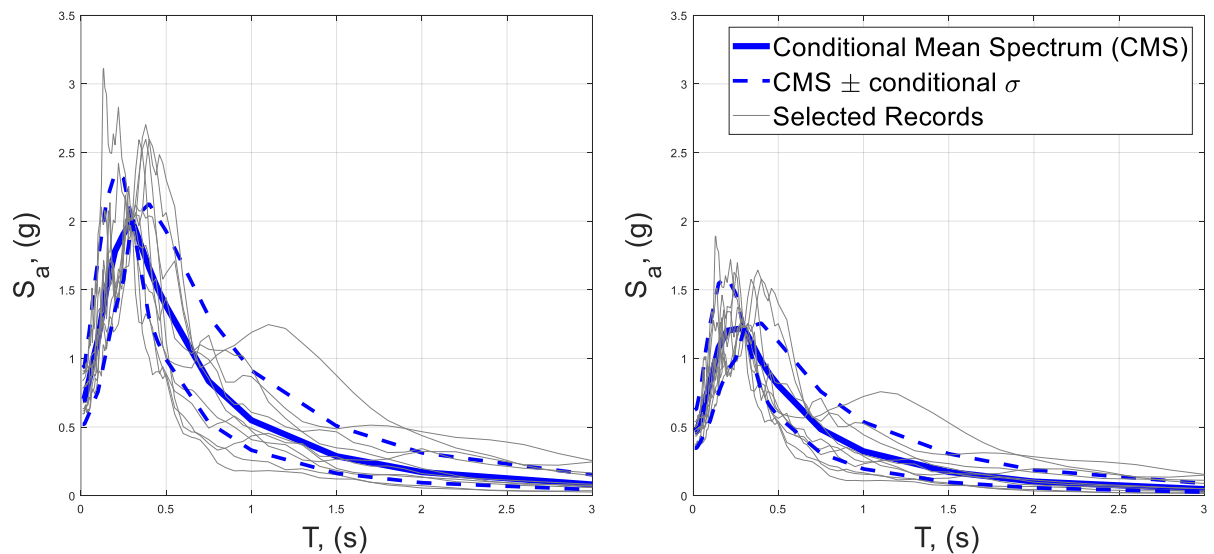


Figure 6-7 Examples of ground motion records selected to match the conditional spectrum (CS) at the maximum credible and design level earthquake intensities (2% and 10% probability of exceedance in 50 years respectively) for the 5 storey CBFs at the Oakland site

6.2.3.1 Number of Intensity Levels and Ground Motion Records

A sensitivity analysis was performed to establish the number of intensity levels, and ground motion records at each intensity level, necessary to accurately calculate lifetime performance metrics. For the two storey CBF at the Oakland site designed using $q = 4$, analysis was performed using 40 ground motion records selected to match the CS at 14 different ground motion intensities. The performance metrics of interest, namely expected annual repair costs and downtime, calculated from these structural analyses using PACT were taken to represent the 'exact' values of these metrics. The performance assessment calculation was then repeated using reduced numbers of ground motion records and intensity levels and the observed performance metrics compared with their 'exact' values. Through this process it was demonstrated that performing analysis at 8 different ground motion intensity levels, using 10 ground motion records at each, allowed both lifetime and intensity-specific performance metrics to be calculated with reasonable accuracy, whilst significantly reducing computational demand. Therefore, for each case study frame, NLTHA was performed using 10 ground motion records at intensity levels with 2% (MCE), 5%, 10% (DLE), 15%, 30%, 50% (SLE), 70% and 95% probabilities of exceedance in 50 years. This combination gives a reasonably even distribution of intensity levels across the range of hazard curve considered in performance assessment, as can be seen from Figure 6-4(b).

To develop collapse fragility functions necessary for performance assessment, in some cases it was necessary to perform further analysis at higher intensity levels. In these cases, records were selected to match the CS at 1% and 0.5% probabilities of exceedance in 50 year intensity levels. If further intensity levels were required the records selected at the 0.5% in 50 year intensity level were incrementally scaled in steps of 0.5g. This IDA type approach was adopted at this point because deaggregation results are not available for the very high intensity levels above 0.5% in 50 years (United States Geological Survey, 2008) and consequently the CS cannot be calculated. It should be noted that the sole purpose of analyses at intensities above the MCE was to allow collapse fragility functions to be developed. The *EDPs* obtained were not incorporated in the PACT lifetime performance assessment calculations in accordance with the seismic hazard range recommended by FEMA P-58. Collapse assessment is discussed further in Section 6.3.1.2.

6.3 Results

6.3.1 Seismic Response Analysis

Figures 6-8 to 6-13 show the seismic response analysis results obtained by applying the selected ground motions to the OpenSees structural models of each of the case study CBFs. The median

values of the maximum inter-storey drift and absolute floor acceleration results recorded in the 10 time history analyses at each intensity level are presented. As expected, as the earthquake intensity (represented by the spectral acceleration at the conditioning period, $S_a(T^*)$), to which the ground motion records are scaled increases, the *EDPs* of interest increase accordingly.

The peak inter-storey drift increases consistently with the design behaviour factor, which is as expected given that increasing q results in a more flexible structure. In contrast, peak floor accelerations display less sensitivity to the behaviour factor, particularly at lower ground motion intensities when the frames behave elastically. This is attributable to the fact that the periods of the frames analysed here tend to lie within the constant acceleration region of the spectrum, as illustrated in Figure 6-4(a), and therefore changing the behaviour factor, and consequently the stiffness and structural period, doesn't greatly affect the maximum acceleration response.

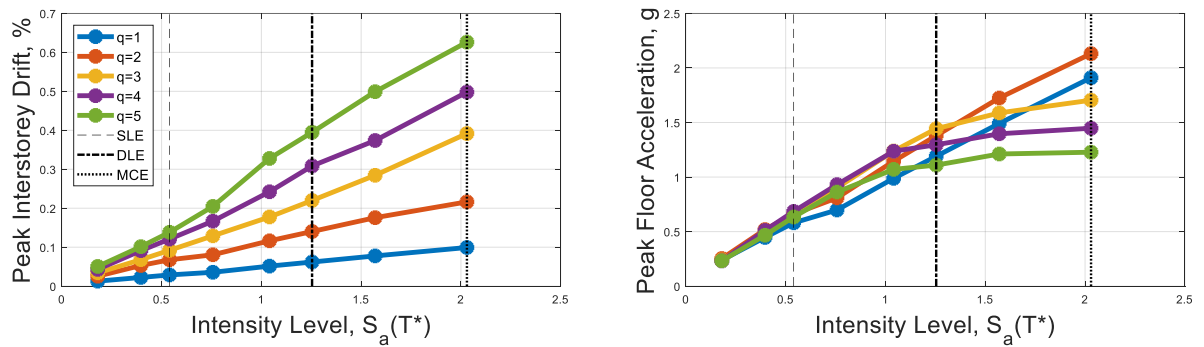


Figure 6-8 Oakland site 2 Storey CBF

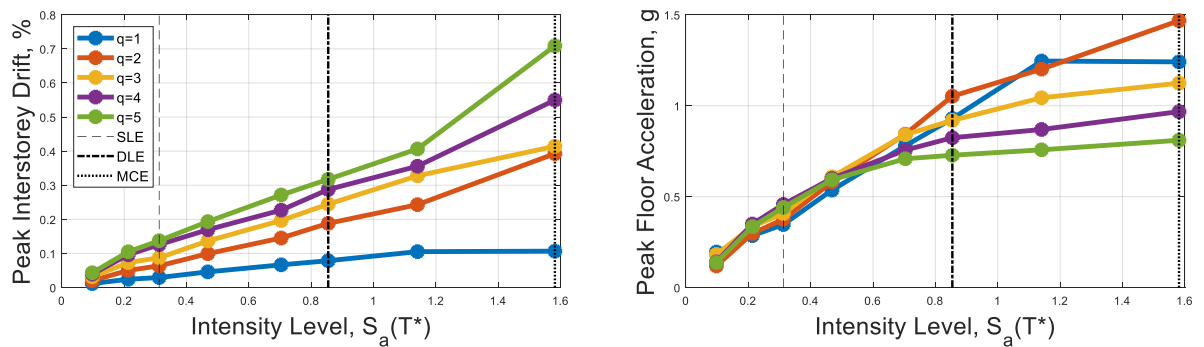


Figure 6-9 Seattle site 2 Storey CBF

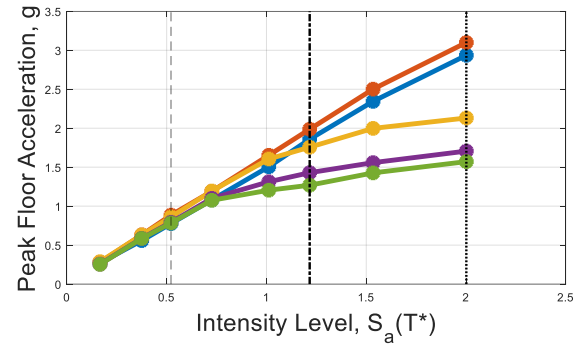
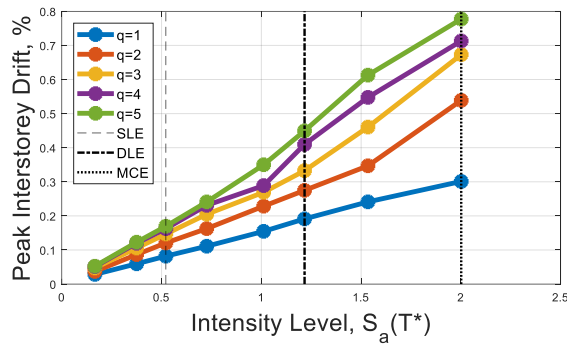


Figure 6-10 Oakland site 5 Storey CBF

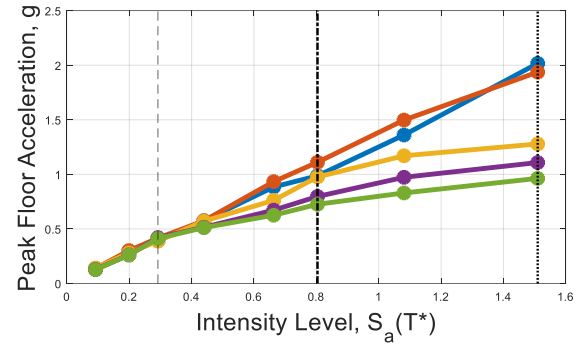
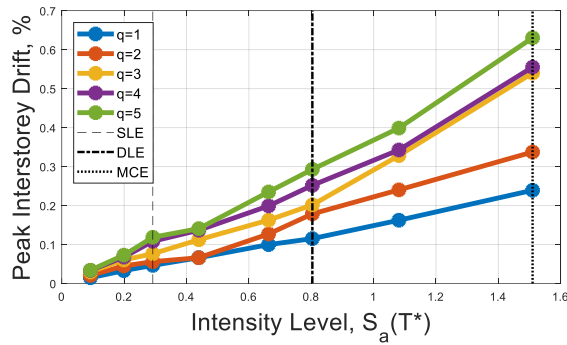


Figure 6-11 Seattle site 5 Storey CBF

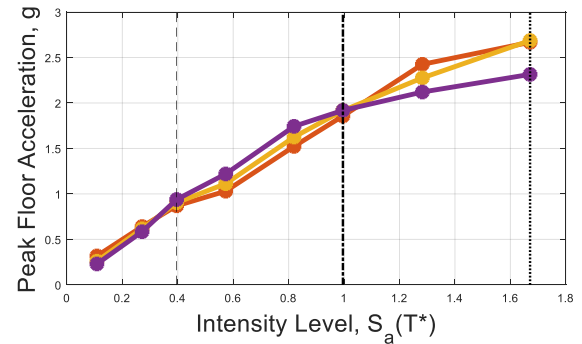
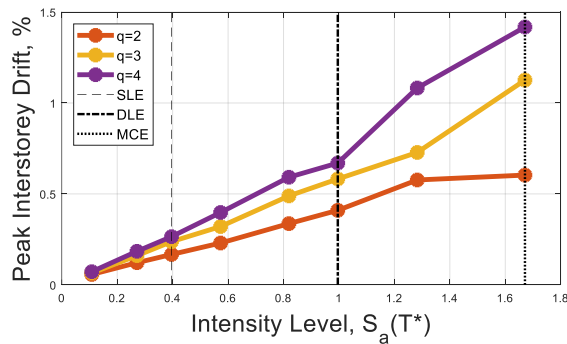


Figure 6-12 Oakland site 10 Storey CBFs

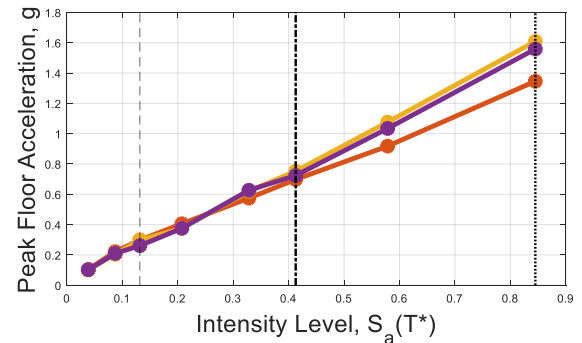
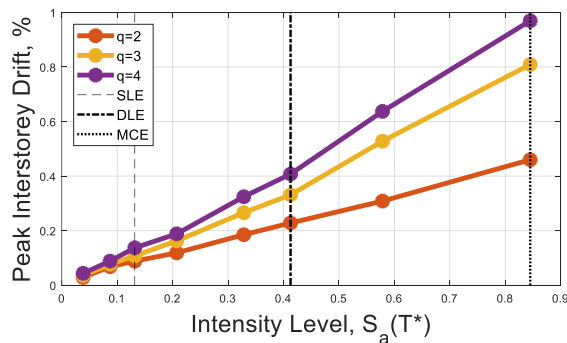


Figure 6-13 Seattle site 10 Storey CBFs

The trends in peak *EDPs* discussed above can also be inferred from Figure 6-14, which shows the storey-level *EDPs* for the 5 storey CBFs at the Seattle site for the SLE, DLE and MCE. The value of

the behaviour factor has a much stronger influence on the peak inter-storey drift profile than on the peak floor acceleration response, particularly at the SLE. As the frames begin to yield at the DLE and especially at the MCE, peak floor acceleration becomes more sensitive to the behaviour factor, with the response values decreasing as more yielding is experienced, i.e. as q is increased.

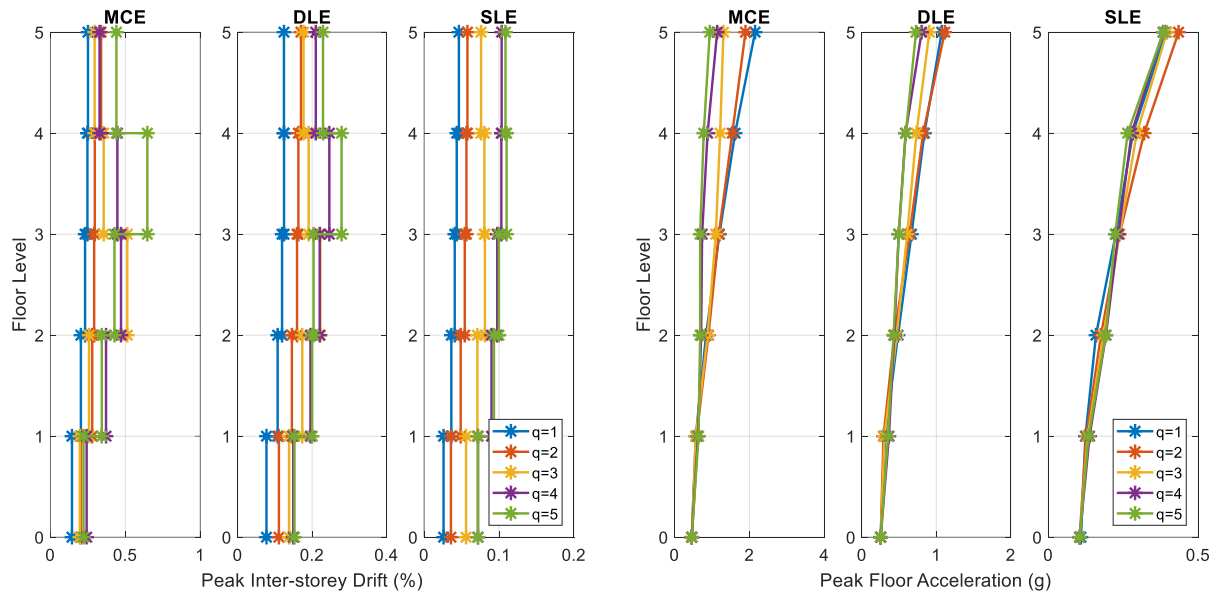


Figure 6-14 Median peak inter-storey drift and peak floor acceleration at each storey level for the 5 storey CBFs at the Seattle site at MCE, DLE and SLE

As previously mentioned, in Eurocode 8 based design it is assumed that the compression brace buckles and only the tension diagonal provides any lateral resistance. In light of the tension only assumption, the fact that the behaviour at the SLE appears to be linear elastic in all cases with median peak drift values of less than 0.2%, is interesting to note from a design perspective. Therefore, it would appear reasonable to consider the contribution of both the tension and compression diagonal in the serviceability limit state displacement checks in Eurocode 8 based design, even if the tension-only assumption is made for the DLE. As mentioned, none of the designs considered here were influenced by the drift limitations imposed by Eurocode 8. However, in other cases where these do become important, for example if the non-structural components require the imposition of a more stringent drift limit, it would be both more realistic and beneficial from a design viewpoint to consider the additional stiffness provided by the compression diagonal.

6.3.1.1 Residual Drift

Two separate methods were used to calculate residual drift. Firstly, the NLTHAs were allowed to run for 10 seconds beyond the length of the ground motion record and the peak inter-storey drift at this point was recorded. However, residual drift can be difficult to accurately predict and

is highly sensitive to the component modelling assumptions related to post-yield hardening or softening and unloading response (FEMA, 2012a). As the models employed in this study are distributed plasticity models and do not incorporate any deteriorating components, it is possible that the recorded residual drifts do not accurately reflect the true values. Consequently, the method for estimating Peak Residual Drift recommended in FEMA P-58 (FEMA, 2012a) was also employed. This involves estimating peak inter-storey residual drift, Δ_r , as a function of peak inter-storey drift experienced during the earthquake, Δ , and the inter-storey drift at yield, Δ_y , as per Equation 6.1:

$$\text{if}(\Delta < \Delta_y) \quad \Delta_r = 0 \quad 6.1a$$

$$\text{if}(\Delta_y < \Delta < 4\Delta_y) \quad \Delta_r = 0.3(\Delta - \Delta_y) \quad 6.1b$$

$$\text{if}(\Delta > 4\Delta_y) \quad \Delta_r = (\Delta - 3\Delta_y) \quad 6.1c$$

The drift at yield, Δ_y , can be estimated as the storey drift ratio associated with storey shear forces equal to the expected yield strength of the braces (FEMA, 2012a).

In general there is little difference between the two methods; at most intensity levels peak residual drifts for CBFs are generally negligible due to the inherent stiffness. However, at the highest intensity levels for the 10 storey CBFs, the values of peak inter-storey drift recorded in NLTHA are in some cases more than an order of magnitude less than those obtained using Equation 6.1, as shown in Figure 6-15. Hence, in order to obtain conservative values for performance metrics, the calculated residual drifts are used in the results presented below.

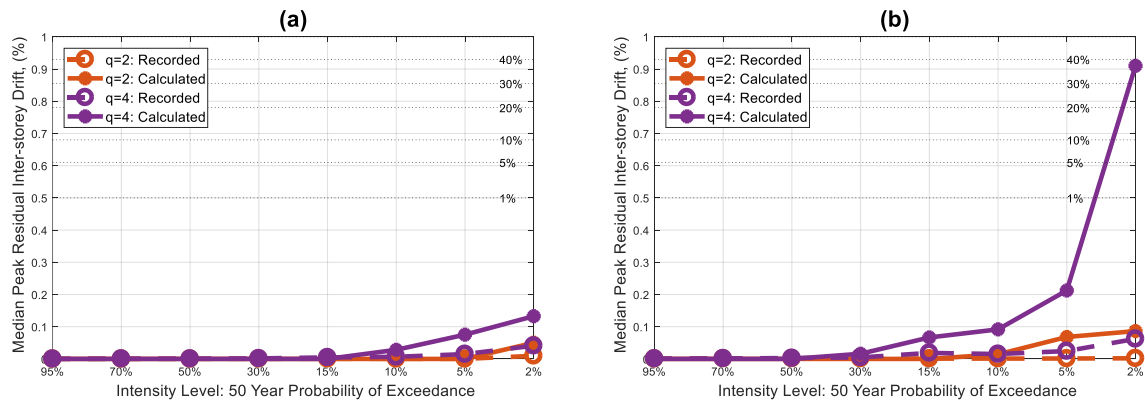


Figure 6-15 Value of peak residual inter-storey drift obtained from NLTHA and calculated using Equation 6.1 for (a) 5 storey CBFs at the Oakland site designed using $q=2$ and $q=4$ and (b) 10 storey CBFs at the Oakland site designed using $q=2$ and $q=4$. The probability of demolition given these residual inter-storey drift values is also marked.

In accordance with the recommendations of FEMA P-58, the probability of demolition given residual drift is described by a lognormal distribution with a median of 1% and a logarithmic standard deviation of 0.3. This fragility function is employed for all case study buildings, however it is noted that the applicability of this fragility function . Some values for the probability of demolition due to residual drift are marked in Figure 6-15; from this the very low probability of demolition irrespective of the method of calculation in most cases can be appreciated.

6.3.1.2 Probability of Collapse

Collapse fragility functions were developed for each frame using the so-called limited-suite analysis method described in FEMA P-58 (FEMA, 2012a). For each intensity level the number of collapses, defined by the occurrence of numerical instability, was divided by the total number of analyses to give the probability of collapse at the intensity level. A lognormal distribution was then fitted to these probabilities in order to develop collapse fragility functions. As mentioned previously, in some cases it was necessary to perform analysis at intensity levels higher than the maximum intensity included in the loss assessment calculation, i.e. the MCE, in order to obtain enough collapses to develop fragility functions.

To account for the uncertainty in developing collapse fragility functions with a limited number of ground motion records, a minimum value of 0.6 was set for the lognormal standard deviation of these fragility curves, in accordance with the FEMA P-58 recommendations. This means conservative values are obtained for collapse prediction at the intensity levels below the median, i.e. those included in the loss assessment calculation. This can be appreciated from Figure 6-16.

As an example, the development of collapse fragility functions for the 5 storey CBF designed using $q=5$ for the Oakland site is illustrated in Figure 6-16. The bar chart in Figure 6-16(a) shows the number of collapses, defined by numerical non-convergence, at each intensity level where NLTHA was performed. From this it can be seen that NLTHA was carried out at the 1% and 0.5% in 50 year intensity levels in order to obtain enough collapses to develop collapse fragility functions. It can also be appreciated that the number of non-convergences, i.e. collapses, increases with increasing earthquake intensity, as expected. The probability of collapse at each intensity level is shown by the green circles in Figure 6-16(b). The dotted blue line is the lognormal distribution fitted to these probabilities, which has a median of $\mu_{Sa} = 3.29$ and lognormal standard deviation of $\beta_{ln,Sa} = 0.44$. The solid blue line is the expanded distribution, accounting for uncertainty in the method, with the same median and a lognormal standard deviation equal to 0.60; this is the fragility function ultimately used in performance assessment.

It can be seen how this is conservative compared to the originally fitted distribution at the lower intensity levels. The red dotted line shows the approximated collapse fragility function with $\mu_{Sa} = 2.93$ and $\beta_{ln,Sa} = 0.70$, calculated using the simplified method based on the design based shear proposed in FEMA P-58. This is reasonably similar, but slightly more conservative, than the widened distribution developed from the results of NLTHA used in loss analysis.

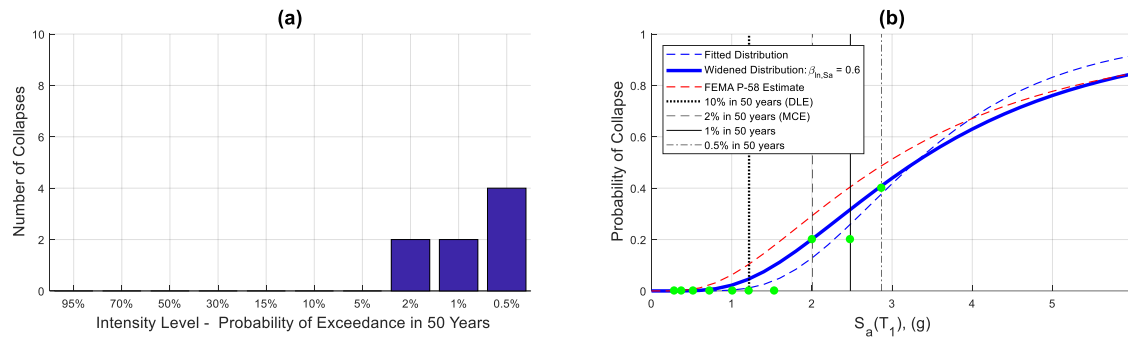


Figure 6-16 Example of fitting a lognormal distribution to collapse data in order to develop collapse fragility curve

As an example of how the behaviour factor impacts on the probability of collapse, Figure 6-17 shows the collapse fragility functions developed for the 10 storey CBFs, while details of all the collapse fragility functions used in loss assessment are provided in Table 6-2. It can be seen that in general, collapse probability increases with both the behaviour factor and frame height.

The definition of collapse in numerical analysis is still somewhat disputed in literature. In this study, numerical non-convergence has been chosen to indicate structural collapse, however this is dependent on various elements of the analysis algorithm adopted such as the integration scheme, solution algorithm and tolerances. Other methods of collapse definitions include defining maximum inter-storey drift limits (e.g. a 4% inter-storey drift limit as recommended by FEMA 356) or by finding the point where the slope of the IDA curve reduces to 20% of its initial value (Villaverde, 2007), while very recently vertical deflection limits have also been proposed as a possible collapse criterion (Zhao, 2019). The fragility functions in Table 6-2 are likely to be sensitive to the definition of collapse employed. However, in the context of this study which aims to examine overall expected losses, the impact of the definition of collapse is likely to be minimal because of the low weighting of infrequent, high intensity earthquakes that can lead to collapse.

Brace fracture in CBFs is generally deemed to occur at inter-storey drifts of approximately 1.5 to 2% (Lignos and Karamanci, 2013, FEMA, 2012a). From Figures 6-8 to 6-13 it can be seen that the median peak inter-storey drift for the 10 storey frames at the Oakland site at the MCE approaches this value while in some cases peak ductility demand in the braces exceeds

predicted brace ductility capacity values (Nip et al., 2010). This is reflected in the relatively high probabilities of collapse for the $q=3$ and $q=4$ designs (greater than 10% at the DLE and greater than 30% at the MCE), as seen in Figure 6-15 and Table 6-2. For the remaining structures examined, median peak inter-storey drift at the MCE is less than 1%, making brace fracture and subsequently collapse unlikely; however the conservative estimation of collapse means that reasonably high collapse probabilities are still obtained. Despite this, as will be shown later on, the impact of collapse on computed lifetime performance metrics is small.

Table 6-2 Collapse Fragility Functions for case study frames, defined by median value, μ_{sa} , and lognormal standard deviation, $\beta_{ln,sa}$. To allow comparison between the different seismic hazards, the collapse probability at the Maximum Credible Earthquake, $P(C|MCE)$, is also presented.

No. of Stories	q	Oakland Site; $a_q = 0.5g$			Seattle Site; $a_q = 0.3g$		
		Median, μ_{sa}	Dispersion, $\beta_{ln,sa}$	$P(C MCE)$	Median, μ_{sa}	Dispersion, $\beta_{ln,sa}$	$P(C MCE)$
2	1	14.49	0.60	0.00	7.07	0.60	0.01
	2	6.03	0.60	0.03	3.04	0.60	0.14
	3	4.52	0.60	0.09	3.00	0.60	0.14
	4	4.00	0.60	0.13	2.69	0.60	0.19
	5	3.50	0.60	0.18	2.63	0.60	0.20
5	1	5.74	0.60	0.04	7.43	0.60	0.00
	2	3.74	0.60	0.30	6.11	0.60	0.01
	3	3.43	0.60	0.19	3.79	0.60	0.06
	4	3.39	0.60	0.19	3.17	0.60	0.27
	5	3.29	0.60	0.21	2.46	0.60	0.21
10	2	3.52	0.60	0.11	1.74	0.60	0.11
	3	2.23	0.60	0.31	1.39	0.60	0.20
	4	2.02	0.60	0.37	1.18	0.60	0.29

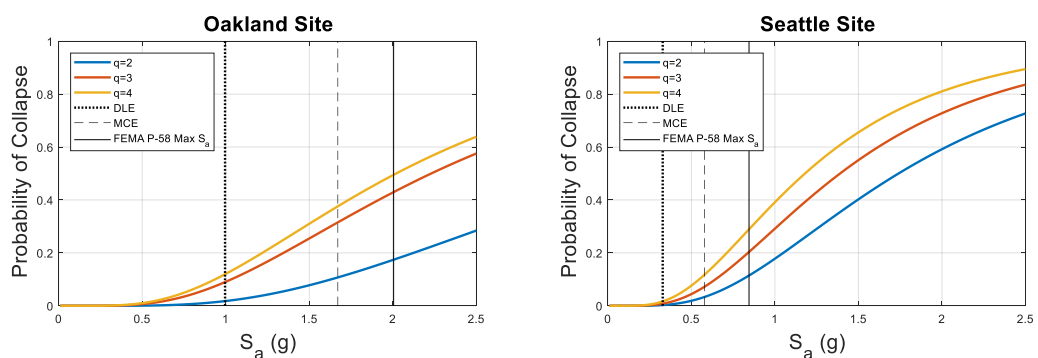


Figure 6-17 Collapse fragility functions for 10 storey CBFs; showing also the spectral acceleration at the DLE, MCE and 1% in 50 year (the maximum spectral acceleration considered in the FEMA P-58 performance assessment calculation) intensity levels

6.3.2 Expected Losses

Repair costs and business downtime were the two performance metrics used to assess the performance of the structures examined. These were estimated using PACT. Building

performance models, listing the structural and non-structural elements, were developed in PACT for this purpose. The *EDPs* recorded in structural analysis, namely maximum inter-storey drift and floor acceleration at each level and peak residual inter-storey drift, as well as the collapse fragility functions, were also inputted. Using this information PACT carries out the performance assessment calculation to evaluate various performance metrics.

6.3.2.1 *PACT Building Model for Performance Assessment*

The PACT building performance model lists the building assets at risk and their exposure to seismic hazard. In order to develop this, the buildings were assumed to be office buildings and the type and quantities of non-structural components used in the loss analysis were obtained using the normative quantities recommended by FEMA P-58, which gives median values for the type and number of non-structural components in typical buildings. A dispersion term, which enables consideration of the variation in these quantities between different buildings, is also provided. Table 6-3 presents the non-structural elements included in each of the calculations. The quantities are different for the 2 storey buildings due to the larger floor area per storey. The fragility and consequence functions from FEMA P-58 were used for loss assessment for all components.

Table 6-3 Non-structural components included in the PACT building model for performance assessment for case study buildings (*included at ground floor only, **included at roof level only)

PACT ID	Component	Quantity per floor level (2 storey buildings)	Quantity per floor level (5 & 10 storey buildings)	Unit	Dispersion (Lognormal Standard Deviation)
Drift-sensitive Non-structural Components					
B2022.001	Curtain Walls	1860	1046.4	Square Feet (SF)	0.6
C1011.001a	Gypsum Partition Walls	620	349	Linear Feet (LF)	0.2
C2011.001b	Prefabricated steel stair	1	1	Unit(s)	0.2
Acceleration-sensitive Non-structural Components					
C3027.001	Raised Access Floor	4650	2616	SF	0.2
C3032.001a	Suspended Ceiling	5580	3140	SF	0.01
C3034.001	Independent Pendant Lighting	93	52	Unit(s)	0.3
D2021.011a	Cold Water Piping	90	50	LF	0.2
D3041.011a	HVAC Galvanized Sheet Metal Ducting less than 6 sq. ft in cross sectional area	470	260	LF	0.2
D3041.012a	HVAC Galvanized Sheet Metal Ducting - 6 sq. ft cross sectional area or greater	120	70	LF	0.2
D3041.031a	HVAC Drops / Diffusers in suspended ceilings	55.8	31	Unit(s)	0.5
D3041.041a	Variable Air Volume (VAV) box	43.4	24	Unit(s)	0.2
D4011.021a	Fire Sprinkler Water Piping	1240	700	LF	0.1
D4011.031a	Fire Sprinkler Drop	56	31	Unit(s)	0.2
D5012.021a	Low Voltage Switchgear (<350 Amp)	1	1	Unit(s)	0.4
D5012.013a	Motor Control Center*	1	1	Unit(s)	0.5
D1014.011	Traction Elevator *	1	1	Unit(s)	0
B3011.011	Concrete tile roof**	1674	942	SF	1.3
D3031.011a	Chiller**	1	1	Unit(s)	0.1
D3031.021a	Cooling Tower**	1	1	Unit(s)	0.1
D3052.011a	Air Handling Unit**	1	1	Unit(s)	0.2

Brace and gusset plate connection assemblies were represented in the building performance model using the FEMA P-58 fragility and consequence functions for “Special Concentric Braced Frame with HSS braces, tapered gusset plates & design to AISC minimum standard, Single Diagonal Brace”, which is numbered B1033.022 within PACT. The appropriate weight per unit length was selected for each bracing member.

6.3.2.2 Uncertainty in Structural Modelling

PACT employs a Monte Carlo procedure to develop a large number of earthquake realizations from a limited number of NLTHAs. Each realization represents one possible building

performance outcome given a particular earthquake scenario. This means that the number of potential earthquakes considered in the performance calculation is much greater than the number of NLTHAs performed.

The *EDPs* for each realization are developed based on the distribution of *EDPs* obtained from NLTHA. In order to account for the uncertainty in structural modelling these *EDP* distributions are widened using a factor, termed β_m . The impact of incorporating this uncertainty factor on an *EDP* distribution is shown in a simplified manner in Figure 6-18.

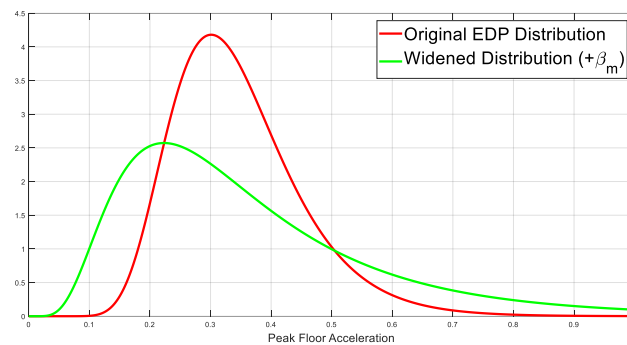


Figure 6-18 Simplified representation of the impact of β_m on *EDP* distribution for the development of PACT realizations

The value of β_m used for all loss assessments in this study was calculated using Equation 6.2:

$$\beta_m = \min \left\{ \frac{\sqrt{\beta_c^2 + \beta_q^2}}{0.5} \right\} \quad 6.2$$

where β_c and β_q represent the uncertainty associated with building construction quality assurance and quality and completeness of the structural model respectively. A value of 0.4 is recommended by FEMA P-58 for both β_c and β_q for buildings at the preliminary design stage where only the seismic resisting components are included in the structural models. Combining these two uncertainties through Equation 6.2 gives a value of 0.565, which is greater than the maximum recommended value of 0.5. Consequently a β_m value of 0.5 was employed for all loss analyses.

6.3.2.3 Performance Metrics

Figure 6-19 shows the present value of the estimated lifetime losses expressed as a percentage of initial cost for each building type. Total financial losses were defined as the cost of implementing repairs plus the estimated revenue losses resulting from business downtime. Downtime revenue losses are calculated as the loss in income for leasing office space assuming a rental value of \$25/ft²/year (Terzic et al., 2014). The initial costs are estimated as \$250/ft², plus

an additional cost of \$4000 per tonne of extra steel used as the behaviour factor is reduced (Terzic et al., 2014).

Expected Annual Losses (*EAL*) are calculated by integrating the losses at each intensity level over the hazard curve for the site in question. The median annualized repair costs and downtime for each case study building are given in Table 6-4. The present value (*PV*) of lifetime losses is then calculated from the *EALs* using Equation 6.3 (Wen and Kang, 2001a):

$$PV = EAL \times \frac{1 - e^{-rt}}{r} \quad 6.3$$

where *r* is a discount factor accounting for inflation, taken here as 3%, and *t* is the lifetime of the building, taken here as 50 years.

Table 6-4 Annualized repair costs and annualized downtime calculated for each of the case study buildings

No. of Stories	<i>q</i>	Oakland Site, $a_g = 0.5g$		Seattle Site, $a_g = 0.3g$	
		Annualized Repair Costs (% Initial Cost)	Annualized Downtime (Days)	Annualized Repair Costs (% Initial Cost)	Annualized Downtime (Days)
2	1	0.27	1.56	0.17	0.83
	2	0.35	2.05	0.21	1.10
	3	0.39	2.33	0.29	1.55
	4	0.45	2.67	0.30	1.68
	5	0.49	2.95	0.31	1.73
5	1	0.29	4.64	0.15	2.00
	2	0.36	5.54	0.17	2.25
	3	0.42	6.31	0.20	2.72
	4	0.49	7.29	0.23	3.18
	5	0.51	7.49	0.25	3.45
10	2	0.44	12.71	0.16	4.40
	3	0.63	15.96	0.23	5.71
	4	0.88	20.64	0.29	6.98

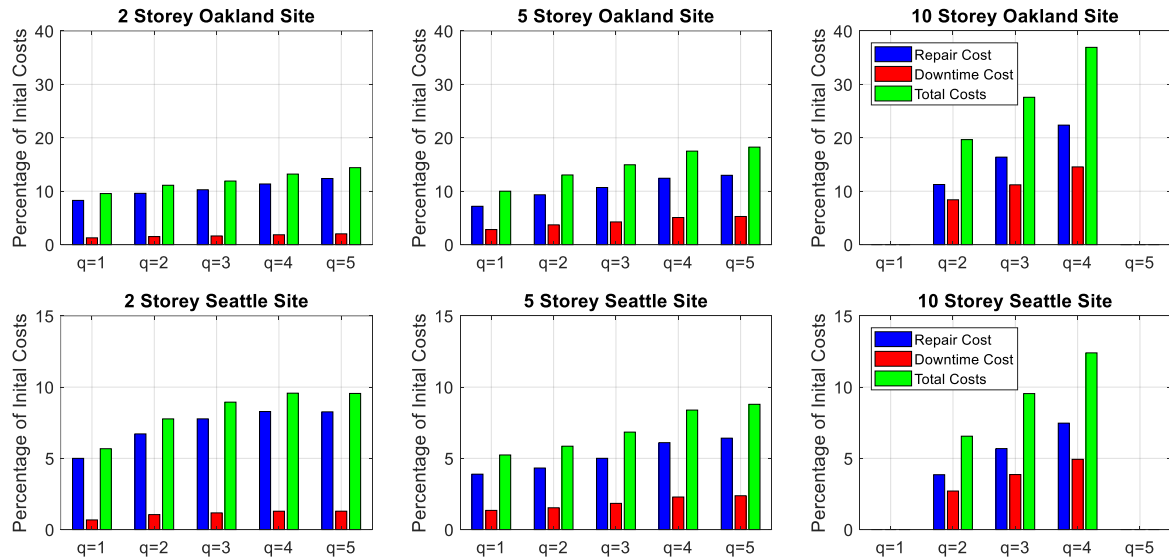


Figure 6-19 Present value of economic losses

From Figure 6-19, it can be seen that the total seismic financial loss incurred is lowest for the $q=1$ frames (or $q=2$ for the 10 storey frames where a $q=1$ frame was not analysed) and increases as the behaviour factor is increased. The expected losses increase with the number of stories due to the increased inter-storey drift in the taller, more flexible frames.

For the highly seismic Oakland site, repair costs range from approximately 7% to 22% of initial costs and are higher for the taller 10 storey CBFs. For the Seattle site, with a mid-range seismic hazard, repair costs remain relatively constant, or even decrease, with frame height and range from approximately 4% to 8% of initial costs. In all cases, repair costs increase with the behaviour factor. Downtime costs always increase with both frame height and behaviour factor, and the contribution of downtime related losses to overall losses becomes increasingly important as the number of stories increases. The present values of these repair costs are generally of the same order of magnitude as those obtained in other studies. For Special Concentrically Braced Frames in highly seismic regions designed to US provisions (AISC, 2005), (ASCE/SEI, 2006), Hwang and Lignos (2017a) obtained costs ranging from approximately 15% to 25% of the building replacement cost, while for moment resisting frames the same authors (2017b) obtained lifetime losses of 10% to 20%. Both of these studies examined repair costs only and didn't include downtime related costs, which in this study are observed to be as high 15% of initial costs for the 10 storey building at the Oakland site designed with $q = 4$.

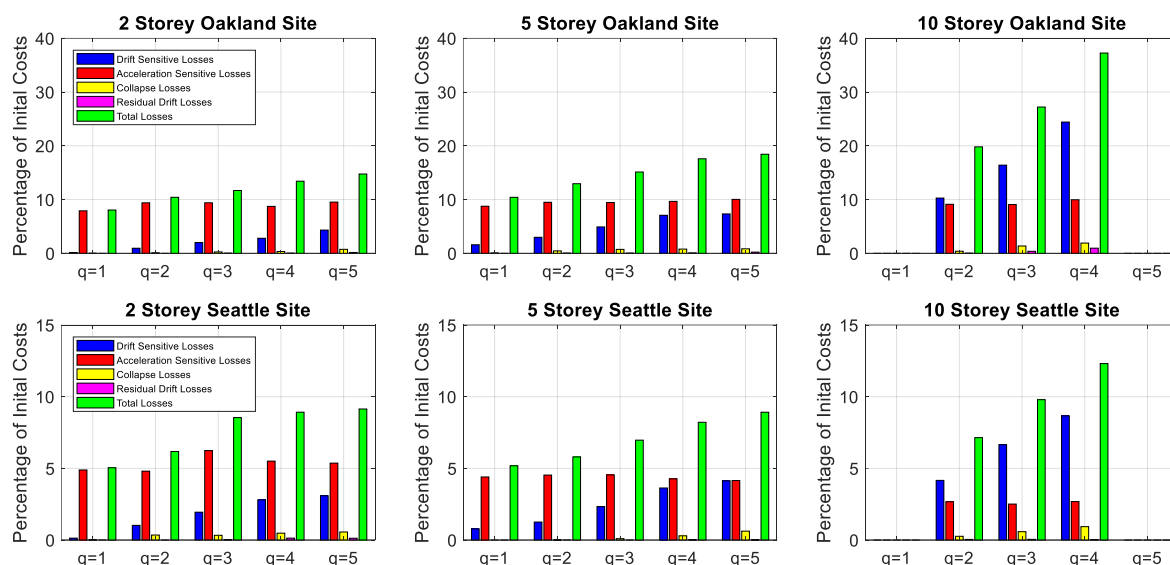


Figure 6-20 Present value of lifetime losses as a percentage of initial costs for the case study CBFs

Figure 6-20 shows the breakdown of financial losses between drift and acceleration sensitive components as well as the contribution of collapse dependent losses and losses due to demolition as a result of excessive residual drifts to overall losses. As can be seen from Table 6-3, drift sensitive components include the structural system as well as some non-structural elements such as curtain walls, while acceleration sensitive components are non-structural elements such as service ducts, raised access floors and suspended ceilings. Losses to acceleration sensitive components are the major contributor to overall losses in the two and five storey buildings. However, drift resultant losses become more significant as the number of stories is increased, contributing more than acceleration dependant losses to the overall estimated losses in the 10 storey CBFs. Losses due to collapse are a minimal contributor to overall losses, contributing less than 8% of total losses in all cases. Losses due to residual drifts are even less important (always less than 3% of total losses and less than 1.5% in all bar 1 case), to the extent that their contribution to lifetime losses is not visible in Figure 6-20 for the majority of the case study buildings.

Drift sensitive losses grow with increased behaviour factors to a much greater extent than acceleration dependant losses. This trend can be explained by considering the results of structural analysis discussed previously. Peak inter-storey drift was shown to increase with the behaviour factor, in contrast to peak floor acceleration, which was relatively insensitive to q , particularly at the lower intensity levels. Consequently, the behaviour factor impacts on drift sensitive losses to a much greater extent. Given that drift dependant losses become more significant with frame height, the benefit of reducing the behaviour factor to limit drift

dependant losses becomes more pronounced as the number of stories is increased. Overall, additional total lifetime cost penalty from using dissipative ($q=4$ or $q=5$) rather than non-dissipative ($q=1$ or $q=2$) design ranges from 2% to 16% of initial costs, with the highest penalty observed for the tallest building at the higher seismicity site.

Losses are greater for the Oakland site compared to the Seattle site, which makes sense given the respective seismic hazards. This is primarily due to the extent of the damage suffered at frequently occurring intensity levels (50% in 50 years and below), something which can be appreciated from Figures 6-21 to 6-23. As these losses are associated with more frequent earthquakes, their contribution to the lifetime losses calculation is heavily weighted and therefore influential on the final result. There is a larger difference between the two sites for the losses for the 10 storey CBFs compared to the 2 and 5 storey CBFs. This can be explained by examining the hazard curves for the frames in question in Figure 6-4(b), where it can be seen that the hazard reduction between the sites is much greater for the 10 storey frames. (The design ground acceleration value for the Seattle site, $0.3g$, is 60% of that at the Oakland site, $0.5g$. The spectral accelerations at the 10% in 50 year intensity level at the Oakland site are approximately $1.25g$, $1.20g$ and $1.0g$ for the average periods of the 2, 5 and 10 storey frames respectively, while for the Seattle site the figures are 0.85 , 0.80 and 0.40 respectively. This means that, for the frames considered, the seismic hazard at the Seattle site is on average 68%, 66% and 40% of that at the Oakland site. Clearly the reduction in seismic hazard is greatest for the 10 storey structures and the extent of this reduction is not fully captured by the EC8 design spectrum. This inadvertently highlights one of the weaknesses of the Eurocode 8 design spectrum, as discussed by Bommer and Stafford (2009), where spectral shape is dictated solely by peak ground acceleration and the design spectrum can diverge from the seismic hazard at higher periods).

Many of the trends discussed above can also be appreciated by examining Figure 6-21 to 6-23, which show intensity-specific losses for the case study CBFs at the MCE, DLE and SLE. The lifetime losses shown in Figures 6-20 are less than the MCE and DLE specific losses in Figure 6-21 to 6-23 because of how the losses at these intensity levels are weighted in the lifetime performance calculation. For the 2 and 5 storey structures damage to acceleration sensitive components is the dominant contributor to overall losses at the more frequently occurring SLE, however at the MCE and DLE this balance is observed to depend on the behaviour factor, with drift dependant losses tending to be more important for the higher q designs. For the 10 storey CBFs drift dependant losses are more significant than acceleration sensitive losses each of the

intensity levels shown, irrespective of the behaviour factor. It should also be noted here that losses at the MCE for the 10 storey CBFs designed using $q=4$ at the Oakland site exceed 100% of initial cost, something which is possible because both downtime and repair cost are included in the calculation. Contributions from collapse losses also become significant for the higher behaviour factor frames at the MCE and are the primary contributor to overall losses in some cases. Demolition losses only become important for the MCE for the $q=3$ and $q=4$ 10 storey frames and for the $q=5$ 5 storey frame, at the Oakland site.

For the intensity levels shown, the difference between losses at the two sites is greatest for the more frequently occurring SLE events. The trend is observed for all the case study CBFs but is particularly evident for the 10 storey frames, where the SLE losses are almost negligible for the Seattle site but reach over 30% of initial costs for the Oakland site. The reasons for this difference between the two sites is because at the low intensity levels the weak ground motions often result in no damage at the Seattle site, whereas the slightly stronger motions at the Oakland site generally results in some level of damage and losses. Given that these low intensity levels are heavily weighted in the lifetime performance assessment calculation, this trend has a significant impact on lifetime losses, particularly for the 10 storey CBFs.

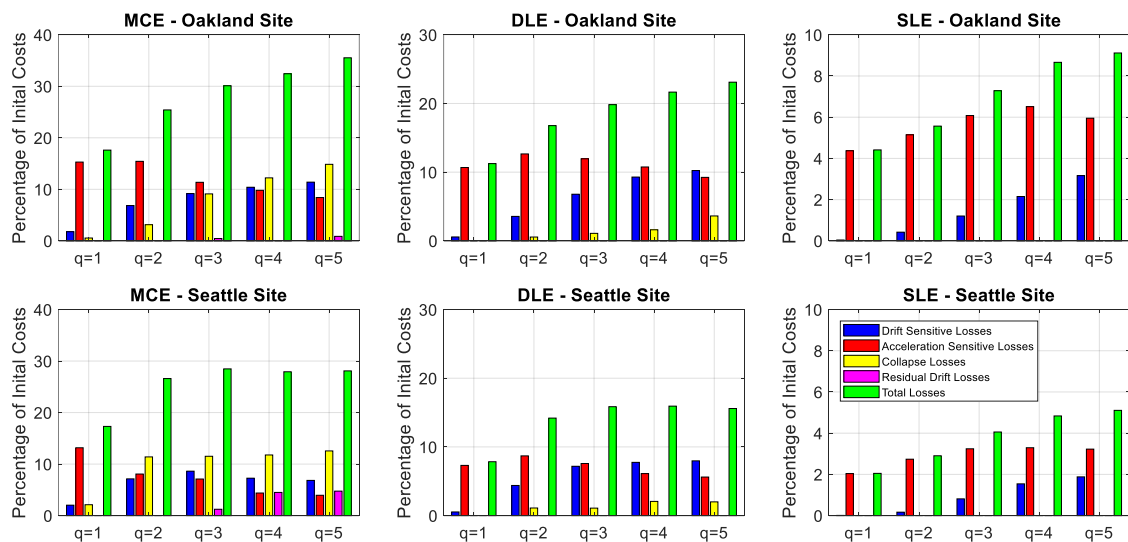


Figure 6-21 Intensity-specific losses for the two storey CBFs

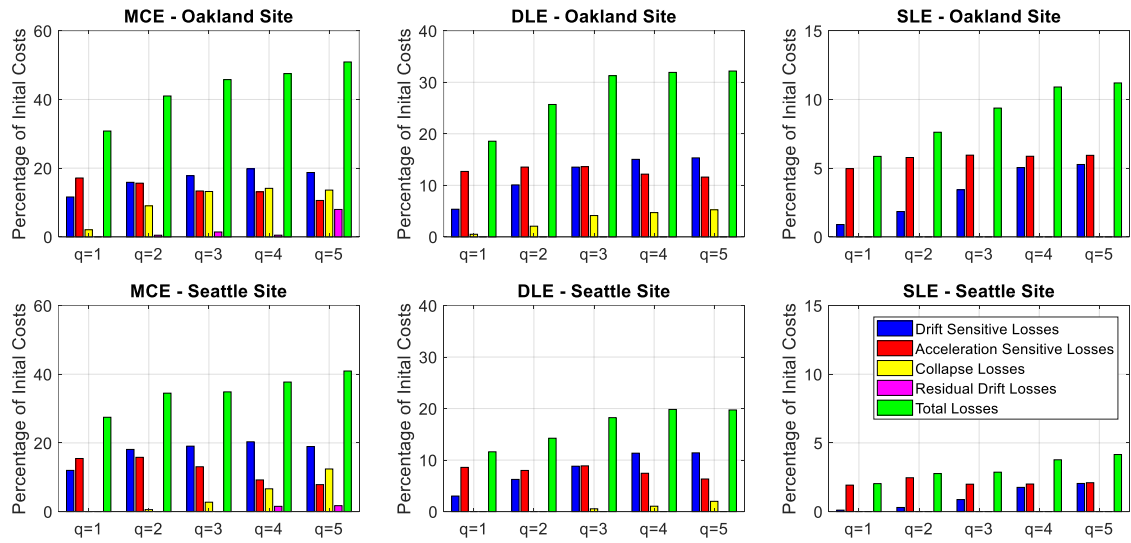


Figure 6-22 Intensity-specific losses for 5 storey CBFs

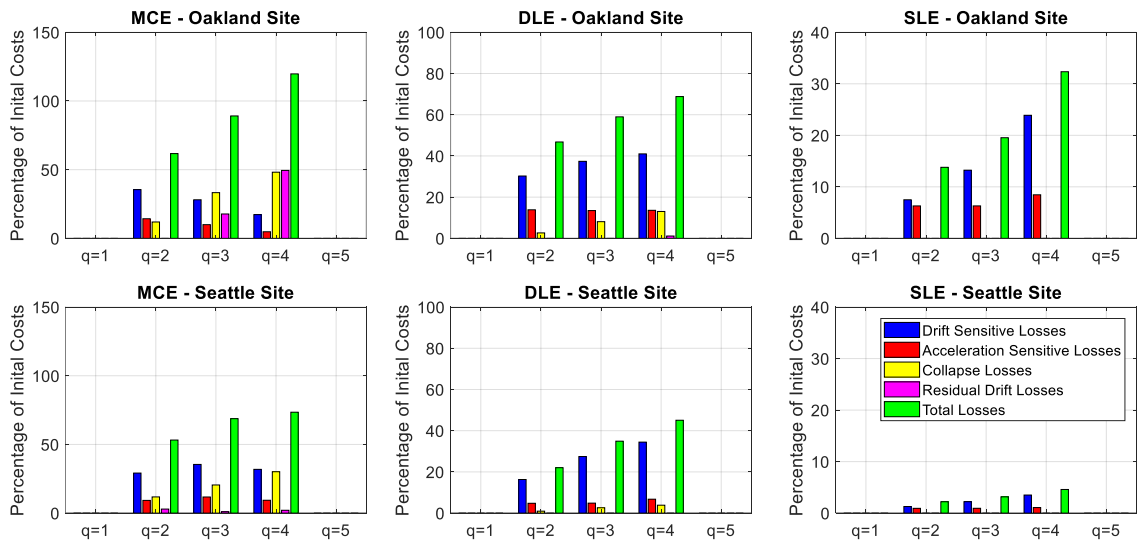


Figure 6-23 Intensity specific losses for 10 storey CBFs

6.3.2.3.1 Impact of Peak Residual Drift Calculation Method

Figures 6-24 and 6-25 show the impact of the method of calculating peak residual inter-storey drift on lifetime and intensity specific losses respectively for the 10 storey CBFs at the Oakland site. As discussed in Section 6.3.1.1, peak residual inter-storey drift was both recorded at the end of NLTHA and calculated using the method proposed in FEMA P-58 using Equation 6.1. For the 10 storey CBFs, at higher intensity ground motions recorded residual drifts were found to be over an order of magnitude smaller than the corresponding calculated values in some cases. Hence, as mentioned, the values of expected losses used elsewhere in this chapter are derived using the more conservative calculated residual drifts.

However, despite this, it can be seen from Figure 6-24, that the impact of demolition due to excessive residual drifts on lifetime losses is minimal. This low contribution of residual drift losses to overall losses for CBFs has been noted previously by Hwang and Lignos (2017a), where they typically accounted for approximately 1% of lifetime repair costs. Hence, irrespective of how residual drift is calculated, it can be concluded that the inherent stiffness of low and mid-rise CBFs means that building condemnation is unlikely and is not particularly important from a lifetime loss perspective. This is in contrast to MRFs, where losses resulting from demolition due to excessive residual drift can account for over one fifth of total lifetime losses (Hwang and Lignos, 2017b).

Looking at intensity specific losses, from Figure 6-25 it can be seen that demolition losses become significant at the MCE when residual drifts are calculated using Equation 6.1 as opposed to recorded. Indeed, for the 10 storey frame at the Oakland site designed using $q=4$, demolition losses are the largest contributor to total losses when residual drifts are calculated based on the peak inter-storey drift. The proportional loss contribution at the MCE from calculated residual drifts is in line with the values reported by Hwang and Lignos (2017a). At the DLE and the SLE the impact of residual drifts not notable, irrespective of whether the calculated or recorded value is used. Therefore, it can be concluded that the residual drift calculation method is only important for very high intensity earthquakes. The low weighting of these earthquakes in the lifetime loss calculation explains why the impact on lifetime losses of the method employed is minimal.

Finally, it should be noted that the demolition losses presented are all dependant on the fragility function representing the probability of demolition given residual drift. As mentioned in Section 4.2 and shown in Figure 4-7, a lognormal distribution with a median of 1% and a distribution of 0.3, as recommended in FEMA P-58, was used for this purpose in this study. However, as was noted by Karavasilis et al. (2015) there is still some debate over the ideal values to use in this distribution and additional research is needed to evaluate the residual drift limit value beyond which the structure is demolished.

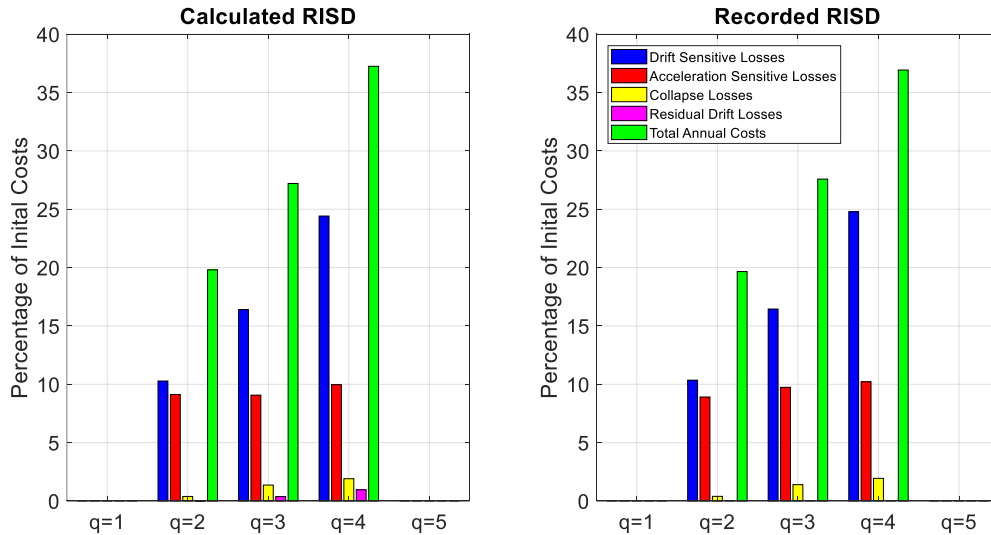


Figure 6-24 Impact of residual drift calculation method on the lifetimes losses experienced by the 10 storey CBFs at the Oakland site

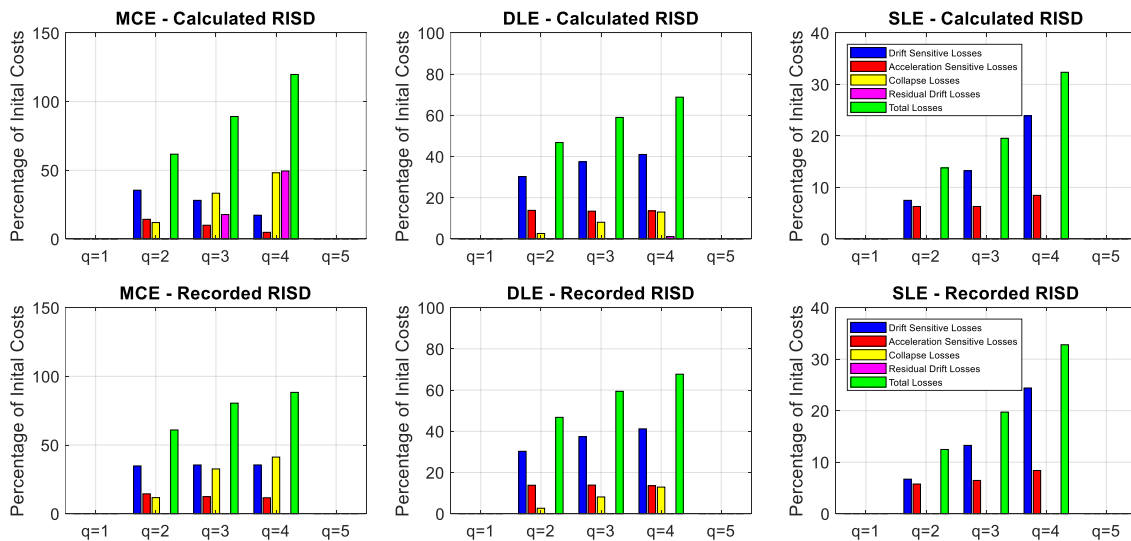


Figure 6-25 Impact of residual drift calculation method on the intensity-specific losses experienced by the 10 storey CBFs at the Oakland site

6.3.3 Analysis of Lifetime Losses

6.3.3.1 Loss Impact Factor - LIF

In order to further understand the impact of the behaviour factor on calculated lifetime losses, a linear trend line is fitted to the total expected losses for each frame height and site. This concept is illustrated in Figure 6-26. The slope of this line, which is termed the Loss Impact Factor or LIF_q , indicates the influence of q on the expected losses; a greater LIF_q means that q has a larger impact on expected losses. The LIF values obtained for each set of case study buildings are given in Table 6-5. The value of the intercept, c , is also given in Table 6-5, such that the present value of expected losses can be approximated as:

$$E(L) = LIF_q(q) + c$$

6.4

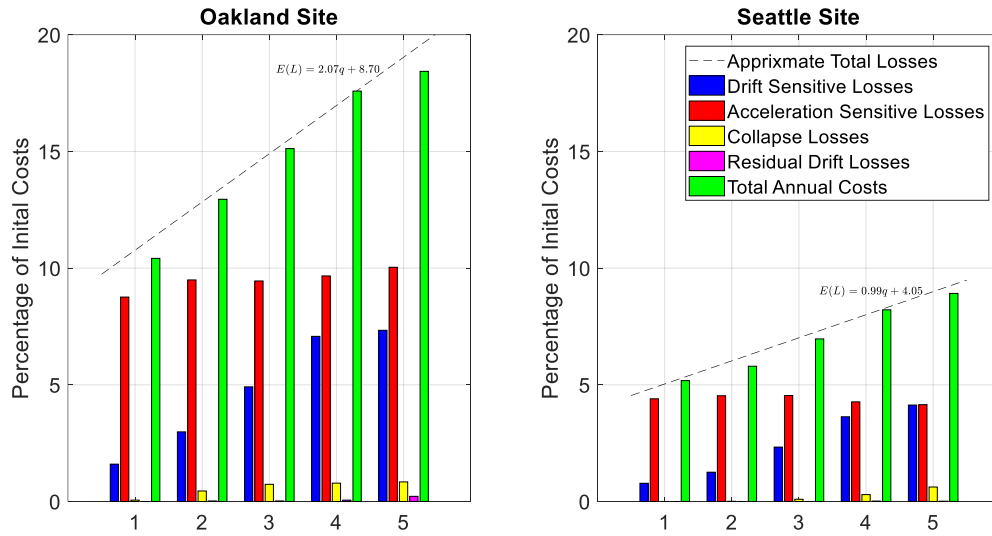


Figure 6-26 Fitting a linear trend line to total expected losses for the 5 storey CBFs; the slope of this line is the LIF_q value

Table 6-5 LIF_q values for the case study CBFs

No. of Stories	Oakland Site		Seattle Site	
	LIF_q (%initial/ q)	c	LIF_q (%initial/ q)	c
2	1.64	6.75	1.09	4.28
5	2.07	8.70	0.99	4.05
10	8.72	1.92	2.58	2.00

From Table 6-5, it can be seen that the LIF_q generally increases with both frame height and seismic hazard. This indicates that the impact of q on expected losses becomes more important as frame height and seismic hazard are increased. The value of the LIF_q indicates what percentage of the initial costs are saved by reducing q by 1; for example for the 10 storey frames savings of almost 9% of initial cost, for the Oakland site, and more than 2.5% in Seattle, can be achieved by reducing the design behaviour factor by 1. For the 2 and 5 storey frames this saving is in the order of 1 to 2% of initial cost.

It is possible to calculate a Loss Impact Factor with respect to any design parameter in order to approximately examine how it impacts expected losses. Table 6-6 gives values for LIF with respect to the mass of steel and the fundamental period, termed LIF_M and LIF_T respectively. Clearly, these parameters are somewhat related to q , as both structural mass and period are functions of the design behaviour factor; however it is still relevant to examine the respective $LIFs$ to develop an appreciation for the potential savings that can be made.

Table 6-6 LIF_M and LIF_T values for the case study CBFs

No. of Stories	Oakland Site		Seattle Site	
	LIF_M (%initial/Mg)	LIF_T (%initial/s)	LIF_M (%initial/Mg)	LIF_T (%initial/s)
2	-0.13	51.47	-0.11	28.16
5	-0.07	55.28	-0.05	20.10
10	-0.13	91.98	-0.04	22.46

The LIF_M values presented are negative, indicating that losses reduce as the mass of steel used in the structure is increased. The magnitude of the LIF_M values ranges from 0.04 to 0.13 percent of initial cost per Mg of steel. On average, approximately 0.08% of initial cost is saved by increasing the weight of the structure by 1Mg, or, on a more relevant scale, increasing the weight of the structure by 50Mg saves on average 4% of initial costs.

The positive LIF_T values indicate that increased fundamental period results in increased losses, or conversely, increased frame stiffness leads to reduced losses. The trends noted are similar to those highlighted for LIF_q ; the impact of fundamental period on expected losses generally increases with both seismic hazard and frame height. For the Seattle site a design alteration that reduces the fundamental period by 0.1s saves approximately 2.5% of initial cost. The variation of LIF_T is more extreme for the Oakland site, with a 0.1s reduction in T_1 leading to loss reductions of approximately 3.6%, 5.6% and almost 10% of initial costs for 2, 5 and 10 storey CBFs respectively.

6.3.3.2 Return on Investment – LIF_S

Finally, the LIF calculated with respect to the estimated initial cost, termed LIF_S , is presented. This is calculated using dollar values, as opposed to percentages, for both expected losses and initial costs. This means that LIF_S is a measure of how effectively the extra money invested initially to reduce q is utilized on average across each of the behaviour factors examined. Therefore, LIF_S is a measure of the return on investment; an LIF_S value with a magnitude equal to 1 means that increasing the initial cost by \$1 results in savings of \$1. LIF_S magnitudes less than 1 mean that increasing the initial cost by \$1 results in savings of less than \$1, while LIF_S magnitudes greater than 1 mean that increasing the initial cost by \$1 results in savings of greater than \$1. Hence an LIF_S magnitude greater than 1 implies that the initially more expensive, less ductile frames make the extra investment worthwhile, whereas the opposite is true for LIF_S magnitudes less than 1.

Table 6-7 Return on Investment, LIF_T , values for the case study CBFs

No. of Stories	Oakland	Seattle
	$LIF_{\$}$ (\$ _{lost} /\$ _{spent})	$LIF_{\$}$ (\$ _{lost} /\$ _{spent})
2	-0.90	-0.78
5	-0.61	-0.51
10	-2.61	-0.88

Table 6-7 presents the $LIF_{\$}$ values for the buildings analysed. Firstly, it can be seen that the $LIF_{\$}$ values are all negative, meaning there is a negative correlation between initial costs and expected losses, i.e. losses are reduced as investment increases. It can be seen that for the 2 and 5 storey frames, and the 10 storey frame at the Seattle site, the magnitude of the $LIF_{\$}$ values are less than 1, ranging from approximately 0.5 to 0.9 \$/\$. This means that on average lifetime losses are reduced by \$0.50 to \$0.90 for every extra dollar spent initially. Clearly, this represents a poor return on investment. This suggests that for these scenarios employing highly dissipative designs, i.e. using behaviour factors of $q=4$ or $q=5$, is the best way to reduce total life costs. In contrast for the 10 storey frames at the Oakland site the magnitude of the $LIF_{\$}$ value is 2.61; hence for every \$1 invested approximately \$2.50 are saved. Clearly, this suggests using initially more expensive, less-dissipative designs, i.e. $q=2$ or less, is ultimately economically beneficial for taller frames at highly seismic sites.

Comparison between Two Storey CBFs and MRFs

In order to compare CBF performance to MRF, a set of 2 storey MRFs, illustrated in Figure 6-27, were designed for the Oakland site. Table 6-8 compares the two storey CBF and MRF designs. Unlike the CBFs, for the MRFs the drift limitations were often the controlling factor in design. Thus, the higher q value MRF designs have large overstrength relative to q and tend to exhibit similar performance, regardless of the behaviour factor used. From Table 6-8, it can be seen that the MRFs are typically more flexible, as indicated by the longer fundamental periods, and require a higher mass of steel than the corresponding CBF designs.

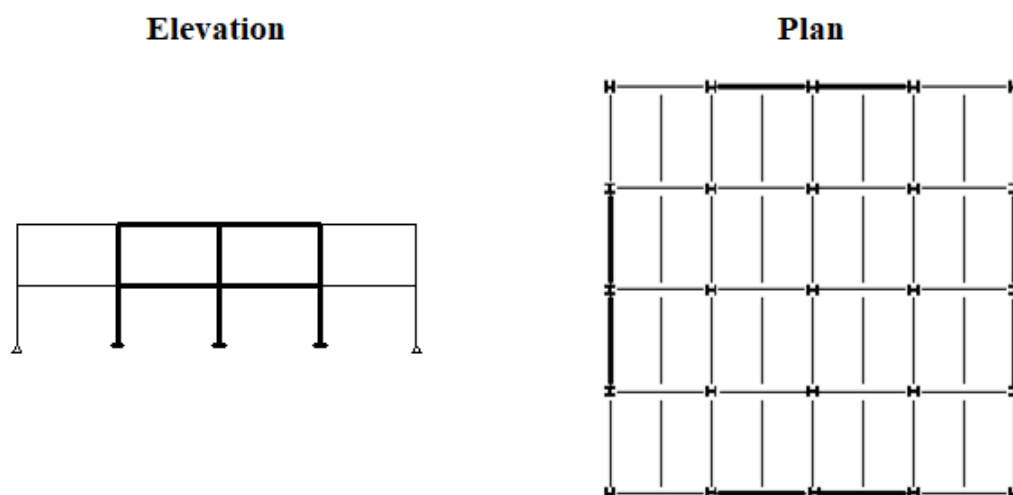


Figure 6-27 Elevation and Plan of 2 storey moment resisting frame (MRF)

Table 6-8 Comparison between two storey CBFs and MRFs, showing the influence of the behaviour factor, q , on the mass of steel required and fundamental structural period, T_1

q	CBF		MRF	
	Mass of Steel (Mg)	T_1 (s)	Mass of Steel (Mg)	T_1 (s)
1	91.4	0.12	91.6	0.30
2	70.6	0.16	76.3	0.38
3	58.8	0.20	75.1	0.39
4	44.8	0.23	71.4	0.41
5	41.2	0.24	68.7	0.42

6.3.4 EDPs

Figure 6-28 compares the calculated response of the two storey CBFs and MRFs at the Oakland site designed using $q=1$, $q=3$ and $q=5$. The previously discussed relationships between the behaviour factor and the EDPs for different CBF designs also apply when comparing MRFs and CBFs. Peak inter-storey drift is much greater for the more flexible MRFs. However, median peak floor acceleration values are broadly similar for the two frame types, particularly for the more frequent, lower intensity ground motions. As before, this can be attributed to the frame designs generally lying in the constant acceleration region of the spectrum, irrespective of frame type or behaviour factor.

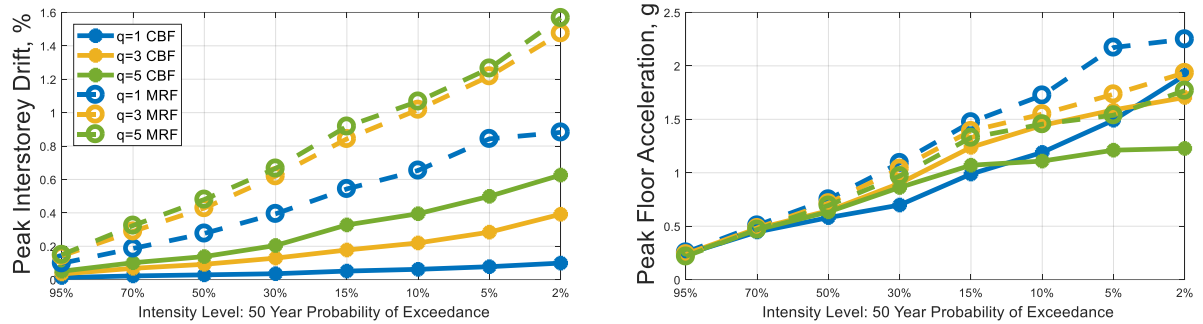


Figure 6-28 Comparison of Peak Inter-storey Drift and Peak Floor Acceleration for 2 storey CBF and MRF for Oakland site

6.3.5 Performance Metrics

Figure 6-29 compares the present value of expected economic losses for the two storey CBFs and MRFs at the Oakland site. Losses are slightly greater for the MRFs. This is because drift induced losses are more significant for the MRFs than CBFs; a consequence of the greater inter-storey drift experienced by MRFs. However, the drift dependant losses do not increase proportionally with the peak drift demand, which is generally over twice as high for the MRFs. This is because MRFs can accommodate greater drifts before structural damage begins to occur. In the fragility functions used in FEMA P-58 structural damage in CBFs, primarily brace buckling, can occur at inter-storey drifts as low as 0.35%, whereas structural damage is initiated MRFs at drifts almost an order of magnitude greater. Acceleration dependent losses are similar for the two frame types, which again can be explained by the similarity in peak floor acceleration demands as shown in Figure 6-28. However, the rate of increase in lifetime losses with q is greater for the CBFs. This is a function of how q impacts MRF design; the designs for higher q values are controlled by stiffness requirements rather than strength, which is influenced by the behaviour factor. Losses from collapse or demolition due to excessive residual drifts are minimal contributors to overall losses for both frame types. However, demolition losses are slightly more prominent for the MRFs compared to the CBFs. The results of other studies (Hwang and Lignos, 2017b) would suggest that these trends would become more pronounced were taller frames examined.

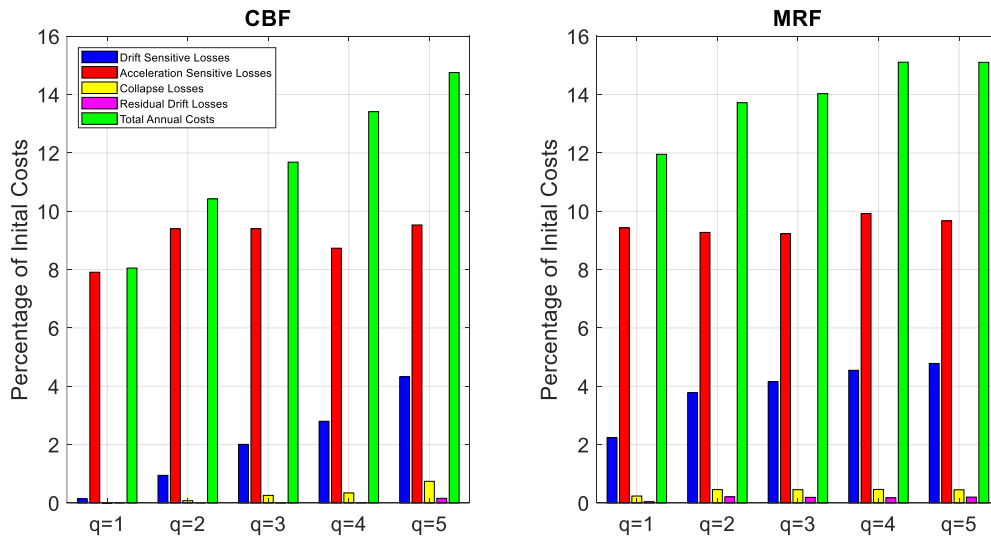


Figure 6-29 Present value of expected losses for 2 storey CBFs and MRFs at the Oakland site

6.3.6 LIF and Return on Investment

Table 6-9 compares the LIF_q , LIF_M , LIF_T and LIF_S values for the CBFs and MRFs. The LIF_q value is greater for the CBFs compared to the MRFs. This reflects the greater impact of q on expected losses for CBFs, visible in Figure 6-29 and which, as explained, primarily results from the impact of the drift limitations imposed by EC8 on MRF designs with higher q values. The LIF_M and LIF_T values for the two frame types are similar however, suggesting that, once they can be accommodated within the restrictions imposed by the code, changes to frame stiffness or structural weight have a similar impact on expected losses for the two frame types. The magnitude of the LIF_S is greater than 1 for the MRFs, suggesting that unlike the CBFs, it is beneficial to invest in the initially more expensive, lower- q value designs.

Table 6-9 Comparison of LIF_q , LIF_M , LIF_T and LIF_S for the 2 storey MRFs and CBFs at the Oakland site

	LIF_q (%initial/ q)	LIF_M (%initial/Mg)	LIF_T (%initial/s)	LIF_S (\$lost /\$spent)
MRF	0.70	-0.14	47.36	-2.17
CBF	1.64	-0.13	51.47	-0.90

This trend can be seen more clearly when the approximate extra investment and expected savings are compared on a case-by-case basis. It can be seen that for the MRFs examined the best return on investment (ROI) is achieved for the $q=2$ design.

6.4 Discussion of Results

It is relevant to examine the results presented above in the context of the development of seismic design code methods towards performance-based goals and procedures. When considering a specific structural form like CBFs, any alternative design process (for example the changes in brace overstrength rules proposed by Brandonisio et al. (2012) or displacement based design methods such as those proposed by Calvi and Sullivan (2009)), unless it involves the use of special devices like base isolation or damping devices, will at a fundamental level result in a modified frame with different strength and stiffness properties. For steel building frames, these two properties are closely linked, hence the variations in lifetime costs with design behaviour factor presented above reflect changes in both the lateral stiffness and lateral resistance of the CBF. Due to the high weighting of more frequent earthquakes in which frames behave elastically in the lifetime performance assessment procedure, elastic response was observed to dominate expected lifetime losses. Therefore, for any change in the design process, its impact on lateral stiffness, and hence fundamental period, is of greatest importance from a lifetime performance perspective.

This study investigated the impact of design ductility demand, represented by q , on lifetime losses. However, given the importance of response in the elastic range for lifetime loss calculations and the close relation between strength and stiffness for steel frame buildings, q essentially acts as an indirect proxy for frame stiffness, with increased q leading to reduced frame stiffness, which in turn dictates the expected losses.

The designer has some control over drift-dependant losses as increasing frame stiffness by using a lower q value invariably reduces these. Eurocode 8 imposes drift limits at the SLS. However, these limitations rarely affect low or mid-rise CBF designs and therefore appear to be ineffective at controlling drift-dependant damage, particularly for the more flexible frames examined. This was also noted by Del Gobbo et al. (2018). Drift-dependant losses were observed to be more important for taller frames; emphasising that the importance of design choices that alter stiffness increases with frame height.

On the other hand, the results of this study suggest that the constraints of conventional seismic design offer little scope to control acceleration-dependant lifetime losses, which were observed to dominate losses in low-rise frames. It has been suggested that design codes should attempt to limit acceleration-dependant damage (Del Gobbo et al., 2018). However the results of this study suggest that this would be difficult to achieve simply by optimising CBF strength and stiffness. More flexible, dissipative designs ($q = 4$ or 5) displayed only slightly reduced acceleration-

dependent losses compared to non-dissipative designs ($q = 1$ or 2), and this reduction was not observed in all cases. These results indicate that the greater hysteretic damping experienced by dissipative CBFs is not effective in reducing global accelerations, and that alternative approaches involving further energy dissipation through, for example, added damping devices should be explored instead. Moreover, where reduced acceleration-dependent losses were observed in dissipative frames, this was invariably associated with greater drift-dependant losses, indicating poor coordination of overall seismic performance. While further investigation of this topic is necessary, it is suggested that the range of building heights, seismicity and behaviour factors considered in this study captures most potential parametric variations relevant to CBFs, and that better coordination of all losses may be achievable with other structural forms such as eccentrically-braced frames. In short, the results indicate that by varying q , or alternatively the elastic stiffness, the designer can limit drift dependant losses but cannot limit acceleration dependant losses effectively. It is difficult to see how this can be done for CBFs in general, irrespective of the design process. Fundamentally, lifetime losses are linked to elastic response, therefore only through changing the elastic response can expected losses be controlled,

Although the results of this study show that losses to acceleration-sensitive components dominated total losses in many cases, Hwang and Lignos (2017a) note that, given the damage observed in the Christchurch earthquakes, it is possible that these losses are overestimated in the FEMA P-58 methodology. Furthermore, studies comparing numerical model outputs to experimental results (Ryan et al., 2017) have shown that earthquake response simulation models, such as the OpenSees models using established modelling assumptions employed here tend to overestimate peak floor acceleration at lower ground motion intensities. Given the extent of their contribution to overall lifetime losses, further calibration of these key steps in the computation of acceleration-dependent losses is desirable.

As shown in Figure 4.3(b), performance based seismic design is an iterative process where a structure is designed and then its ability to meet various performance objectives is assessed, through some performance assessment methodology. Second generation PBEE methods propose using financial performance objectives, as the results are more easily understandable for all stakeholders. For example, a building could be designed to achieve the performance objectives that maximum expected lifetime losses should be less than 15% of the building's initial cost and expected losses of less than 30% of initial cost at the DLE. Therefore, the process of loss assessment, as undertaken in this Chapter, is an integral part of performance-based design procedures.

This Chapter involved examining the expected losses for a range of potential structural designs for various CBFs. The results show important trends in the losses experienced by of CBFs, and these trends can be used to inform performance-based designs. For example, if reducing lifetime losses is a performance objective, the results here show that the way to achieve this is through increased elastic stiffness. This is not an instantly obvious; the designer may be tempted to try to reduce lifetime losses by reducing accelerations through reducing the elastic stiffness or utilizing the ductility capacity. However, the results presented here suggest such an approach would be futile.

The practical implementation of this iterative performance-based design procedure is hampered by the difficulty and computational expense of loss assessment. To make this, and therefore performance-based design, feasible, there is a need to develop methods to quickly assess losses, or alternative performance measures, without the need for computationally expensive time history analysis. If this can be done, the iterative performance-based design framework shown in Figure 4.3(b) can become a viable design philosophy.

Computed lifetime costs, particularly for the 10 storey CBFs, were observed to be highly sensitive to the inter-storey drift level at which damage is first accounted for in the brace fragility function used in damage analysis. For the results presented above, the FEMA P-58 fragility and consequence functions for “Special Concentric Braced Frame with HSS braces, tapered gusset plates & design to AISC minimum standard, Single Diagonal Brace” (ID = B1033.022), where the first damage state has a median drift value of 0.37%, was used in all analyses. However, if the “Special Concentric Braced Frame, design to AISC minimum standards, Single Diagonal Brace” fragility and consequence functions (ID B1033.032), where the first damage state has a median drift value of 0.74%, are used instead, significantly lower drift sensitive losses can be obtained. This is illustrated in Figure 6-30 for the 10 storey CBFs at the Oakland site. Both of these brace fragility functions are single-parameter fragility functions based solely on inter-storey drift; dual parameter fragility functions which can also account for the influence of brace slenderness or width-to-thickness ratio on the probability of different levels of damage have also been developed (Lignos and Karamanci, 2013), but have not been implemented in PACT. However, using such fragility functions may capture brace damage more realistically, and consequently give a better estimate of incurred losses.

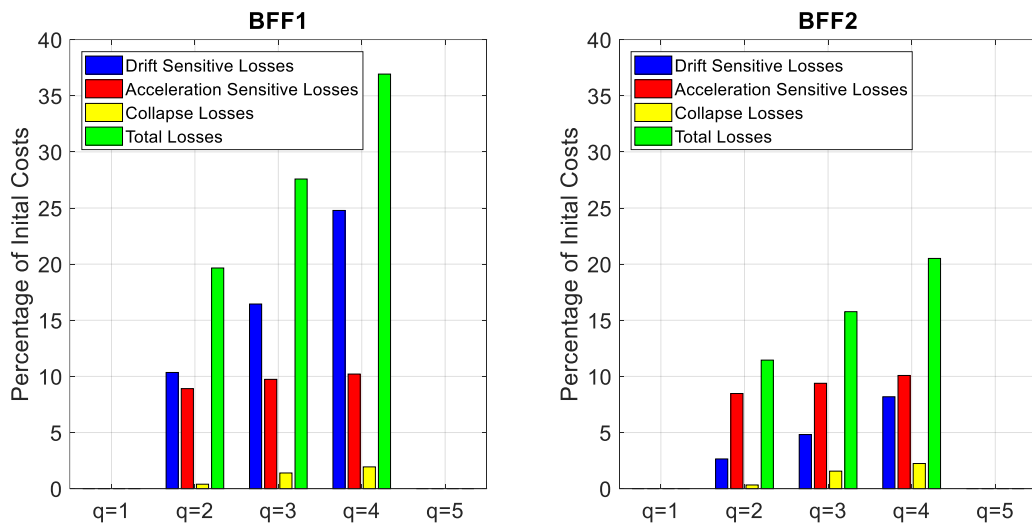


Figure 6-30 Influence of choice of Brace Fragility Function (BFF1 = FEMA P-58 ID B1033.022, BFF2 = FEMA P-58 ID B1033.032) on lifetime losses for the 10 storey CBFs at the Oakland site.

Lifetime performance metrics have been calculated using ten NLTHAs at eight intensity levels. While this combination of analyses and intensity levels was shown to give acceptably accurate performance metrics for a two-storey structure, it is possible that more analyses are required to achieve the same level of accuracy for the taller building frames. Specifically, a more analyses are likely to provide a better prediction of inelastic response under high intensity earthquakes. However, because of the higher weighting of more frequent, less intense events in lifetime performance assessment, the impact on lifetime performance metrics is likely to be minimal. Similarly, collapse fragility functions were developed from a limited-suite analysis method involving only ten ground motion records at a limited number of intensity levels. It is generally accepted in literature that more robust collapse fragility functions can be developed from more comprehensive IDA type analysis using in the order of 40 ground motion records, with more sophisticated methods of selecting analysis intensity levels (Vamvatsikos and Cornell, 2002). Also, as discussed, the definition of collapse can impact the collapse fragility functions calculated. Furthermore, no deteriorating components were employed in the structural model and the lateral stiffness contribution of gravity framing elements is not accounted for in the modelling process, which can lead to conservative predictions of collapse and losses at high intensity levels (Hwang and Lignos, 2017a). However, even with this conservative approach, the impact of collapse on overall lifetime losses was found to be minimal, suggesting that more elaborate structural and seismic response modelling procedures are not necessary for lifetime loss assessment.

Loss assessment was performed here for single diagonal brace frame designs only, no consideration was given to other potential frame layouts such as X-braced, V-braced or chevron

braced frames. While there are some changes in the design assumptions, there are not expected to be many differences in the overall trends in lifetime losses. As a general rule, the influence of q on the expected lifetime losses would most likely be similar for all braced frame configurations. It is likely that all braced frames, irrespective of the exact bracing layout, would be stiff structures with fundamental periods in a similar range to those examined here. Therefore, increasing q would reduce the frame stiffness and, given that lifetime losses are dominated by elastic response, this would most likely lead to an increase in losses with q .

The assumption that increasing q would reduce frame stiffness is based on the fact that the design of stiff braced frames is unlikely to be affected by the drift limitations or P-Delta effects. This is in contrast to many MRFs, where the influence of q can be slightly different. This is particularly true in MRFs when P-Delta effects become influential; in this case the design load is amplified to an increasing extent with q and higher q values often require larger structural members (for example see the MRF designs to EC8 in Macedo et al. 2019).

6.5 Conclusions

Lifetime performance assessment was performed for a set of CBFs designed using behaviour factors ranging from 1 to 5. These values capture the feasible range of global stiffness and strength properties for frames designed using Eurocode 8. Three different building heights and two different seismic hazard sites were considered. Performance was measured using estimated repair costs and business downtime, which were calculated using the FEMA P-58 performance assessment methodology.

This assessment demonstrated that total expected losses are minimised in non-dissipative designs employing the lowest behaviour factors in design. For the 2 and 5 storey frames, losses are generally dominated by damage to acceleration-sensitive non-structural components, with drift-induced damage being more significant for the 10 storey frames. The results demonstrate that reducing the behaviour factor limits drift-sensitive losses, and as this becomes more significant with frame flexibility and height, the impact of the behaviour factor on performance becomes more pronounced. In contrast, the value of the behaviour factor had relatively little influence on acceleration-sensitive losses.

This study considered economic loss due to both repair cost and downtime. The results for total lifetime repair costs are similar to those previously obtained for SCBFs designed to US codes (Hwang and Lignos, 2017a). The additional downtime costs were observed to be always less than the repair costs, but for the tallest buildings represented over one-third of total costs. Collapse

or demolition-related costs did not make a significant contribution to overall costs. Comparing severe and medium level seismic hazard sites, it was shown that lifetime losses are greater for the severe seismic hazard site, primarily due to the increased damage suffered under frequently occurring seismic events. Total cost, as a proportion of initial cost, was observed to increase with behaviour factor and number of storeys, with the total lifetime cost penalty from using dissipative rather than non-dissipative design observed to be as high as one-sixth of initial costs for the tallest building at the higher seismicity site.

More generally, this work explores how performance assessment methodologies based on response simulation can be employed to improve and optimise the seismic design of building structures. The results obtained indicate the extent to which inter-storey drift, and consequently damage and losses suffered by drift-sensitive components, can be limited by varying the stiffness or strength of the structure, in this study through changing the behaviour factor. In contrast, the designer appears to have little control over lifetime losses suffered by acceleration sensitive elements, at least for typical low- and mid-rise CBFs that lie in or near the constant acceleration region of the design spectrum.

7 Rapid Performance Assessment of CBF Structures

7.1 Introduction

Chapter 6 demonstrated how performance assessment procedures can be used to obtain a wealth of information that can inform design decisions, thereby improving, and potentially optimising, the design of CBF structures. However, the practical application of performance assessment and performance-based design methodologies, such as FEMA P-58, is hampered by the considerable time, costs and expertise required to perform NLTHA, meaning that such practices are generally limited to academia (Günay and Mosalam, 2013). Therefore there is a need to develop simplified yet reliable methods for the estimation of the response parameters used in performance assessment (Malaga-Chuquitaype and Elghazouli, 2012).

This chapter attempts to address this challenge. The aim is to develop and validate procedures to carry out ‘rapid’ performance assessment without the need for NLTHA. In order to do this, equations are developed to predict peak drift and acceleration response for CBFs designed to Eurocode 8 (CEN, 2004). As structural response is dependent on the particular details of the design methodology employed, the unique design rules imposed for CBFs in Eurocode 8, primarily the brace slenderness and overstrength limitations, mean that specific equations for CBFs designed to Eurocode 8 can give improved predictions for *EDPs* of interest.

Various models for predicting *EDPs* of interest were discussed in Chapter 4. These include various empirical methods and models developed from analysis of SDOF systems. However, according to Kumar et al. (2013), the most realistic *EDP* prediction methods are those developed from models of multi-storey frames designed to a specific code or design philosophy. Therefore, in this study, a large number of NLTHAs are carried out for different frames designed to Eurocode 8 in order to develop a databank of *EDPs* representative of various earthquake scenarios. The data obtained is then used to develop equations to predict *EDPs*, which can then be used for performance assessment.

The Chapter begins by detailing the development of this *EDP* dataset through NLTHA. Equations are fitted to this data, and compared to various models developed elsewhere. Finally the *EDPs* predicted by the equations are used to carry out ‘rapid’, i.e. non-NLTHA, performance assessment for case study buildings. This is done for some of the case study buildings that were examined using NLTHA in the previous Chapter. Thus the *EDPs*, and the subsequent performance metrics, obtained using the proposed ‘rapid’ method are compared with those obtained using comprehensive NLTHA-based performance assessment.

7.2 Proposed Rapid Performance Assessment Framework

Figure 7-1(a) illustrates the key steps necessary for performance assessment using the NLTHA, as employed in Chapter 6. The role of NLTHA in this is to obtain *EDPs*, namely peak inter-storey drift, peak floor acceleration and residual inter-storey drift, at various intensity levels. These *EDPs* are then used in the performance assessment calculation.

Figure 7-1(b) outlines the alternative ‘rapid’ performance assessment framework proposed, where the *EDPs* are obtained from a simplified analysis procedure, in this case based on a set of predictive equations. Thus the performance measures that can inform and improve design can be obtained whilst avoiding the need for time consuming and complex NLTHA.

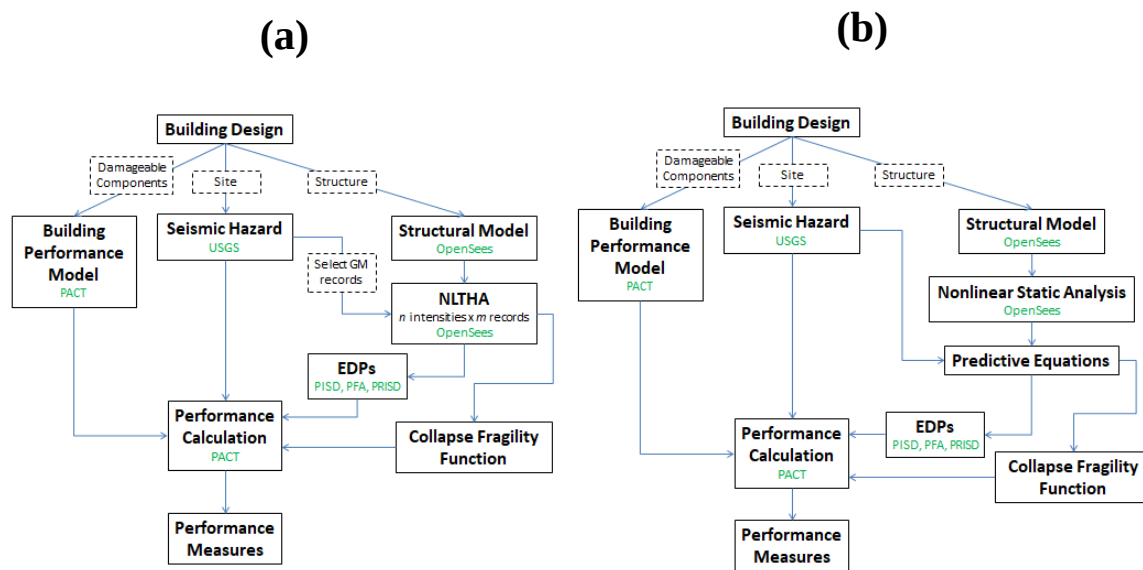


Figure 7-1 (a) Performance assessment procedure employed in Chapter 6 and (b) proposed ‘rapid’ performance assessment procedure developed here

7.2.1 FEMA P-58 Simplified Analysis Procedure

As mentioned in Chapter 4, FEMA P-58 contains a simplified analysis procedure. Equations, which use linear structural models and the lateral yield strength of the structure, are proposed to estimate peak inter-storey drift and peak floor acceleration. These *EDPs* are then used within the regular performance assessment framework to calculate performance metrics. However, these equations are not particular to any design code or methodology, and therefore can be improved upon.

PACT can be used to carry out simplified performance assessment. When using PACT for NLTHA-based assessment, the user inputs *EDPs* at each storey level obtained from each NLTHA. For the simplified procedure, the primary inputs are distributions representing peak inter-storey drift

and peak floor acceleration at each intensity level of interest. These distributions are defined by a median value of each *EDP* at each floor level and a dispersion term, which is constant over the building height.

The two other structural response inputs for performance assessment within the FEMA P-58 framework relate to collapse and residual inter-storey drift. As briefly touched on in Chapters 4 and 6, FEMA P-58 proposes an approximate method for developing collapse fragility functions, based on the design base shear, which doesn't involve NLTHA. Likewise, as discussed in Chapter 6, peak residual inter-storey drift can be predicted based on peak inter-storey drift, as per Equation 6.1, again without the need for NLTHA.

Therefore, the goal of this chapter is to develop methods to predict peak inter-storey drift and floor acceleration at each floor level, as well as a variation term that describing record-to-record and modelling uncertainty. Performance assessment can then be carried out using these distributions within the simplified method in PACT. As these methods are developed specifically for CBF structures designed to Eurocode 8, it is envisaged that better accuracy in *EDP* prediction, and subsequently in the calculation of performance measures, will be achieved compared to more generally applicable *EDP* prediction methods.

7.3 Dataset Development

A set of 2, 3, 4, 5, 6 and 7 storey CBFs illustrated in Figure 7-2, were designed to Eurocode 8 and modelled in OpenSees. NLTHA was performed for each of these frames for multiple potential earthquake scenarios and relevant *EDPs* recorded. Thus a large dataset of *EDPs* capturing CBF response for a wide range of scenarios was developed. The details of this work are outlined in the following sections.

7.3.1 Structural Design & Modelling

The structures were designed in accordance with Eurocode 3 (CEN, 2005) and Eurocode 8. The frames were initially designed for gravity loads according to Eurocode 3, with unfactored permanent and variable loads of 3 kN/m² and 3.3 kN/m² applied at each floor level and 2.5 kN/m² and 0.6 kN/m² at the roof level. Earthquake resistant design was then carried out in accordance with Eurocode 8. Seismic design forces were calculated using the Eurocode 8 Type 1 spectrum, and design was carried out using a behaviour factor of $q=4$. For each frame height, this was done using four different combinations of soil type and design ground acceleration, a_g , as summarised in Table 7-1, meaning a total of 24 different concentrically braced frames were designed and analysed.

Bay widths and storey heights were kept constant at 6m and 3.6m respectively, except where central columns were used meaning that braced bay widths were halved, as can be seen in Figure 7-2. Buildings were designed to be square in plan with seismic resistance provided by perimeter braced frames. UK steel profiles were used for the beams and columns, while the bracing members selected were either rectangular or square hollow sections, all with assumed yield strength of 275 N/mm².

The design actions to be resisted by the structural members were calculated through elastic analyses using the Eurocode 8 lateral force method for regular structures. The lateral resistance of the structure was assumed to be provided by the tension diagonals only. Based on the results of this analysis, bracing members were designed to resist axial forces due to the seismic design forces. The Eurocode 8 non-dimensional slenderness limitations and overstrength differential criteria were obeyed in the brace design procedure. Columns and beams were capacity designed to resist the combination of gravity and seismic actions stipulated by Eurocode 8, with member sizes kept constant at each level.

The combination of the brace slenderness and overstrength limitations, as opposed to the axial design force, was the controlling factor in the selection of brace sizes in some cases. Ductility Class High (DCH) requirements were followed, hence all bracing members possess Class 1 cross sections, as defined by Eurocode 3, to ensure sufficient ductility capacity for dissipative behaviour. The EC8 deformation checks for second order effects and damage limitation requirements were also performed as part of the design process, but as is the case for many CBF structures due to their high stiffness, these did not impact any of the designs. Details of the designs and the resulting structural characteristics are given in Tables 7-1 and 7-2.

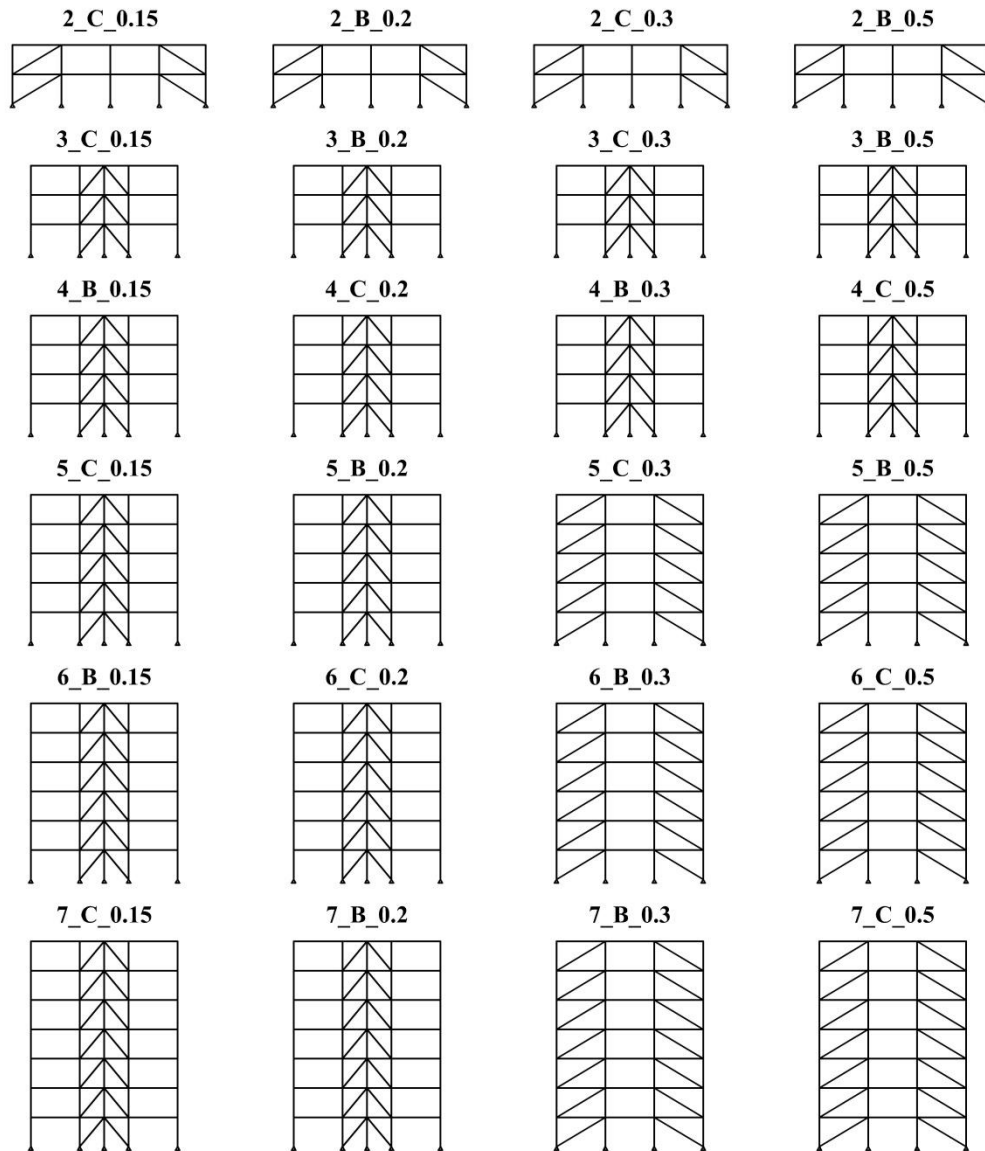


Figure 7-2 Elevations of frames analysed: bay widths = 6m, storey heights = 3.6m

Table 7-1 Design details and structural characteristics of frames analysed: first mode mass participation ratio (γ), minimum and maximum non-dimensional brace slenderness (λ), column to brace stiffness ratio β_k (as defined by Karavasilis et al. (2007)) and β_3 , relative storey stiffness (as defined by Kumar et al. (2013))

Frame ID	Storeys	a_g (g)	EC8 Design Soil Class	T_1 (s)	Height (m)	λ_{min}	λ_{max}	γ	β_k	β_3
2_B_0.5	2	0.5	B	0.20	7.2	1.71	2.99	0.87	0.011	1.17
2_C_0.3	2	0.3	C	0.26	7.2	1.48	3.22	0.88	0.010	1.09
2_B_0.2	2	0.2	B	0.31	7.2	1.72	3.05	0.88	0.009	1.11
2_C_0.15	2	0.15	C	0.35	7.2	1.71	2.99	0.87	0.013	1.17
3_B_0.5	3	0.5	B	0.22	10.8	1.02	1.63	0.80	0.012	1.14
3_C_0.3	3	0.3	C	0.29	10.8	0.98	1.77	0.82	0.011	0.99
3_B_0.2	3	0.2	B	0.34	10.8	1.40	1.73	0.82	0.014	1.05
3_C_0.15	3	0.15	C	0.39	10.8	1.54	1.98	0.83	0.019	0.96
4_C_0.5	4	0.5	C	0.26	14.4	0.92	1.68	0.73	0.019	1.30
4_B_0.3	4	0.3	B	0.34	14.4	1.01	1.46	0.76	0.019	1.15
4_C_0.2	4	0.2	C	0.42	14.4	0.98	1.72	0.77	0.018	1.22
4_B_0.15	4	0.15	B	0.47	14.4	1.38	1.98	0.78	0.019	1.16
5_B_0.5	5	0.5	B	0.31	18	1.01	1.74	0.78	0.032	0.99
5_C_0.3	5	0.3	C	0.35	18	1.32	1.71	0.76	0.019	0.94
5_B_0.2	5	0.2	B	0.45	18	1.34	1.72	0.74	0.016	1.15
5_C_0.15	5	0.15	C	0.53	18	1.45	1.98	0.73	0.039	1.19
6_C_0.5	6	0.5	C	0.35	21.6	0.98	1.74	0.72	0.024	1.44
6_B_0.3	6	0.3	B	0.42	21.6	1.37	1.96	0.75	0.021	1.07
6_C_0.2	6	0.2	C	0.54	21.6	0.99	1.74	0.71	0.029	1.16
6_B_0.15	6	0.15	B	0.60	21.6	1.14	1.98	0.71	0.030	1.17
7_B_0.5	7	0.5	B	0.42	25.2	1.00	1.72	0.73	0.027	1.14
7_C_0.3	7	0.3	C	0.46	25.2	1.34	1.71	0.74	0.024	1.15
7_B_0.2	7	0.2	B	0.64	25.2	0.89	1.73	0.70	0.028	1.25
7_C_0.15	7	0.15	C	0.70	25.2	1.01	1.98	0.70	0.035	1.25

Table 7-1 introduces three parameters that have not yet been encountered in this study; the mass participation ratio corresponding to the first mode, γ , ratio of column to brace stiffness, termed α by Karavasilis et al. (2007) and β_k by Elghazouli (2010) and here, and the relative storey stiffness, β_3 .

The mass participation ratio corresponding to the first mode is calculated as:

$$\gamma = \frac{(\sum_{j=1}^N m_j \bar{\phi}_{j,1})^2}{(\sum_{j=1}^N m_j \bar{\phi}_{j,1}^2)(\sum_{j=1}^N m_j)} \quad 7.1$$

where $\bar{\phi}_{j,1}$ is the value of the normalized first mode shape at level j , m is the mass at level j and N is the number of storeys.

The ratio of column to brace stiffness is calculated as:

$$\beta_k = \frac{n_c I_c L_{br}}{n_{br} A_{br} h^3 \cos^2 \theta} \quad 7.2$$

Where I_c is the second moment of area of the columns, L_{br} is the length of the bracing member, A_{br} is the cross sectional area of the bracing member, h is the storey height, θ is the angle between the brace and the beam, and n_c and n_{br} are the number of columns and braces respectively. As the value of β_k changes over the frame height, the value at the middle level is used for this study (Karavasilis et al., 2007). Previous studies have shown that a higher column to brace stiffness ratio allows better redistribution of the drifts after yield has occurred (Elghazouli, 2010), and hence limits soft storey type behaviour.

The relative storey stiffness, β_3 , defined by Kumar et al. (2013), is obtained using the inter-storey drift profile corresponding to the first model shape of the structure obtained from an Eigenvalue analysis. It is calculated as the ratio of maximum inter-storey drift for the upper half of the frame to value for the lower half:

$$\beta_3 = \frac{\max(\Delta\varphi_i)_{uh}}{\max(\Delta\varphi_i)_{lh}} \quad 7.3$$

Where $\Delta\varphi_i$ is the inter-storey drift at floor level i determined using the inter-storey drift profile for the first mode shape and the subscripts uh and lh correspond to the upper and lower halves of the frame. For a frame with an odd number of storeys the upper half is rounded to the lower number, for example for a seven storey frame the fourth storey is considered to be in the lower half. β_3 is intended to capture the impact of comparatively weak or flexible storeys experiencing early yielding and subsequently higher drift demands.

Table 7-2 Details of beam, column and brace section sizes selected for frames

Level	Beam (UKB)	Column (UKC)	Brace (SHS or RHS)	Beam (UKB)	Column (UKC)	Brace (SHS or RHS)	Beam (UKB)	Column (UKC)	Brace (SHS or RHS)	Beam (UKB)	Column (UKC)	Brace (SHS or RHS)
	2_C_0.15			2_B_0.2			2_C_0.3			2_B_0.5		
2	305x165x40	152x152x51	70x70x3.6	305x165x40	203x203x46	70x70x5	305x165x40	203x203x60	70x70x8	305x165x40	203x203x86	80x80x12.5
1	305x165x40	152x152x51	120x120x4	305x165x40	203x203x46	120x120x5	305x165x40	203x203x60	140x140x6.3	305x165x40	203x203x86	200x120x10
	3_C_0.15			3_B_0.2			3_C_0.3			3_B_0.5		
3	305x165x40	152x152x37	70x70x3	305x165x40	203x203x46	80x80x3	305x165x40	203x203x46	80x80x5	305x165x40	203x203x46	90x90x8
2	305x165x40	203x203x46	90x90x4	305x165x40	203x203x60	100x100x5	305x165x40	203x203x60	140x140x5	305x165x40	254x254x89	140x140x10
1	305x165x40	203x203x52	90x90x5	305x165x40	203x203x71	100x100x6.3	305x165x40	203x203x86	140x140x6.3	305x165x40	254x254x132	140x140x12.5
	4_B_0.15			4_C_0.2			4_B_0.3			4_C_0.5		
4	305x165x40	152x152x37	70x70x3	305x165x40	152x152x44	80x80x3.2	305x165x40	152x152x51	100x100x4	305x165x40	203x203x46	140x80x6
3	305x165x40	152x152x51	100x100x4	305x165x40	203x203x46	90x90x6.3	305x165x40	203x203x60	100x100x8.8	305x165x40	203x203x86	140x140x10
2	305x165x40	203x203x60	90x90x6.3	305x165x40	203x203x71	140x140x5	305x165x40	203x203x100	140x140x8	305x165x40	254x254x132	300x100x10
1	305x165x40	203x203x86	90x90x7.1	305x165x40	203x203x100	90x90x10	305x165x40	254x254x132	200x120x8	305x165x40	356x406x235	250x150x12
	5_C_0.15			5_B_0.2			5_C_0.3			5_B_0.5		
5	305x165x40	152x152x37	70x70x3	305x165x40	203x203x46	80x80x3.6	305x165x46	152x152x51	120x120x4	406x178x60	203x203x46	120x120x5
4	305x165x40	152x152x51	90x90x5	305x165x40	203x203x71	120x120x5	305x165x46	203x203x60	200x120x6	406x178x60	203x203x71	200x120x8
3	305x165x40	254x254x73	90x90x7.1	305x165x40	254x254x107	120x120x7.1	305x165x46	254x254x167	140x140x8.8	406x178x60	254x254x167	200x150x10
2	305x165x40	305x305x97	90x90x8.8	305x165x40	305x305x137	120x120x8.8	457x191x133	254x254x167	250x150x8	457x191x133	254x254x167	250x150x10
1	305x165x40	356x368x129	100x100x8	305x165x40	356x368x177	140x140x8	305x165x46	305x305x198	250x150x8	406x178x60	305x305x240	250x150x12.5
	6_B_0.15			6_C_0.2			6_B_0.3			6_C_0.5		
6	305x165x46	152x152x37	70x70x3	305x165x46	152x152x44	80x80x3.6	305x165x54	152x152x51	120x120x4	457x191x98	203x203x46	120x120x6.3
5	305x165x40	152x152x51	120x120x4	305x165x40	203x203x46	120x120x5	305x165x54	203x203x60	140x140x7.1	457x191x98	203x203x100	140x140x12.5
4	305x165x40	203x203x71	90x90x8	305x165x40	203x203x86	140x140x6.3	305x165x54	254x254x107	120x120x12.5	457x191x98	254x254x167	250x150x12.5
3	305x165x40	254x254x89	90x90x10	305x165x40	254x254x107	140x140x8	305x165x54	305x305x198	140x140x12.5	457x191x98	356x406x287	300x200x12
2	305x165x40	254x254x132	100x100x10	305x165x40	254x254x167	140x140x8.8	457x191x133	305x305x198	300x100x10	457x191x133	356x406x287	400x200x12
1	305x165x40	305x305x158	90x90x12.5	305x165x40	305x305x198	140x140x10	305x165x54	356x406x235	250x150x12	457x191x98	356x406x393	400x200x12
	7_C_0.15			7_B_0.2			7_C_0.3			7_B_0.5		
7	305x165x40	152x152x37	70x70x3	305x165x40	203x203x46	80x80x3.2	356x171x67	203x203x52	120x120x4	406x178x74	203x203x60	120x120x5
6	305x165x46	152x152x51	120x80x5	305x165x46	203x203x46	120x80x6	356x171x67	203x203x52	200x120x6	406x178x74	203x203x60	200x120x8
5	305x165x40	203x203x71	100x100x7.1	305x165x40	203x203x86	100x100x8.8	356x171x67	203x203x113	200x120x10	406x178x74	203x203x127	140x140x12.5
4	305x165x40	203x203x100	120x120x7.1	305x165x40	203x203x113	200x150x6	356x171x67	254x254x167	250x150x10	406x178x74	254x254x167	250x150x12
3	305x165x40	254x254x132	90x90x12.5	305x165x40	254x254x167	140x140x8.8	356x171x67	305x305x240	250x150x12	406x178x74	356x406x287	300x200x10
2	305x165x40	305x305x158	140x140x8	305x165x40	305x305x198	140x140x10	457x191x133	305x305x283	250x150x12	457x191x133	356x406x287	300x200x12
1	305x165x40	356x368x177	140x140x8	305x165x40	356x368x202	140x140x10	356x171x67	356x406x287	250x150x12.5	406x178x74	356x406x393	300x200x12

Planar models of the seismic resisting frames were developed in OpenSees (McKenna, 2011). As before, following the recommendations of Uriz et al. (2008), the bracing elements were modelled using two force-based nonlinearBeamColumn elements with three integration points per element. An initial camber displacement of 0.1% of the brace length was employed at the midpoint in order to simulate buckling behaviour. As before, the brace elements were discretised into 12 layers across the depth of the cross section, as illustrated in Figure 6-6 (a). The beams and columns were also modelled using force-based nonlinearBeamColumn elements with 3 integration points and discretized fiber sections. Nonlinear rotational spring elements, with initial rotational stiffness and yield moment calculated according to the expressions proposed by Hsiao et al. (2012), were employed at the brace end point to simulate the end restraint imposed on the braces by the gusset plates. Rigid elements were used to model the

remainder of the gusset plates and the sections of the beams and columns in the region of the connection, as illustrated in Figure 6-6 (b). The Steel02 material, which represents the Giuffre-Menegotto-Pinto model, was used for all nonlinear components. As illustrated in Figure 7-3, a fictitious leaning column element (Geschwindner, 2002) was included in order to account for second order effects due to the gravity loads from parts of the building not included in the OpenSees models.

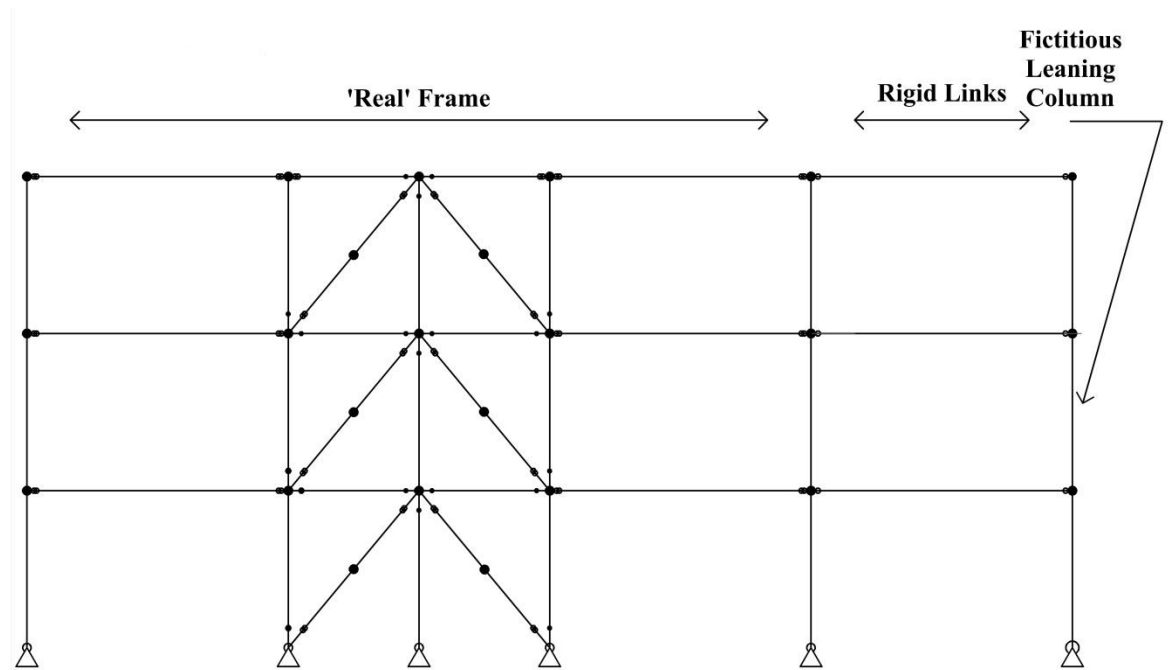


Figure 7-3 OpenSees model of frame 3_B_0.2 including leaning column element

7.3.2 Ground Motion Records

To perform time history analysis, 40 ground motion records were selected from the PEER Strong Motion Database (Chiou et al., 2008), as detailed in Table 7-3. Only one horizontal component from each recording station was selected for use in analysis. As the structures analysed here are not designed for a specific case study site, an MSA approach for record selection was not adopted here, unlike in Chapter 6. Instead the records were chosen to give a wide range of moment magnitude, rupture distance and site class, based on the Eurocode 8 site classification. The distribution of the records with respect to these parameters is shown in Figure 7-4, in which a largely uniform distribution is observable. The rules used by Kumar et al. (2013) to limit the study to far-field records were also followed here; hence the records were chosen with rupture distances in the range of 0–80km for moment magnitudes between 5.5 and 6, and within the range of 20–80 km for magnitudes greater than 6.

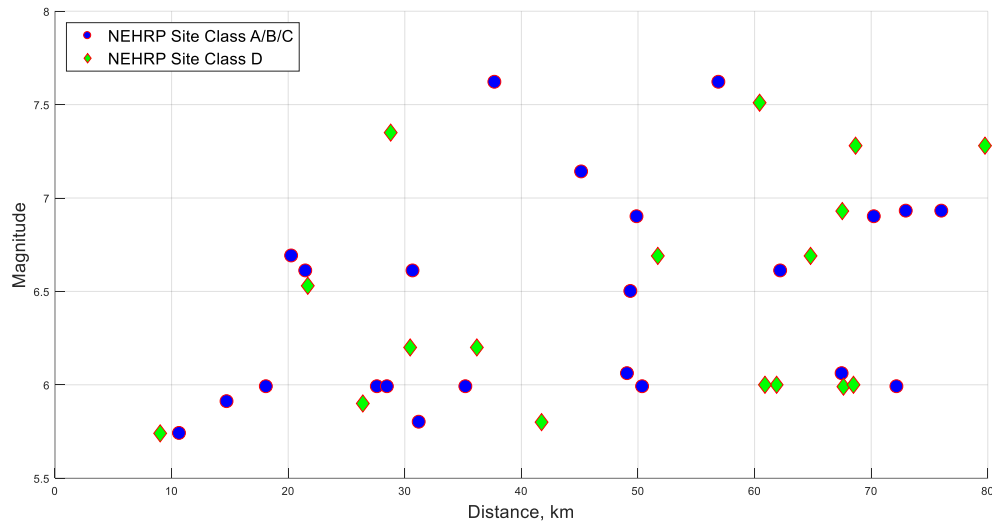


Figure 7-4 Distribution of selected ground motion records with respect to distance and moment magnitude for selected ground motions shown by NEHRP site class (ASCE/SEI, 2010)

The concept of mean earthquake period was briefly discussed in Chapter 4. Although response or Fourier spectra can be used to fully characterize the frequency content of a ground motion record, when comparing response due to different ground motions it is often more convenient to obtain a single scalar parameter to represent frequency content. Various methods have been proposed for this purpose; Kumar et al. (2011) recommend the mean period, T_m , as it related to earthquake magnitude, source-site distance and site conditions. As can be seen in Table 7-3, the ground motion records used in analysis were selected to include a wide range of mean periods, with T_m values ranging from 0.135s to 1.125s.

T_m was originally defined by Rathje et al. (1998) and is determined as the weighted mean of the periods of the Fourier Amplitude Spectrum over a pre-defined frequency range, where the weights are assigned based on the Fourier amplitudes and calculated using the following relationship:

$$T_m = \frac{\sum_i C_i^2 \times \frac{1}{f_i}}{\sum_i C_i^2} \quad 7.4$$

In the above equation, C_i is the Fourier amplitude coefficient corresponding to a frequency f_i obtained from a discrete Fast Fourier Transform. In this study, the calculation was performed for frequencies between 0.25 Hz and 20 Hz, i.e. $0.25\text{Hz} < f_i < 20\text{Hz}$, using a spacing of frequencies equal to 0.05Hz, i.e. $\Delta f = 0.05\text{Hz}$. The concept of T_m is illustrated graphically in Figure 7-5.

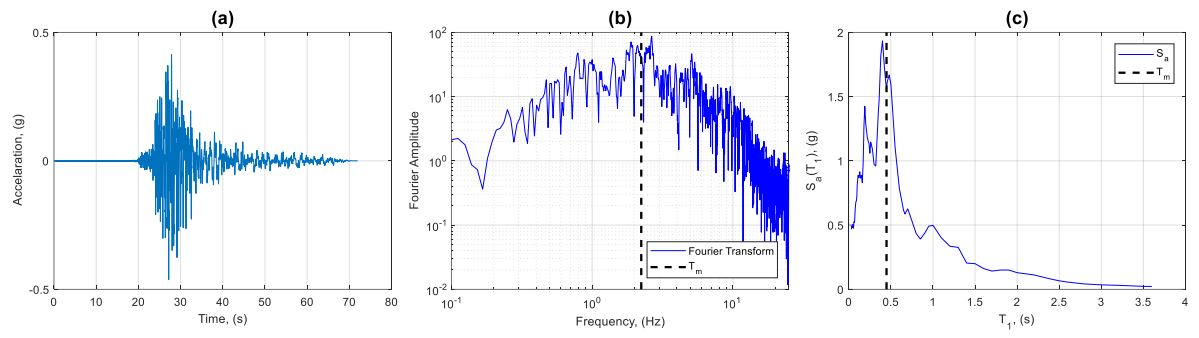


Figure 7-5 Illustration of mean period, T_m ; (a) Ground acceleration time history, (b) Fourier transform of ground acceleration showing T_m and (c) Spectral acceleration showing T_m

Table 7-3 Ground Motion Records used for incremental dynamic analysis

Earthquake Name	Year	PEER Record Sequence Number	Moment Magnitude	R_{rup} (km)	NEHRP Soil Class	Mean Period, T_m (s)
"Chi-Chi Taiwan"	1999	1245	7.62	37.72	B	0.515
"Chi-Chi Taiwan"	1999	1256	7.62	56.93	B	0.684
"Christchurch New Zealand"	2011	8069	6.2	36.18	D	0.599
"Christchurch New Zealand"	2011	8060	6.2	30.46	D	0.486
"Coyote Lake"	1979	146	5.74	10.67	B	0.199
"Coyote Lake"	1979	147	5.74	9.02	D	0.370
"Duzce Turkey"	1999	1620	7.14	45.16	C	0.363
"Friuli Italy-01"	1976	121	6.5	49.38	C	0.454
"Friuli Italy-02"	1976	132	5.91	14.75	C	0.262
"Imperial Valley-06"	1979	172	6.53	21.68	D	0.422
"Kobe Japan"	1995	1102	6.9	49.91	C	0.290
"Kobe Japan"	1995	1109	6.9	70.26	C	0.503
"Kocaeli_ Turkey"	1999	1155	7.51	60.43	D	0.974
"Landers"	1992	860	7.28	68.66	D	0.355
"Landers"	1992	888	7.28	79.76	D	1.125
"Lazio-Abruzzo Italy"	1984	479	5.8	31.23	C	0.334
"Loma Prieta"	1989	743	6.93	67.52	C	0.444
"Loma Prieta"	1989	788	6.93	73	D	0.707
"Loma Prieta"	1989	795	6.93	76.05	B	1.099
"N. Palm Springs"	1986	541	6.06	49.09	C	0.135
"N. Palm Springs"	1986	531	6.06	67.5	C	0.164
"Northridge-01"	1994	1011	6.69	20.29	B	0.323
"Northridge-01"	1994	1066	6.69	64.8	D	0.342
"Northridge-01"	1994	1094	6.69	51.7	D	0.556
"Parkfield-02_ CA"	2004	4080	6	61.89	D	0.237
"Parkfield-02_ CA"	2004	4093	6	68.48	D	0.624
"San Fernando"	1971	87	6.61	30.7	C	0.185
"San Fernando"	1971	80	6.61	21.5	B	0.273
"San Fernando"	1971	94	6.61	62.23	C	0.340
"Tabas Iran"	1978	138	7.35	28.79	D	0.627
"Taiwan SMART1(33)"	1985	486	5.8	41.73	D	0.368
"Taiwan SMART1(5)"	1981	307	5.9	26.4	D	0.502
"Umbria Marche_ Italy"	1997	4353	6	60.89	D	0.798
"Whittier Narrows-01"	1987	598	5.99	28.5	C	0.183
"Whittier Narrows-01"	1987	643	5.99	27.64	B	0.202
"Whittier Narrows-01"	1987	703	5.99	50.39	B	0.206
"Whittier Narrows-01"	1987	653	5.99	67.62	D	0.312
"Whittier Narrows-01"	1987	631	5.99	35.23	C	0.375
"Whittier Narrows-01"	1987	610	5.99	72.2	C	0.428
"Whittier Narrows-01"	1987	680	5.99	18.12	B	0.458

7.3.3 Analysis Procedure

To evaluate the influence of ground motion record and behaviour factor on the *EDPs* experienced by a CBF, an IDA type analysis procedure, similar to that used by Kumar et al. (2013) in their study of drift response, was employed. Firstly, an eigenvalue analysis was performed in

OpenSees to obtain the frame natural period and mode shape. This allowed the mass participation ratio corresponding to the first mode, γ , and the relative store stiffness, β_3 , to be calculated. The frame was then subjected to pushover analysis with a force distribution based on the fundamental mode shape. From this the base shear at the point of first yield termed $V_{bs,yield}$, was obtained.

7.3.3.1 Record Scaling for IDA

Incremental dynamic analysis (IDA) was then carried out by scaling the ground motion records in order to ensure that each frame was evaluated at a consistent level of seismic intensity, taking into account the frame's yield strength and period, T_1 , and different values of the effective behaviour factor, q_{eff} . The scaling factor (SF) applied to each ground acceleration record is given by:

$$SF = \frac{q_{eff} \times V_{bs,yield}}{S_a(T_1) \times \gamma \times m} \quad 7.5$$

where q_{eff} is the behaviour factor, $S_a(T_1)$ is the value of the acceleration spectrum of the unscaled ground motion record at the fundamental period of the structure being examined and m is the structural mass.

Each ground motion record was scaled to achieve q_{eff} values of 1.5, 2, 3, 4 and 5. It should be noted that these are the effective behaviour factor values representing the degree of nonlinearity experienced by the structure as opposed to the design behaviour factor, which was $q=4$ for all frames.

7.3.4 EDP Databank

The records scaled to the various q_{eff} values were then used to perform time history analysis on the OpenSees models of the case study buildings previously described. This meant a large dataset of response parameters was developed from 4800 (40 records x 24 frames x 5 intensity levels) analyses. This dataset is used in the following sections to develop simplified methods for the estimation of response parameters for CBFs designed to EC8.

Figures 7-5 to 7-10 show the peak inter-storey drift (PISD) response for the frames analysed, as well as the median and 16th and 84th percentile values. Brief examination of the data shows that PISD increases with earthquake intensity, as represented by the q_{eff} value used to scale the records for IDA. In some cases for the higher intensity ground motions, particularly for the taller

frames considered, there is evidence of soft storey type behaviour, where drift is concentrated at a single level, generally the top level in these cases.

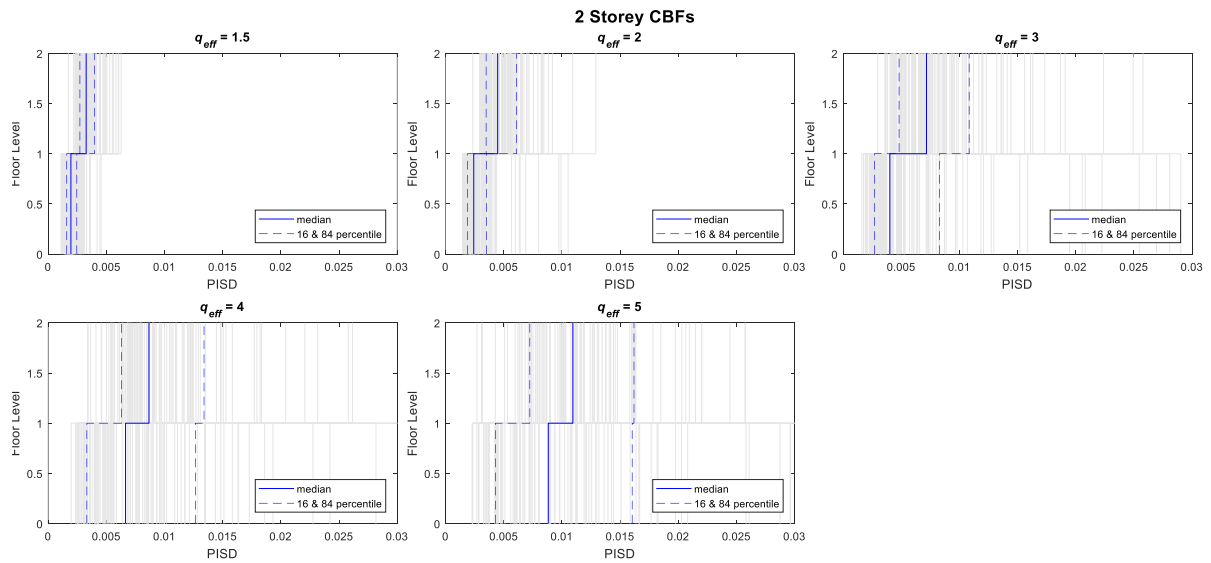


Figure 7-6 Peak inter-storey drift response for 2 storey CBFs for each q_{eff} value

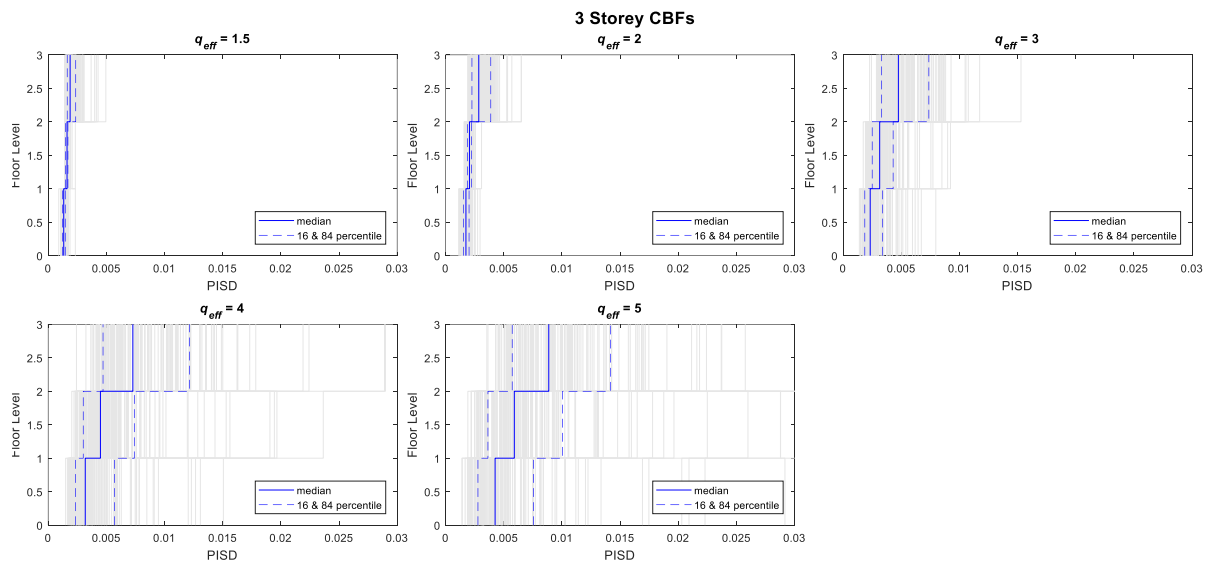


Figure 7-7 Peak inter-storey drift response for 3 storey CBFs for each q_{eff} value

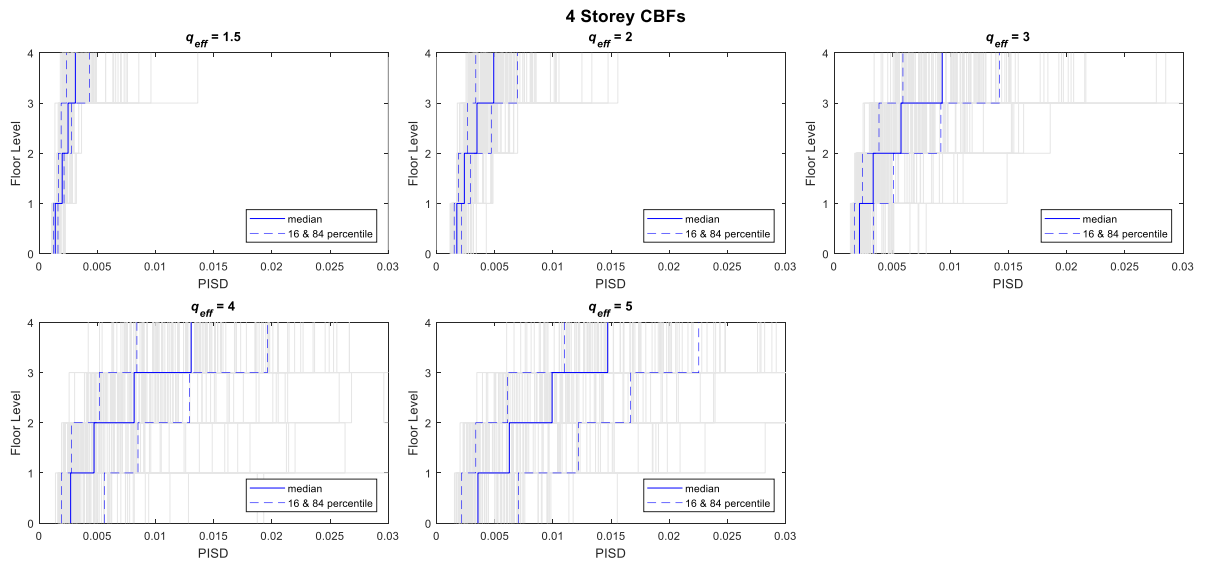


Figure 7-8 Peak inter-storey drift response for 4 storey CBFs for each q_{eff} value

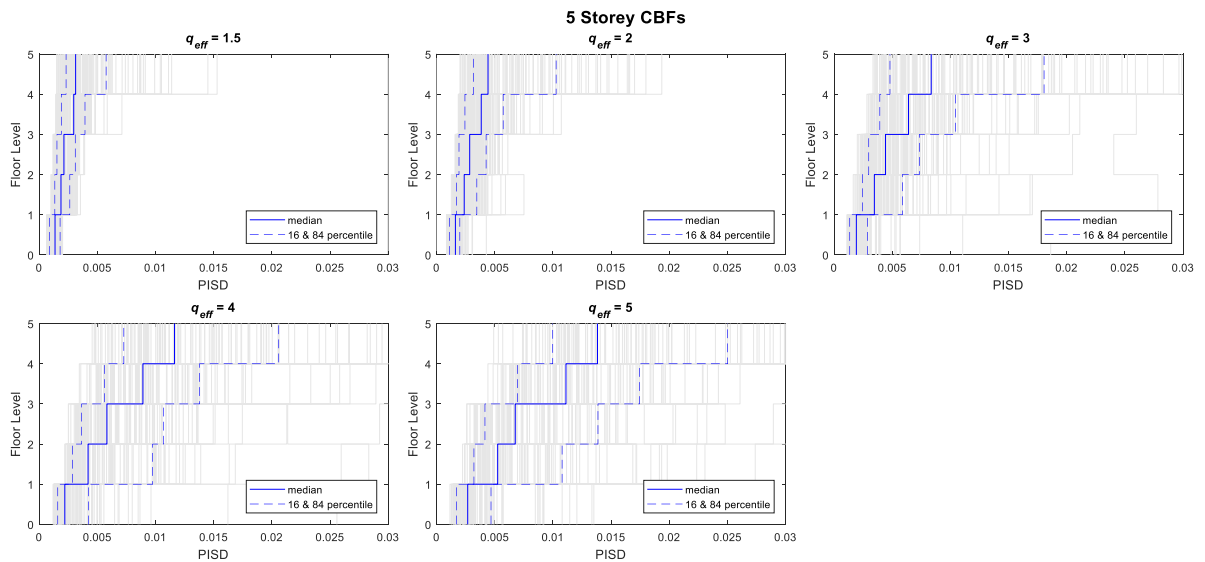


Figure 7-9 Peak inter-storey drift response for 5 storey CBFs for each q_{eff} value

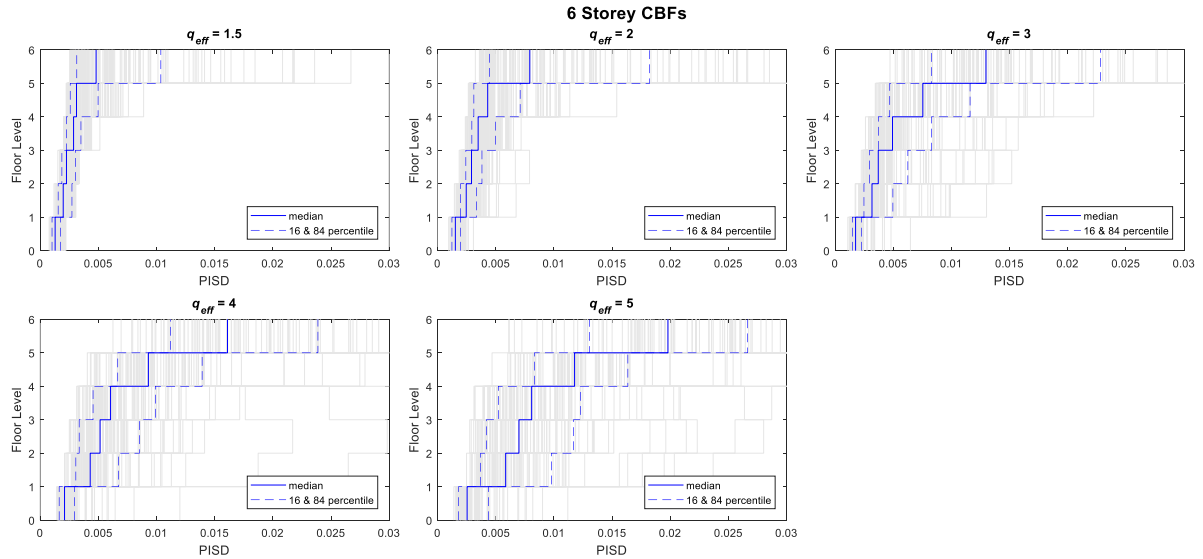


Figure 7-10 Peak inter-storey drift response for 6 storey CBFs for each q_{eff} value

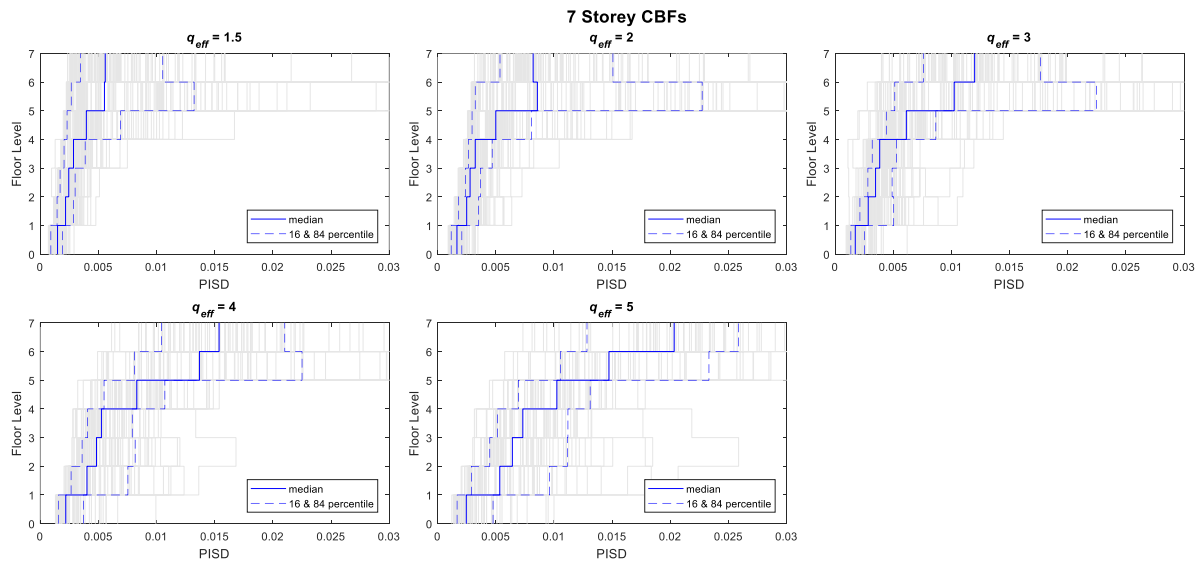


Figure 7-11 Peak inter-storey drift response for 7 storey CBFs for each q_{eff} value

Acceleration response is presented in Figures 7-11 to 7-16. Peak floor acceleration (PFA) is normalized by the peak ground acceleration (PGA). This ratio of PFA to PGA is sometimes termed floor acceleration magnification (Rodriguez et al., 2002). A brief examination of this data shows that the maximum acceleration values generally occur at roof level. Previous studies have demonstrated the existence of a ‘whiplash’ effect for taller buildings, where a significant jump in acceleration is noted at the upper levels (Reinoso and Miranda, 2005); this is noticeable, albeit not particular strongly, for the 7 storey buildings. The floor acceleration magnification is greater for the lower q_{eff} values, with the PFA values becoming closer to the PGA as the degree of nonlinearity increases. This saturation in floor acceleration magnification with increased

nonlinearity, which occurs because acceleration is limited by the yield strength, has been noted elsewhere, for example by Taghavi and Miranda (2012).

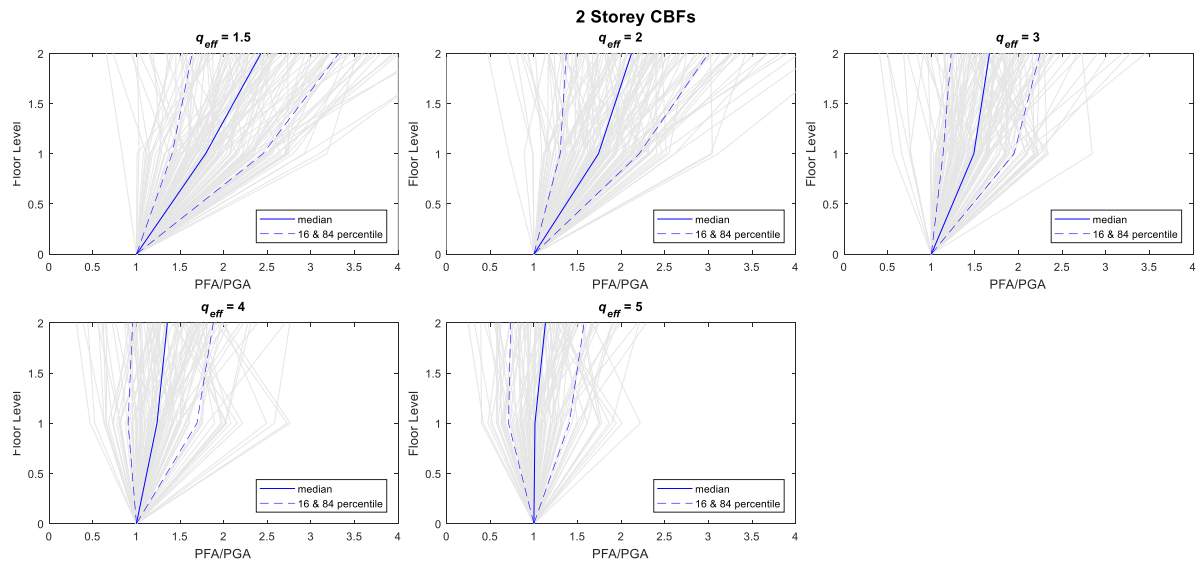


Figure 7-12 Peak acceleration response for 2 storey CBFs for each q_{eff} value

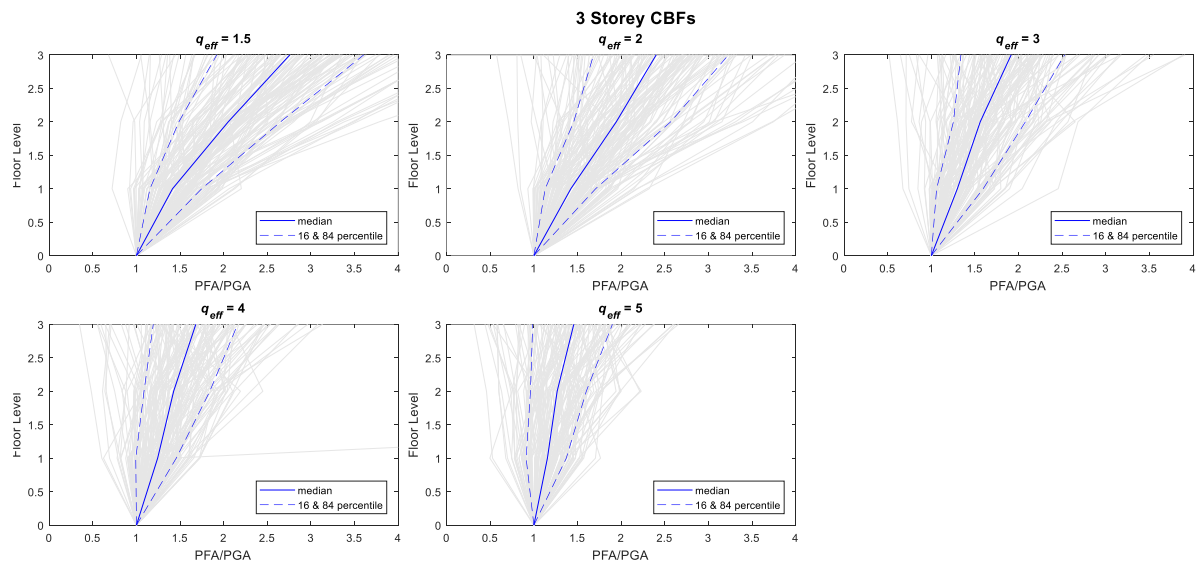


Figure 7-13 Peak acceleration response for 3 storey CBFs for each q_{eff} value

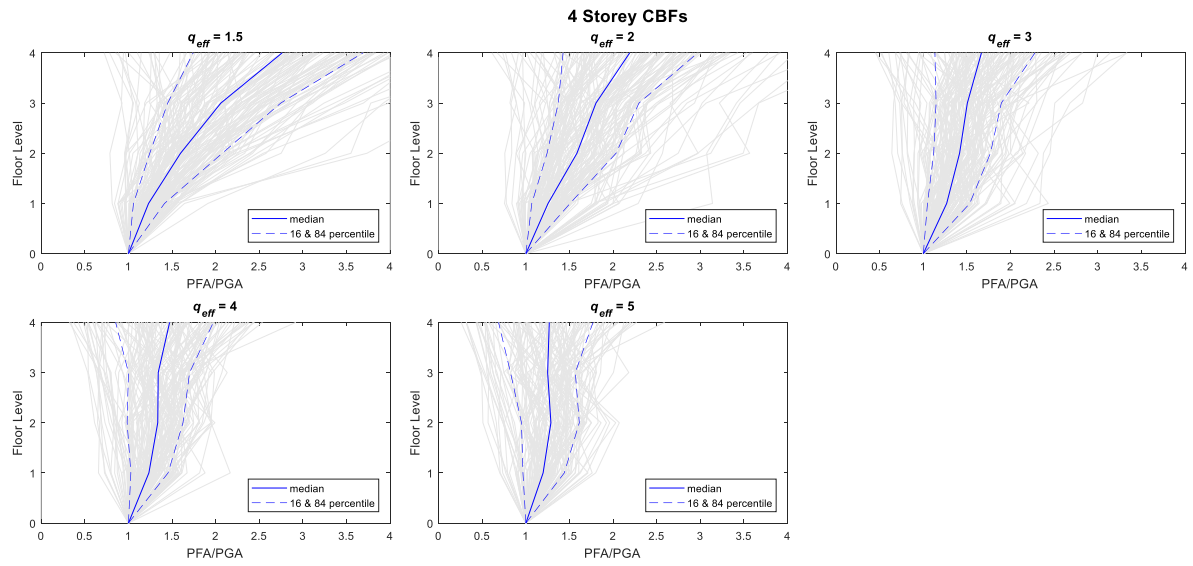


Figure 7-14 Peak acceleration response for 4 storey CBFs for each q_{eff} value

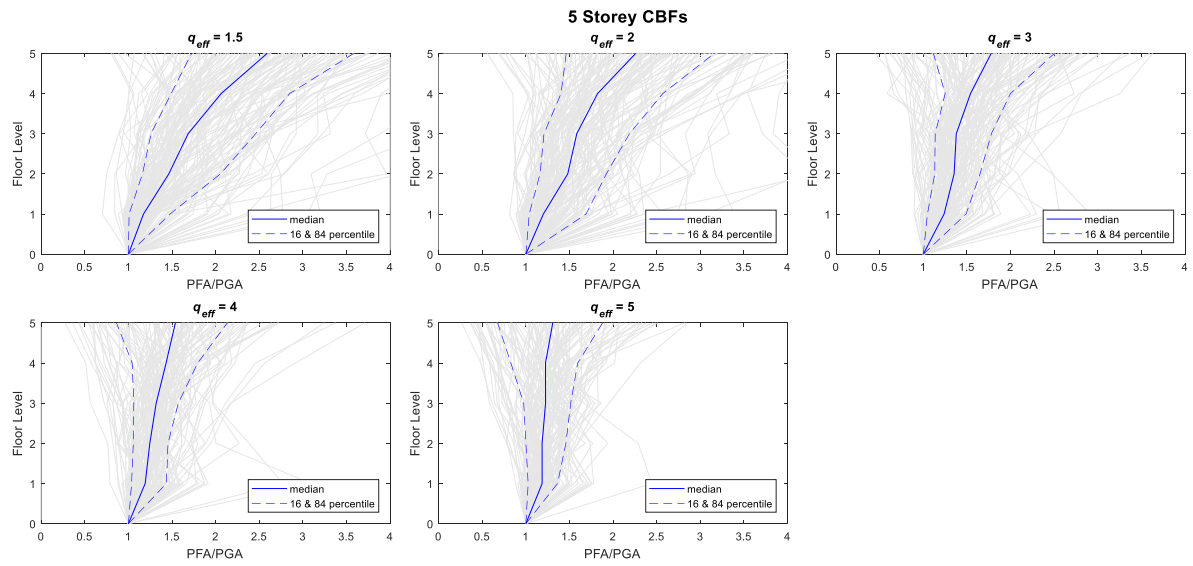


Figure 7-15 Peak acceleration response for 5 storey CBFs for each q_{eff} value

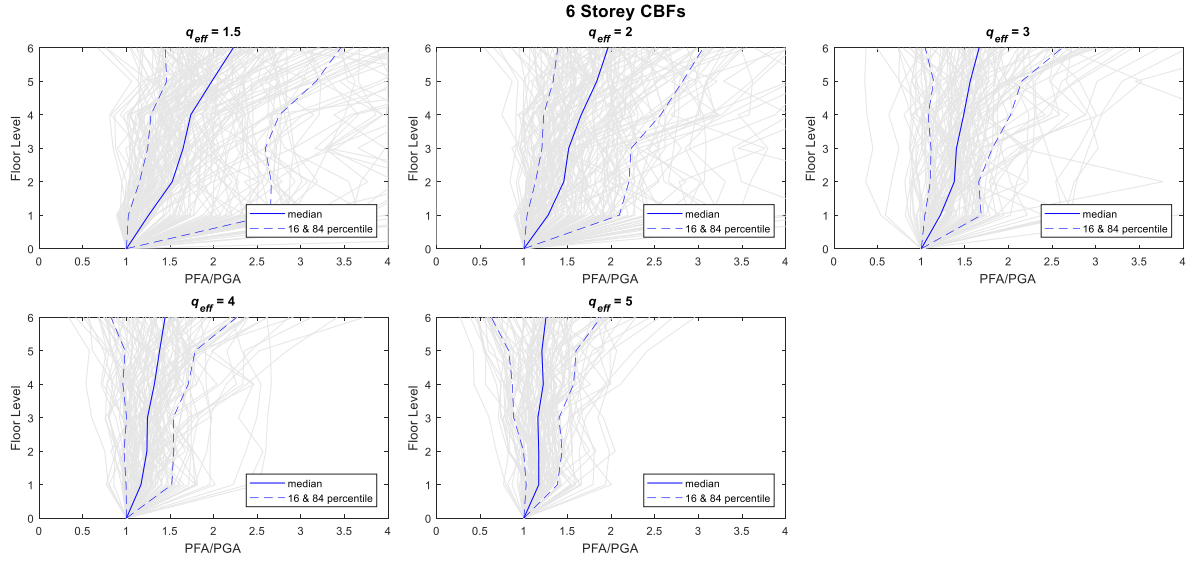


Figure 7-16 Peak acceleration response for 6 storey CBFs for each q_{eff} value

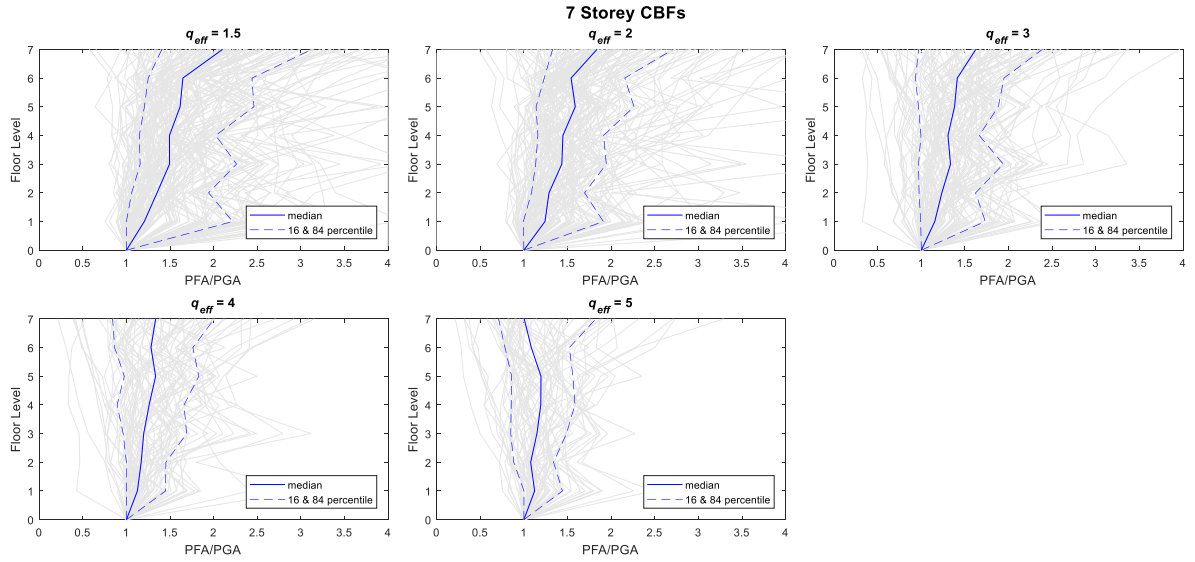


Figure 7-17 Peak acceleration response for 7 storey CBFs for each q_{eff} value

The following sections discuss the fitting of regression equations to this data so that *EDPs* can be predicted without recourse to NLTHA.

7.4 Peak Drift Response Prediction

7.4.1 Normalised Drift Factors

As discussed in Chapter 4, Kumar et al. (2013) performed a large number of dynamic nonlinear time history analyses on MRFs designed to Eurocode 8 and from these developed expressions to predict peak global and inter-storey drift. A global drift modification factor, δ_{mod} , was used to assess roof drift and was calculated using the expression:

$$\delta_{mod} = \frac{\Delta_{max}}{q_{eff} \times \Delta_1} \quad 7.6$$

Where Δ_{max} represents the maximum roof drift recorded in a time history analysis, Δ_1 is the roof displacement at first yield, obtained from pushover analysis, and q_{eff} (termed q in the study) is the value of behaviour factor used to control relative earthquake intensity, as previously discussed. By assuming that the design base shear is equal to the base shear at first yield, it can be shown that EC8 implicitly assumes a constant value of 1 for δ_{mod} .

Similarly a maximum drift modification factor, ϑ_{mod} , was used to assess inter-storey drift:

$$\vartheta_{mod} = \frac{\theta_{max}}{q_{eff} \times \theta_1} \quad 7.7$$

Where ϑ_{max} is the maximum inter-storey drift obtained from time history analysis and θ_1 represents the maximum inter-storey drift at first yield, obtained from pushover analysis. Again, EC8 assumes a constant value of 1 for ϑ_{mod} based on the equal displacements rule.

Multi-variable regression models were proposed to predict δ_{mod} and ϑ_{mod} based on the ratio of fundamental structural period to mean earthquake period, T_1/T_m . Mean earthquake period, T_m , is a parameter used to represent the frequency content of a ground motion.

The following section of the Chapter investigates the relationship between ground motion period and the drift response of CBFs designed to EC8, in a similar manner to which Kumar et al. (2013) examined this relationship for MRFs.

7.4.1.1 Regression Models for δ_{mod} and ϑ_{mod}

Figure 7-18 shows the influence of T_1/T_m on the values of the global drift modification factor, δ_{mod} , and maximum drift modification factor, ϑ_{mod} , obtained with each behaviour factor. A clear trend is evident for δ_{mod} , where the results can be essentially divided into two ranges; those above and below T_1/T_m ratios of 1. For T_1/T_m values greater than 1, the values of δ_{mod} are typically below unity and appear to be relatively insensitive to the exact value of T_1/T_m . However, as T_1/T_m decreases below 1, δ_{mod} increases nonlinearly, with some δ_{mod} values above 4 being observed when $T_1/T_m < 0.5$. Broadly similar trends are observed for ϑ_{mod} . However the magnitudes of the ϑ_{mod} values are generally greater than the δ_{mod} values, while the level of scatter is much greater.

As was pointed out by Kumar et al. (2013), the trends observed here can be explained by resonance between the ground motion and the vibrational periods of the frames. The increase

in δ_{mod} and ϑ_{mod} for lower T_1/T_m values can be attributed to the value of T_m being similar to the elongated post-yield fundamental period, which leads to resonant effects and subsequently high drift demand. Similarly, the slight increase in ϑ_{mod} found at higher period ratios can be explained by resonance between the ground motion and higher modes.

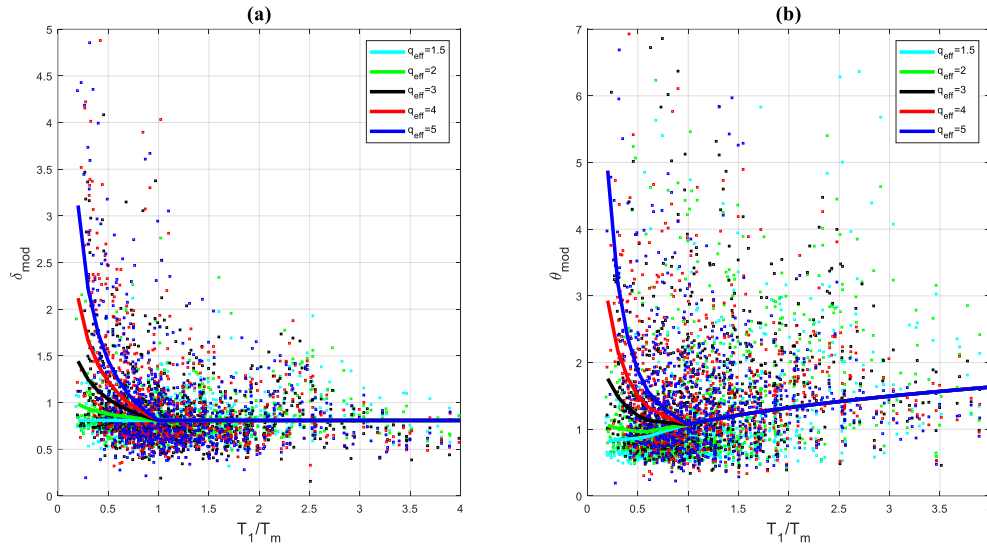


Figure 7-18 (a) Variation in global drift modification factor, δ_{mod} , and (b) Maximum Drift Modification factor, ϑ_{mod} , with T_1/T_m . Regression lines and data points; $q = 2, 3, 4, 5$

These trends are generally similar to those found by Kumar et al. (2013) for MRFs. To facilitate further analysis and for use in the rapid performance assessment framework, a regression model was developed to represent the results shown in Figure 7-18. Given the similar trends, a basic functional form similar to that used by Kumar et al. was adopted:

$$\ln \delta_{mod} = c_0 + c_1 q + (c_2 + c_3 q) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] + c_4 \times \ln \left[\max \left(\frac{T_1}{c_5 \times T_m}, 1 \right) \right] \quad 7.8$$

$$\ln \theta_{mod} = c_0 + c_1 q + (c_2 + c_3 q) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] + c_4 \times \ln \left[\max \left(\frac{T_1}{c_5 \times T_m}, 1 \right) \right] \quad 7.9$$

From examination of the literature a number of further potentially influential variables were identified. These were:

- The column to brace stiffness ratio, termed α by Karavasilis et al. (2007) and β_k by Elghazouli (2010)
- The relative storey stiffness of the upper to lower half of the frame, β_3 , as defined by Kumar et al. (2013)
- The mass participation ratio corresponding to the first mode, γ
- The brace non-dimensional slenderness, $\bar{\lambda}$

- The number of stories in the frame, NS
- Soil conditions, represented by the shear wave velocity at the recording site of the earthquake record, $V_{s,30}$

Kumar et al. (2013) examined the influence of a similar set of variables on MRF drift response and included γ and β_3 in their model for ϑ_{mod} . Here, the influence of other parameters was investigated by examining their impact on the standard error of the regression and the model residuals. From this process it was decided to include β_k , β_3 and γ in the model for ϑ_{mod} . A number of other small adaptations were made to Equations 7.8 and 7.9 before the final model was accepted. Firstly, for ϑ_{mod} , the c_1q term in Equation 7.9 was found to be uninfluential and was therefore removed. Secondly, the slight rise in δ_{mod} for higher T_1/T_m values found by Kumar et al. wasn't noticable in this study. Hence the last term in Equation 7.8, which captures this trend, was excluded from the analysis. For ϑ_{mod} , a rise in this region was noted, hence this term was retained in the regression equation. Here the coefficient c_5 was obtained by regression analysis, whereas Kumar et al. (2013) set this to the average ratio of first to second natural periods of the frames examined in that study. Ultimately, this meant the final regression models employed were:

$$\ln(\delta_{mod}) = c_0 + c_1q_{eff} + (c_2 + c_3q_{eff}) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] \quad 7.10$$

$$\ln(\vartheta_{mod}) = c_0 + (c_2 + c_3q_{eff}) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] + c_4 \times \ln \left[\max \left(c_5 \times \frac{T_1}{T_m}, 1 \right) \right] + c_6\gamma + c_7\beta_3 + c_8\beta_k \quad 7.11$$

Given the lower-bounded nature of δ_{mod} and ϑ_{mod} , log-transformed values were used for regression. As discussed by Kumar et al. (2013), it may be more appropriate to use some function of q to transform the data, but $EDPs$ are assumed to be lognormally distributed within most performance assessment frameworks, including FEMA P-58. Using log transformed data also has the advantage of reducing model complexity and making the results easily compatible with other models. The coefficients obtained from fitting the models described by Equations 7.10 and 7.11 to the data, and the associated 95% confidence intervals, are presented in Table 7-4 and Table 7-5 respectively.

Table 7-4 Coefficients and 95% confidence intervals for regression equation for Global Modification Factor, δ_{mod}

	Coefficient	95% Confidence Intervals	
c_0	-0.195	-0.225	-0.165
c_1	-0.018	-0.027	-0.008
c_2	0.357	0.285	0.429

c_3	-0.239	-0.262	-0.216
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Table 7-5 Coefficients and 95% confidence intervals for regression equation for Maximum Drift Modification Factor,

	δ_{mod}		
	Coefficient	95% Confidence Intervals	
c_0	2.868	2.349	3.386
c_2	0.538	0.417	0.659
c_3	-0.317	-0.344	-0.290
c_4	0.303	0.262	0.345
c_5	1.752	1.345	2.158
c_6	-4.649	-5.105	-4.192
c_7	0.881	0.712	1.049
c_8	-19.557	-22.410	-16.704

It should be noted that brace non-dimensional slenderness, $\bar{\lambda}$, is not included in the regression models, despite being widely recognised as a key parameter in the performance of CBFs. This is a result of the fact that the frames analysed here are all designed according to EC8. The combination of the overstrength and non-dimensional slenderness limitations prescribed tends to produce frame designs with similar slenderness profiles; all frames over two stories have similar values of minimum, maximum and median non-dimensional slenderness. As these values don't change greatly between frames, the influence of $\bar{\lambda}$ on drift response appears to be minimal. This is reflected in the residual plots presented in the following section. If bracing members were designed according to a different design philosophy that allowed a greater variation in slenderness, or, as in the study by Karavasilis et al. (2007), were designed explicitly for variation in slenderness, it is expected that $\bar{\lambda}$ would be more influential in the prediction of drift response.

7.4.1.1.1 Examination of Model Residuals

In Figure 7-18 the residuals for δ_{mod} , calculated as the observed data less the value predicted by the model, are plotted against a series of structural characteristics. The green lines are linear trend lines showing the average variation in residuals with the structural characteristics. No clear trends in the average residuals with any variable, indicating that any other parameters need to be included in the regression model, are evident. The plot of residuals against brace non-dimensional slenderness, which varies over the frame height, is presented using the median value of $\bar{\lambda}$ for each CBF; similar plots are obtained if the minimum or maximum value is used.

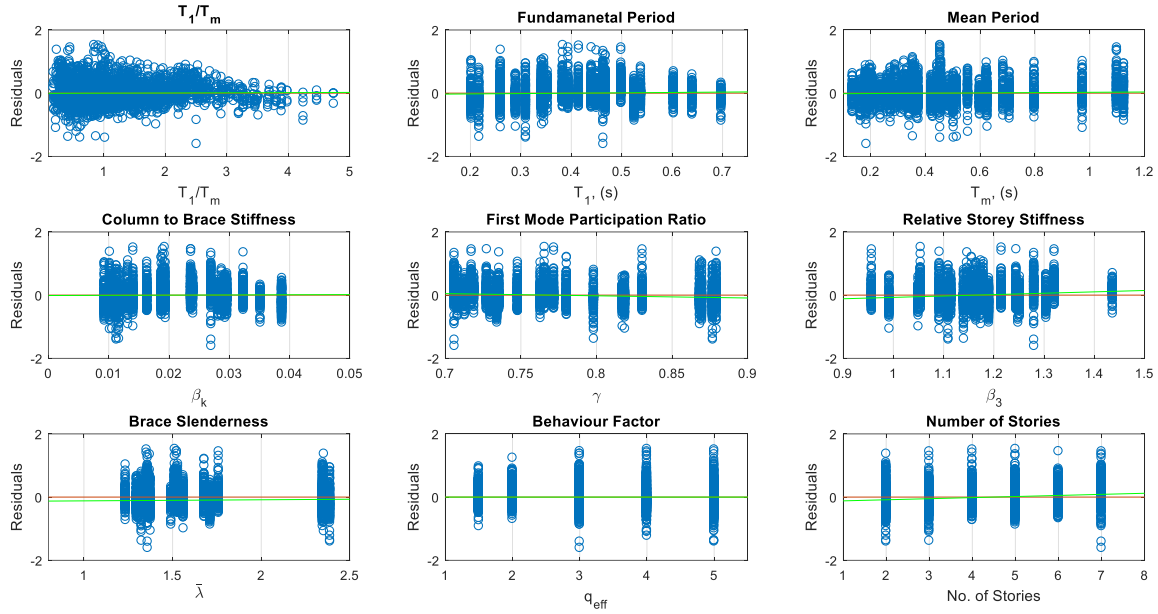


Figure 7-19 Variation in model residuals with a series of structural characteristics for δ_{mod}

Likewise, in Figures 7-20 and 7-21 the residuals for ϑ_{mod} , again calculated as the log transformed observed data less the value predicted by the model, are plotted against a series of structural characteristics. Figure 7-19 shows the residuals calculated using Equation 7.9, i.e. when the contributions of the last three terms in the fitted model, γ , β_k and β_3 , are not considered. When these terms are included in the model, the residual behaviour, as shown, in Figure 7-20 is much improved. The green lines show that the average value of the residuals is approximately zero across the range of each structural characteristics examined, i.e. the residuals are evenly distributed above and below the values predicted by the model. Unlike in Figure 7-19, no obvious trends requiring adaption of Equation 7.11 are evident in Figure 7-20.

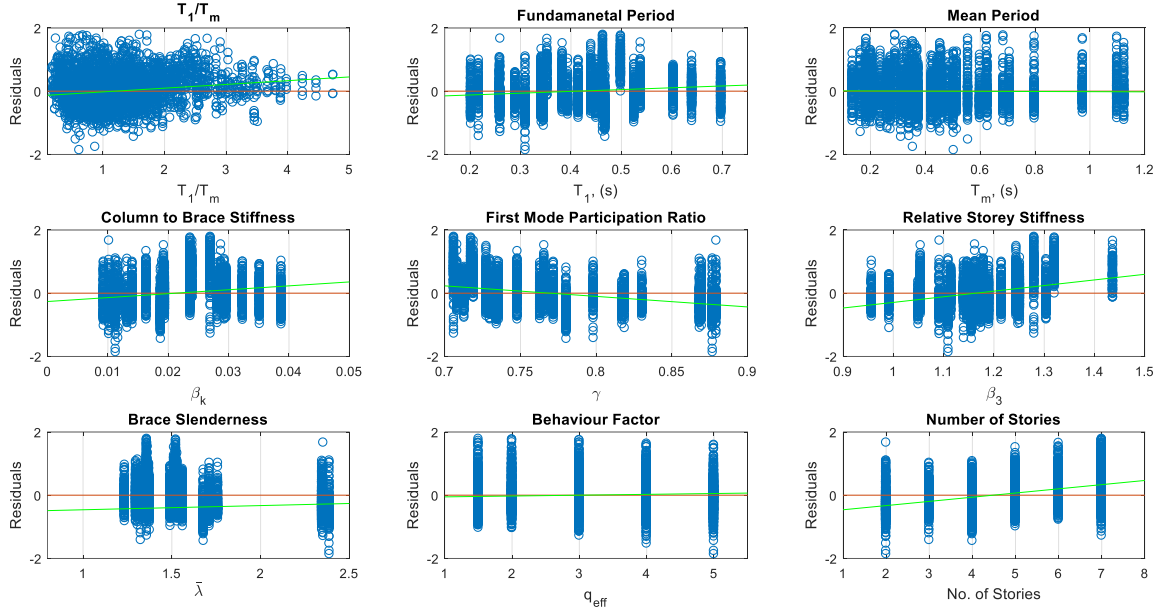


Figure 7-20 Residuals for δ_{mod} calculated using Equation 7.9, with γ , β_3 and β_k not included in the regression model

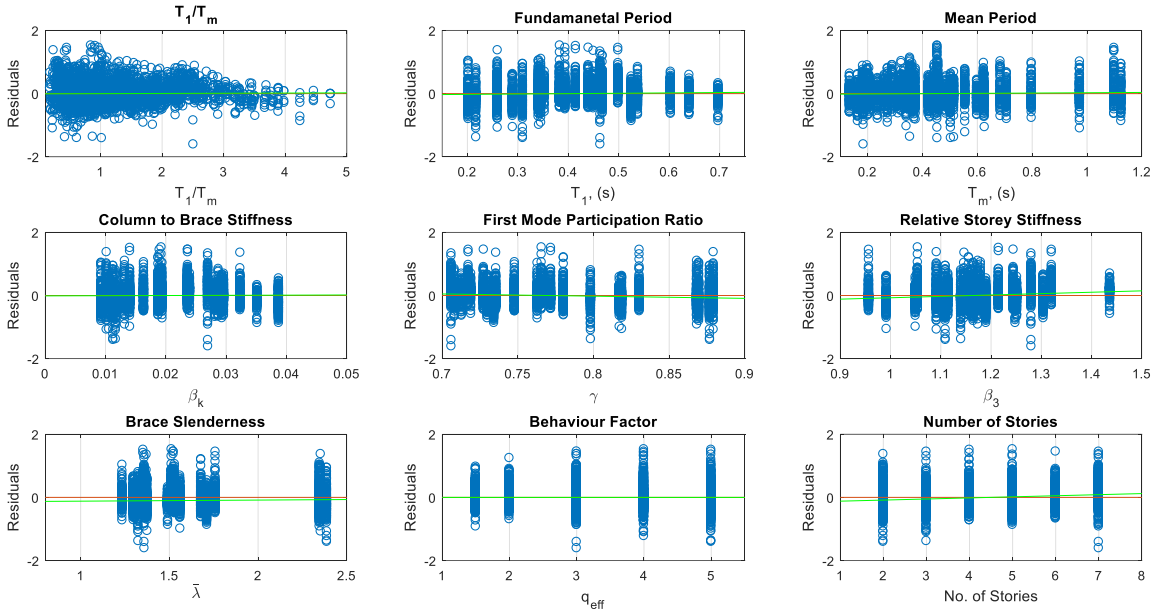


Figure 7-21 Residuals for δ_{mod} calculated using Equation 7.11; showing improved residual behaviour with the developed equation including γ , β_3 and β_k

7.4.1.1.2 Final Models

As already discussed, Eurocode 8 implicitly assumes that for $\delta_{mod} = 1$ for all values of the behaviour factor, q_{eff} . In contrast, the value of δ_{mod} predicted using the equal energies rule varies with q_{eff} and can be shown to be:

$$\delta_{mod} = \theta_{mod} = \frac{q_{eff}^2 + 1}{2q_{eff}} \quad 7.12$$

Figures 7-22 presents the regression models and associated 95% prediction bounds, calculated on the assumption of lognormal distribution of the data, for $q_{eff}=2, 3, 4$ and 5 respectively. The equal displacements and equal energies rules are also shown on these plots. The equal displacements rule, as employed by EC8, is observed to slightly overestimate δ_{mod} for $T_1/T_m > 1$. However, as T_1/T_m reduces below 1, the rule underestimates δ_{mod} , significantly so for the higher behaviour factors. The equal energies rule generally overestimates the observed δ_{mod} values by a greater amount however it too is exceeded at some low period ratio values.

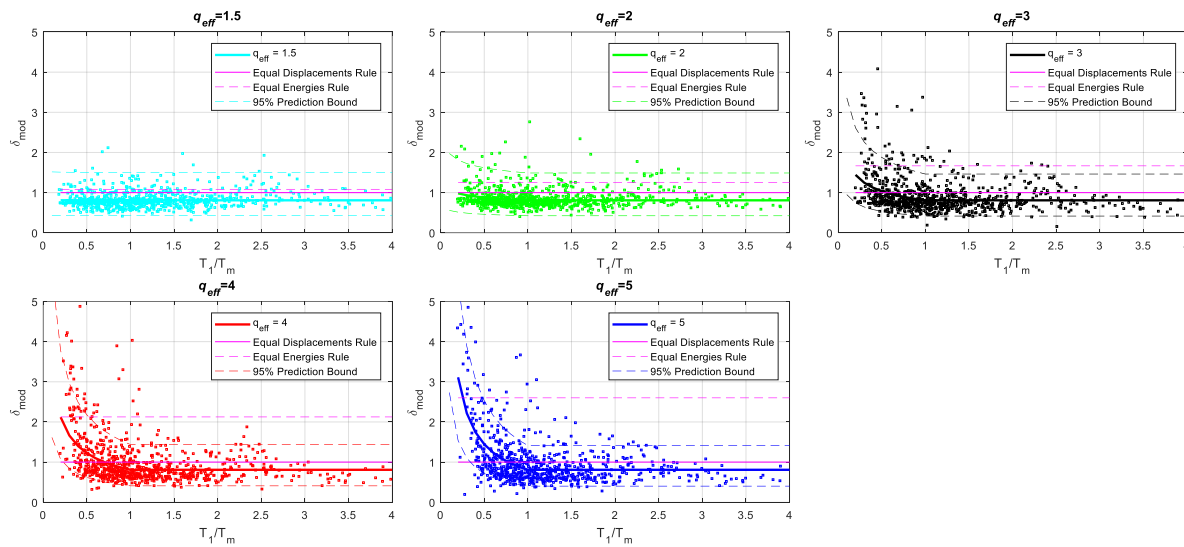


Figure 7-22 δ_{mod} : Regression Models and 95% prediction bounds for each q_{eff} value

Figure 7-23 presents similar plots for ϑ_{mod} . As mentioned previously, the ϑ_{mod} values are larger than the δ_{mod} values presented in Figure 7-22. The equal displacements rule, and consequently EC8, appears to underestimate ϑ_{mod} for almost all T_1/T_m values. For $q_{eff}=2$, the equal energies rule gives ϑ_{mod} values reasonably similar to those predicted by the regression model developed here. As the behaviour factor increases, the equal energies rule predicts values closer to the upper 95% prediction bound. The upper 95% prediction bounds exceed both the equal displacements and equal energies rule for all behaviour factors irrespective of the T_1/T_m value. This reflects the greater level of scatter in the higher T_1/T_m range associated with ϑ_{mod} compared to δ_{mod} , where the equal energies rule is generally close to or exceeds the upper prediction bound.

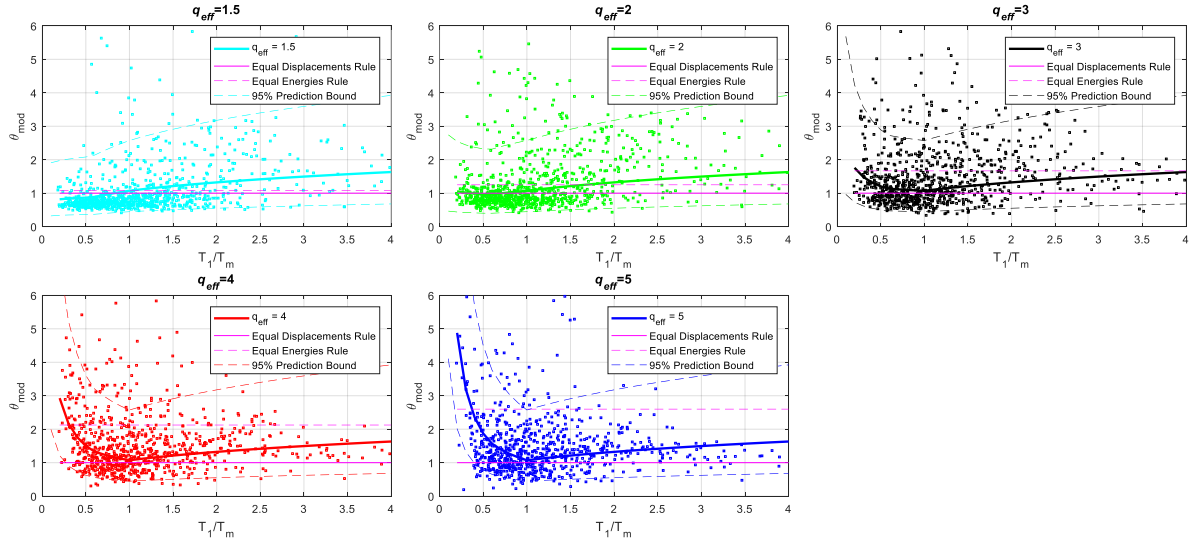


Figure 7-23 ϑ_{mod} : Regression Models and 95% prediction bounds for each q_{eff} value

7.4.1.1.3 Influence of Other Parameters on θ_{mod}

Figure 7-23 is developed using Equation 7.11, with the average values across the dataset for γ , β_k and β_3 . The following section examines the influence of these parameters on the predicted values for ϑ_{mod} .

7.4.1.1.3.1 First Mode Participation Ratio, γ

Predicted ϑ_{mod} values increase as the mass participation ratio corresponding to the first mode, γ , is reduced, as illustrated in Figure 7-24. Intuitively, this makes sense; as higher mode effects become more important, reflected by a less dominant first mode and hence a lower first mode participation factor, inter-storey drifts increase. This trend is shown in Figure 7-24.

As can be seen by examining Table 7-1, γ is closely related to frame height, with lower values corresponding to taller structures where higher modes are more influential. Values considered in this study range from 0.88 for one of two storey frames analysed to 0.70 for one of the seven storey frames. Therefore, the inclusion of γ in the model allows the influence of frame height to be captured by the model without explicitly being included as a variable.

As can be seen from Figure 7-25, γ can be quite influential on ϑ_{mod} , with predicted values almost doubling when the minimum (0.70), as opposed to the average (0.80), γ considered in the study is used in Equation 7.11.

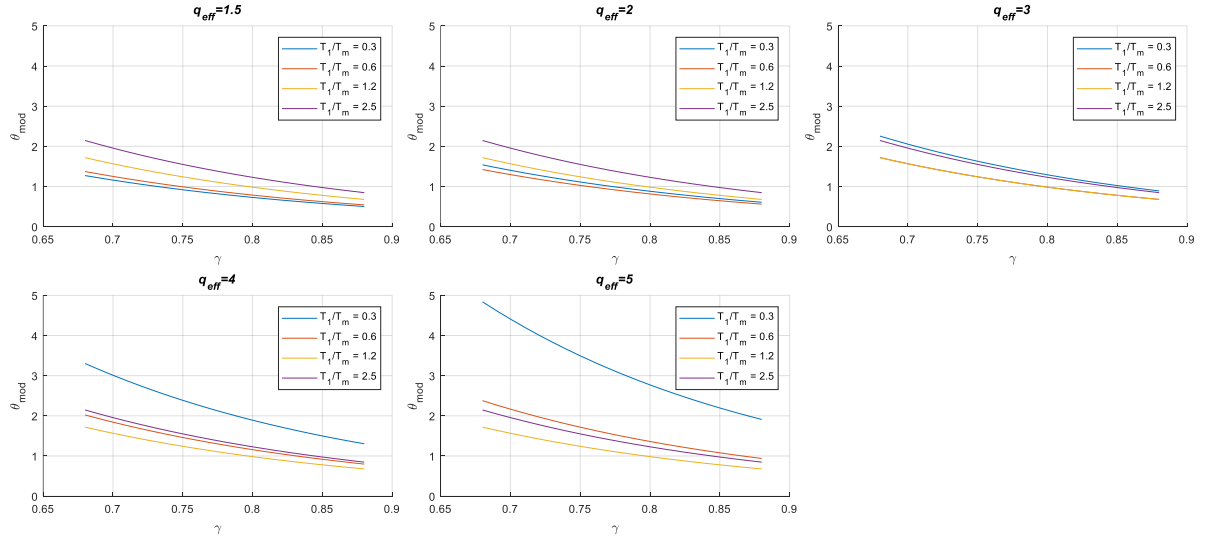


Figure 7-24 Influence of first mode participation ratio, γ , on ϑ_{mod}

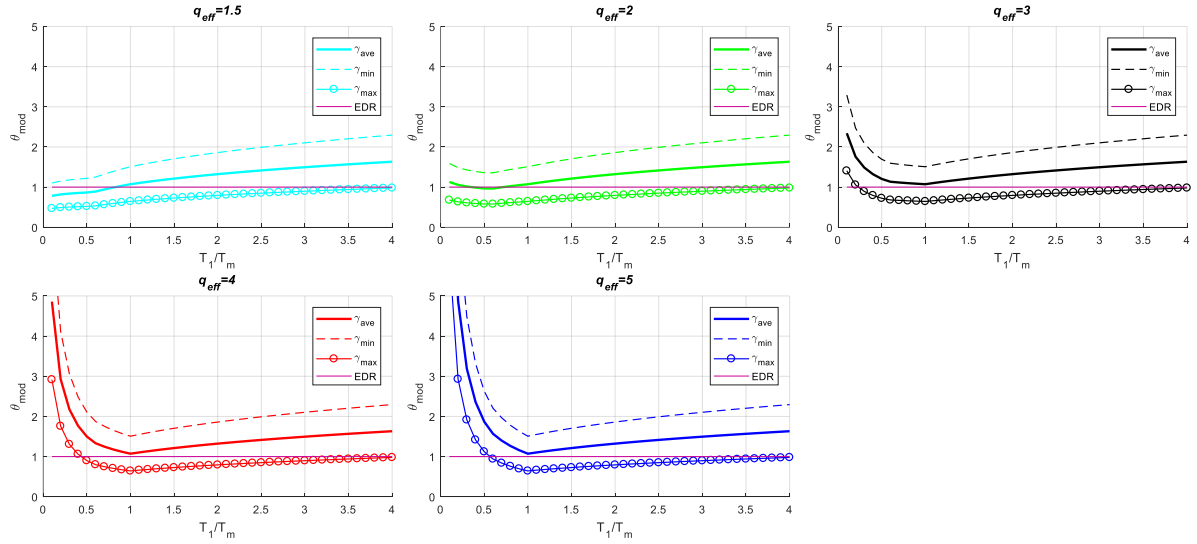


Figure 7-25 Variation in ϑ_{mod} with T_1/T_m for maximum, average and minimum γ values

7.4.1.1.3.2 Brace to Column Stiffness Ratio, β_k

Figure 7-26 shows that ϑ_{mod} decreases as β_k increases. Again, this trend is logical; as the lateral stiffness of the columns increases, the inter-storey drift decreases.

The impact of β_k on the predicated ϑ_{mod} value is less pronounced than γ , but moving from the average ($\beta_k = 0.021$) to the minimum value ($\beta_k = 0.009$) considered in the study can result in an increase of approximately 50%, as can be seen in Figure 7-27.

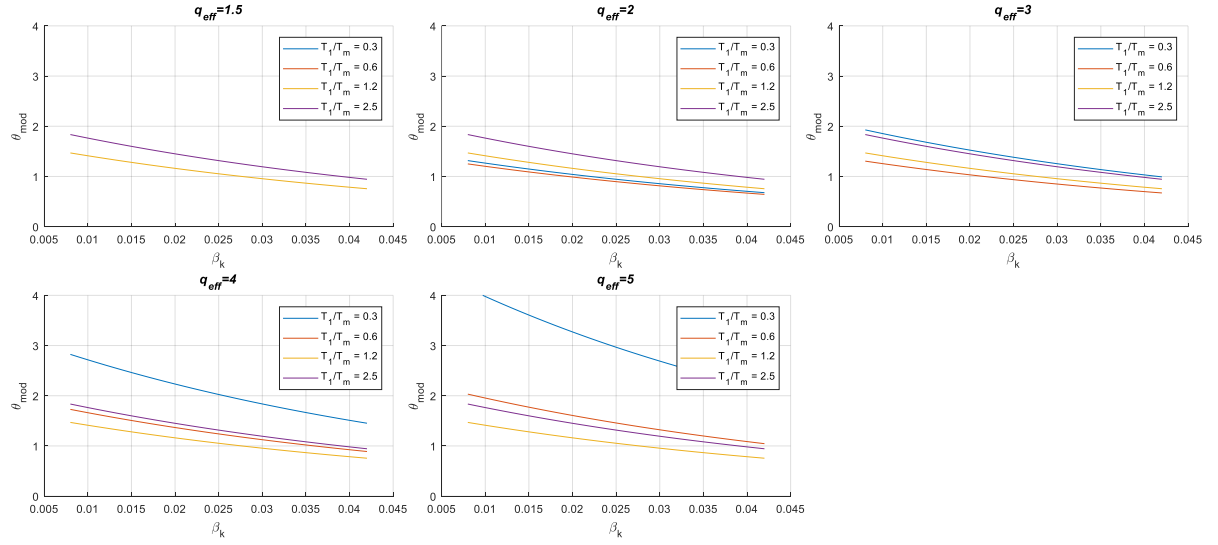


Figure 7-26 Influence of column to brace stiffness ratio, β_k , on ϑ_{mod}

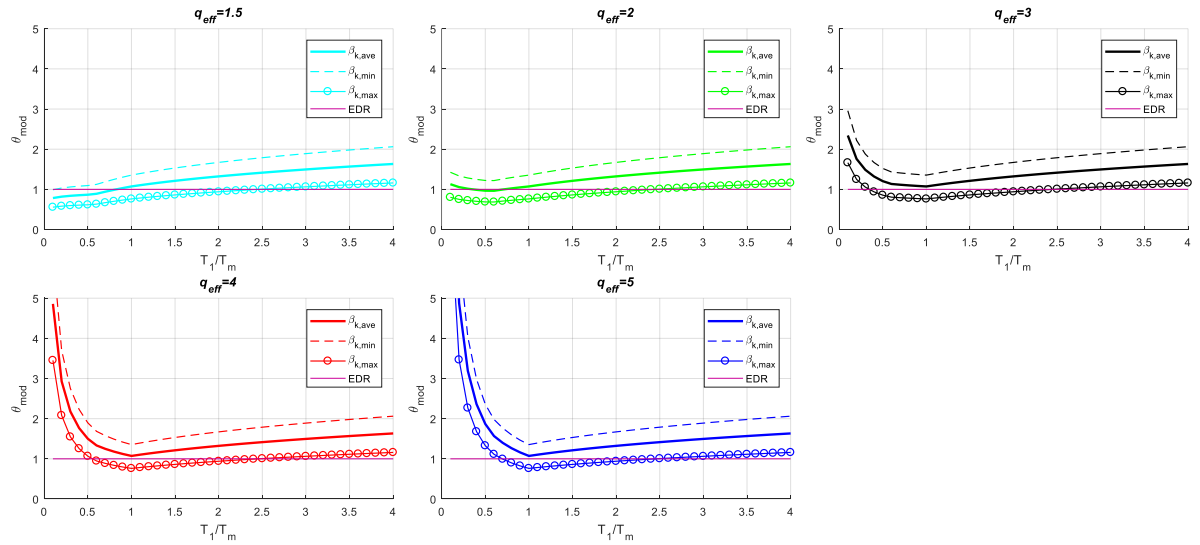


Figure 7-27 Variation in ϑ_{mod} with T_1/T_m for maximum, average and minimum β_k

7.4.1.1.3.3 Relative Storey Stiffness Ratio, β_3

As illustrated in Figure 7-28, ϑ_{mod} increases with β_3 . For the range of values considered in this study (0.96 to 1.44), an increased β_3 generally implies a particularly flexible storey exists in the upper half of the frame. Hence, it makes sense that inter-storey drift would increase with this.

The extent of the potential impact of β_3 is similar to β_k . Increasing from the average to the maximum values used in the study results in an increase of approximately 50% in the increased ϑ_{mod} value, as shown in Figure 7-29.

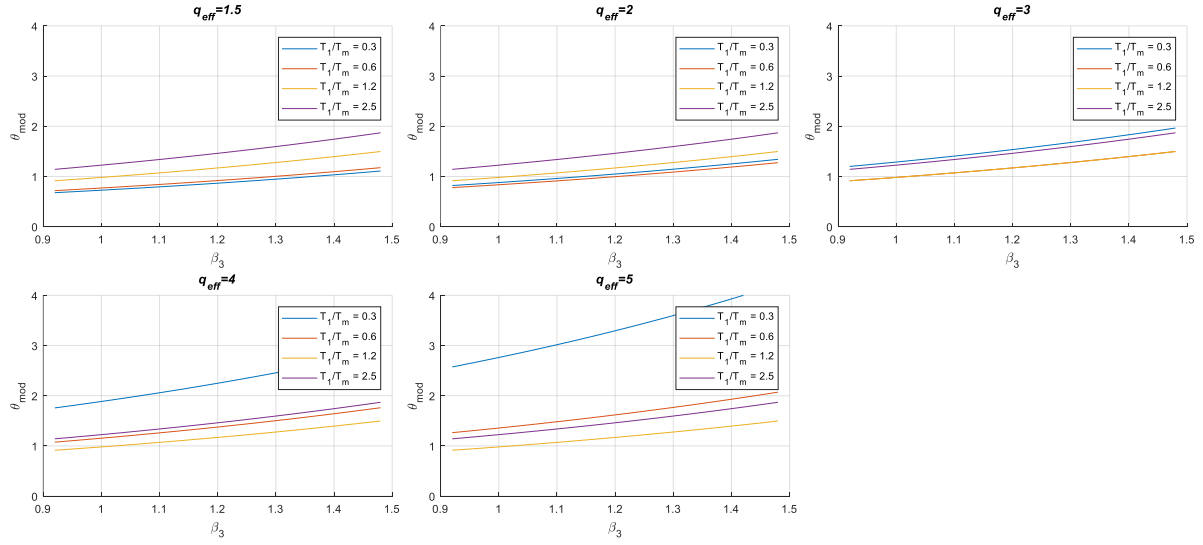


Figure 7-28 Influence of column to brace stiffness ratio, β_3 , on ϑ_{mod}

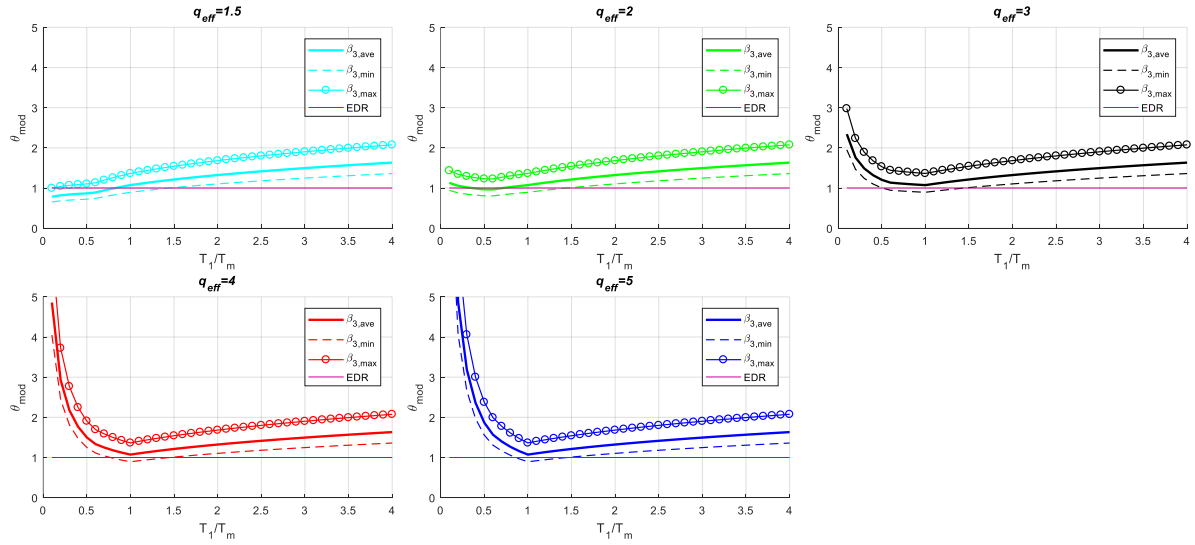


Figure 7-29 Variation in ϑ_{mod} with T_1/T_m for maximum, average and minimum β_3 values

7.4.1.2 Comparisons with Other Studies and Design Codes

Figure 7-30 compares the observed values of δ_{mod} with five previously developed prediction methods: the models for moment resisting frames proposed by Kumar et al. (2013) and Karavasilis et al. (2008), the model for CBFs proposed by Karavasilis et al. (2007), the equal displacements rule and the equal energies rule.

Comparing the results to the regression model proposed by Kumar et al. (2013) for MRFs, it can be seen that the general trend is quite similar; drift is reasonably constant for T_1/T_m values above 1 with a nonlinear rise as this ratio decreases below unity. The values of δ_{mod} observed in this study for higher T_1/T_m ratios are generally very similar to those found by Kumar et al. However, for lower T_1/T_m values, the δ_{mod} values predicted here for CBFs are greater than those from the

Kumar model for moment frames. This is especially evident for T_1/T_m ratios below approximately 0.5, where difference between to two models becomes particularly noticeable.

The study by Karavasilis et al. (2008) on moment resisting frames examined ground motions with lower mean periods than those in Kumar et al. (2013). Consequently, the rise in δ_{mod} in the low T_1/T_m range was not observed and the proposed model is independent of T_1/T_m . However, results are very similar to those found here for CBFs for T_1/T_m ratios above 1.

In contrast, the expression for displacement demand proposed by Karavasilis et al. (2007) in their study on X-braced frames does depend on the ground motion frequency content, represented by the corner period, T_c . It can be manipulated to give a value for δ_{mod} as a function of the behaviour factor, the number of stories, NS , brace slenderness, $\bar{\lambda}$, column to brace stiffness ratio, β_k , and, of most interest here, T_1/T_c . Using the relation:

$$T_m = 1.05T_c \quad 7.13$$

proposed by Kumar et al. (2011), the expression given by Karavasilis et al. (2007) for global drift becomes:

$$\delta_{mod} = \frac{1}{q} \left\{ 1 + \left[(q-1)(0.86^{-1})(NS^{-0.34})(\bar{\lambda}^{-0.7})(\beta_k^{0.1}) \left(\frac{1.05T_1}{T_m} \right)^{-0.24} \right] \right\}^{\frac{1}{0.62}} \quad 7.14$$

In Figures 9 this function is plotted by taking an average value for the number of stories, slenderness and column to brace stiffness used in this study. It can be seen that while the general shapes of the curves are similar, the values for δ_{mod} predicted by Equation 7.14 are less than those observed in this study. This difference is first of all possibly attributable to the series of averages taken here to develop the plots. However, particularly at lower period ratios the difference may also be partly due to the lower range of T_m values employed by Karavasilis et al. (2007); only two of the 30 ground motion records examined had mean periods greater than 0.63s, compared to 11 records in this study. Another potential reason for the difference is the forms of regression equations employed.

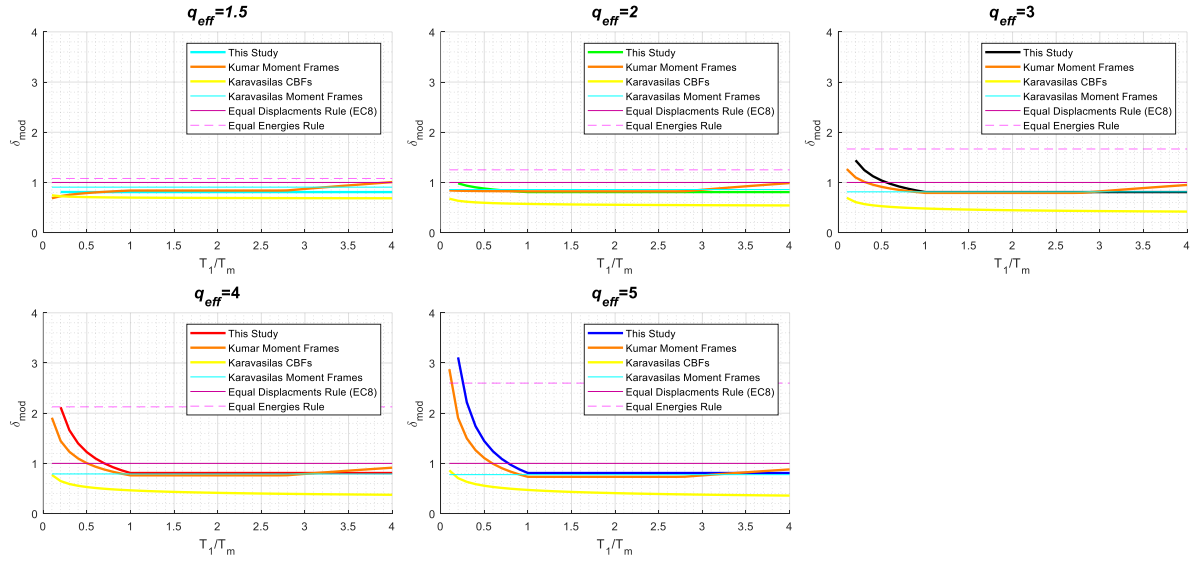


Figure 7-30 δ_{mod} : Comparison with other models

Figure 7-31 compares the values of ϑ_{mod} obtained for each behaviour factor with estimates from the Kumar et al. (2013) model for moment resisting frames and the equal displacements and equal energies rules. It can be seen that the values of ϑ_{mod} for CBFs from this study are always greater than those obtained by Kumar et al. (2013) for moment resisting frames. This can be explained by the tendency of CBFs to develop storey mechanisms where the ductility demand is concentrated at one level. This is in contrast with moment resisting frames, where the post-yield force distributions are more uniform.

Karavasilis et al. (2007) also developed a model for predicting inter-storey drift in CBFs, but this is as a function of global drift. While difficult to compare directly, this did show that greater concentrations of inter-storey drift at a single level were more common in taller frames. Although not evident from any of the plots presented here, this trend was also seen in this study, and is accounted for by the presence of the first mode mass participation factor ratio, γ , which, as discussed, is closely related to frame height, in Equation 7.11.

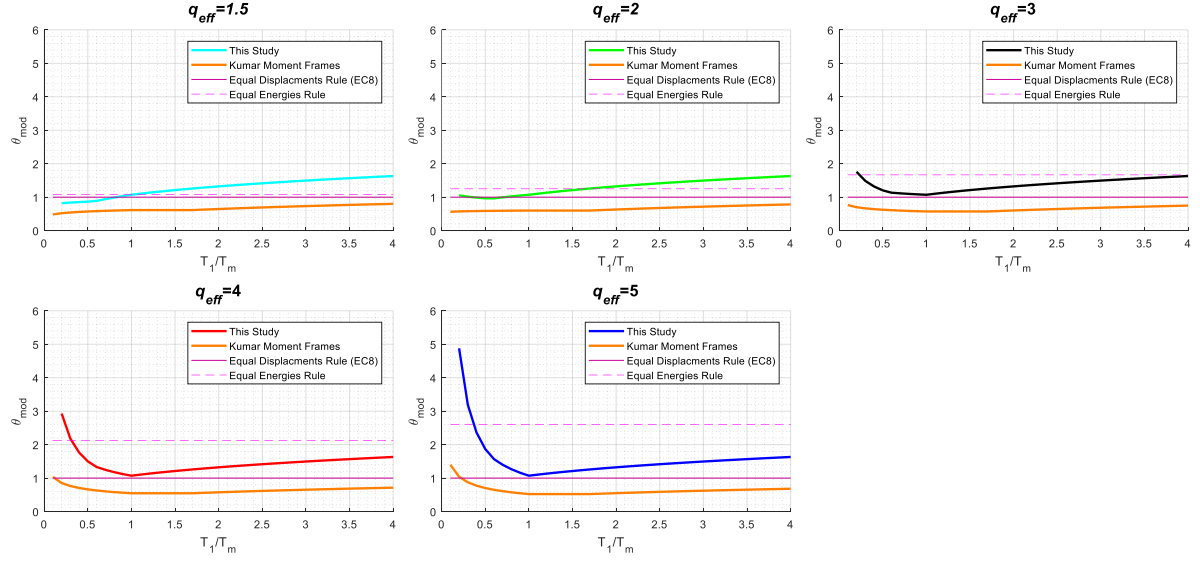


Figure 7-31 ϑ_{mod} : Comparison with other models

The US code provisions (ASCE/SEI, 2010) employ a seismic drift amplification factor, C_d to predict the design seismic drift response. The value of this factor depends on the type of braced frame. For SCBFs a reduction force factor, R , of 6 is proposed, with a corresponding C_d value of 5. δ_{mod} and ϑ_{mod} are given by the ratio of the two:

$$\delta_{mod} = \theta_{mod} = \frac{C_d}{R} = \frac{5}{6} \quad 7.15$$

As shown in Figure 7-32, like the equal displacements rule, the US provisions generally appear to underestimate δ_{mod} for the lower T_1/T_m ratios. However, the US SCBF provision achieves better agreement with the results observed in this study for higher T_1/T_m values.

Both the Eurocode and ASCE provisions appear to underestimate ϑ_{mod} for CBFs designed to EC8. This indicates that employing the same rule to estimate peak global and inter-storey drift for CBFs subjected to high intensity ground motions may not be adequate due to the tendency of CBFs to develop storey mechanisms.

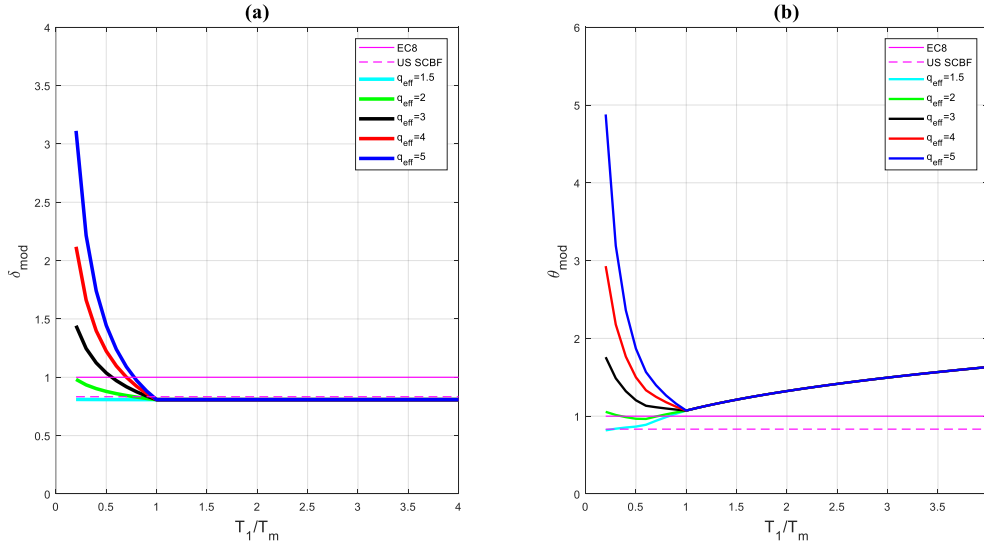


Figure 7-32 (a) Global drift modification factor, δ_{mod} and (b) Maximum drift modification factor, θ_{mod} : regression models for $q = 2, 3, 4, 5$, Comparison with EC8 and AISC Provisions

7.4.2 Obtaining Peak Inter-Storey Drift for Performance Assessment

7.4.2.1 Intensity level Drift Prediction

For performance assessment, Equations 7.10 and 7.11 can be used to predict drift at a ground motion intensity level of interest, where the intensity level is represented by an $S_a(T_1)$ value. Using Equation 7.16, an $S_a(T_1)$ value can be used to calculate q_{eff} , which can subsequently be employed in Equation 7.10 or 7.11 to obtain the global or maximum drift modification factor for the intensity level.

$$q_{eff} = \frac{S_a(T_1) \times \gamma \times m}{V_{BS,yield}} \quad \#7.16$$

The global or peak inter-storey drift can then be calculated by rearranging Equations 7.10 or 7.11 as follows:

$$\Delta_{max} = \delta_{mod} \times q_{eff} \times \Delta_1 \quad 7.17$$

$$\theta_{max} = \theta_{mod} \times q_{eff} \times \theta_1 \quad 7.18$$

These drifts can then be used in performance assessment.

7.4.2.2 Storey Level Drift Prediction

The expressions developed for Δ_{max} and θ_{max} can be used to predict either global, i.e. the roof, drift or inter-story drift unspecified level in the building. However, the FEMA P-58 is a storey level performance assessment methodology. Therefore, if either of these methods is to be used

within FEMA P-58, some method of obtaining inter-storey drifts at all levels needs to be developed.

A simple method for doing this is to use the first mode inter-storey drift profile obtained from an eigenvalue analysis, ϕ^1 , to distribute drifts over the building height.

The inter-storey drift at each level for the first mode shape is calculated as the difference in drift between each floor level for the mode shape, ϕ^1_{Δ} . This can then be normalized by the maximum inter-storey drift, giving:

$$\hat{\phi}^1_{\Delta,i} = \frac{\phi^1_i - \phi^1_{i-1}}{\max(\phi^1_i - \phi^1_{i-1})_{i=1: \text{no.stories}}} \quad 7.19$$

This concept is illustrated in Figure 7-33 for an example 5 storey structure.

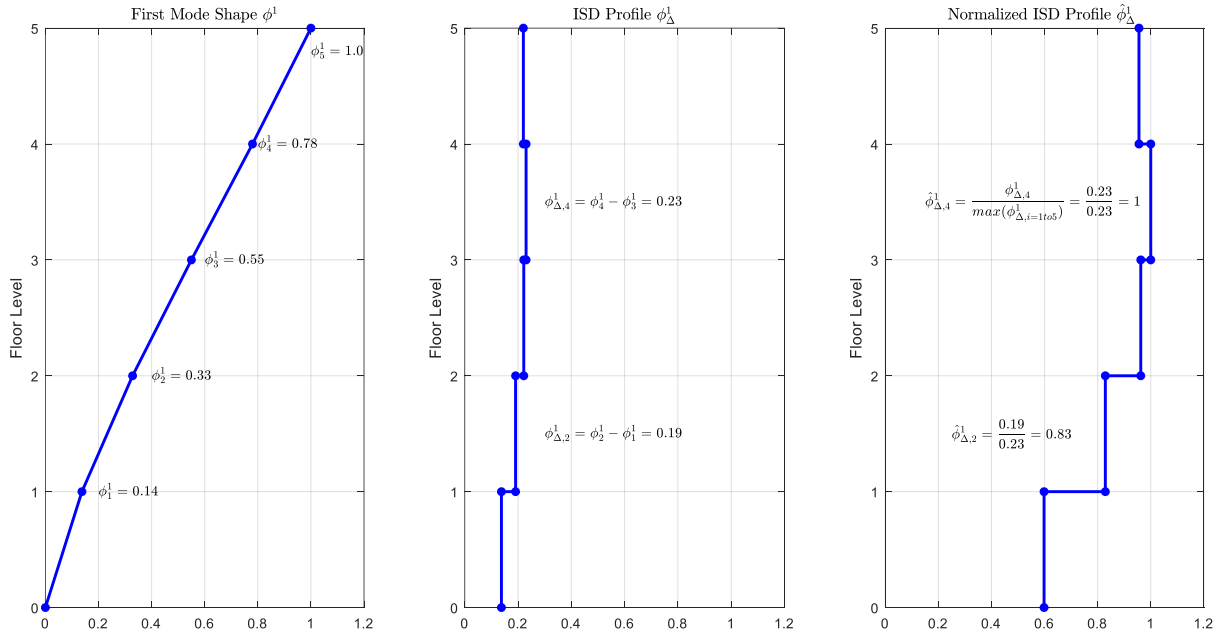


Figure 7-33 Calculation of the normalized ISD Profile from the first mode shape, ϕ^1

This inter-storey drift profile can be used to distribute either the global drift or the peak inter-storey drift value obtained from Equation 7.17 or 7.18 over the building height. Thus, peak inter-storey drift at i^{th} storey, θ_i , can be calculated as the product of the maximum global drift and first mode inter-storey drift at the level of interest:

$$\theta_i = \Delta_{max} \phi^1_{\Delta,i} \quad 7.20$$

Alternatively, ϑ_i can be calculated as the product of the maximum inter-storey drift and normalized first mode inter-storey drift at the level of interest:

$$\theta_i = \theta_{max} \hat{\phi}_{\Delta,i}^1 \quad 7.21$$

This is illustrated graphically in Figure 7-34.

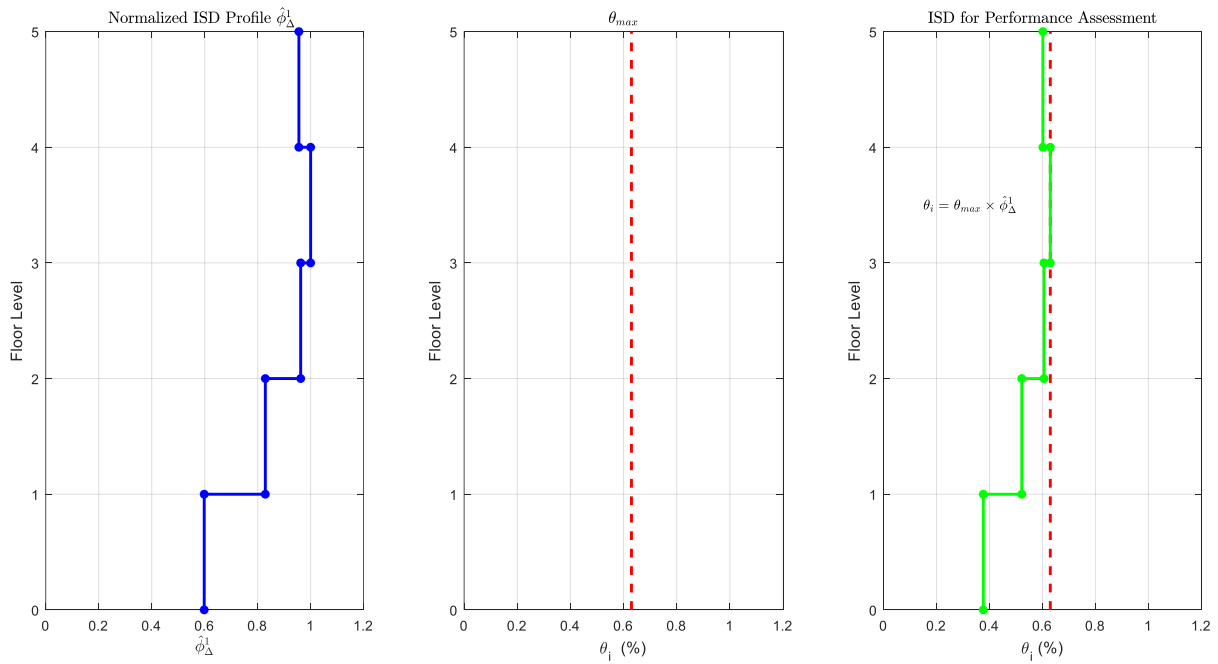


Figure 7-34 Illustration of the development of inter-storey drift profile for performance assessment by Equation 7.21

Using the peak inter-storey drift, as opposed to the global drift, to develop the inter-storey drift profile over the frame height is more conservative as it accounts for the potential development of soft storeys where drift is concentrated at a single level. Both methods are considered in the calculation of performance metrics later in the Chapter.

While relatively straight forward, using the fundamental mode shape to distribute inter-storey drift over the frame height has two main deficiencies. Firstly, it doesn't account for the influence of higher modes and secondly it doesn't account for the change in mode shape after the structure yields.

7.4.2.3 Alternative Model

As an alternative to using the mode shape to convert ϑ_{mod} to a storey drift at each floor, it is possible to obtain a regression expression for inter-storey drift as a function of the relative height of a floor level:

$$Relative\ Height = \frac{h_i}{H} \quad 7.22$$

Where h_i is the height of an individual storey level and H is the overall building height. This approach is adopted in the simplified analysis approach in FEMA P-58. However, attempts to develop such a method in this study showed that it was difficult for regression equations to capture soft-storey behaviour, which becomes important for CBFs at higher intensity levels. Hence, it was found that such regression equations were conservative and tended to underestimate peak drift. This is illustrated in Figure 7-35, where the median inter-storey drift profile for each frame height is compared to a regression model.

The model fitted in Figure is described by the Equation 7.23, which predicts inter-storey drift at level i as a function of q_{eff} , fundamental period T_1 , the total number of stories NS , and the relative height. The coefficients obtained from regression analysis, and the 95% confidence intervals, are presented in Table 7-7.

$$\theta_i = c_0 + c_1 q_{eff} + c_2 T_1 + c_3 NS + c_4 \frac{h_i}{H} + c_5 \left(\frac{h_i}{H} \right)^2 \quad 7.23$$

Table 7-6 Coefficients and confidence intervals for the alternative model describing the variation of inter-storey drift over the frame height

	Coefficient	95% Confidence Intervals	
c_0	-7.965	-8.008	-7.922
c_1	0.355	0.350	0.361
c_2	-1.933	-2.010	-1.856
c_3	0.234	0.228	0.241
c_4	2.267	2.151	2.383
c_5	-0.514	-0.607	-0.420

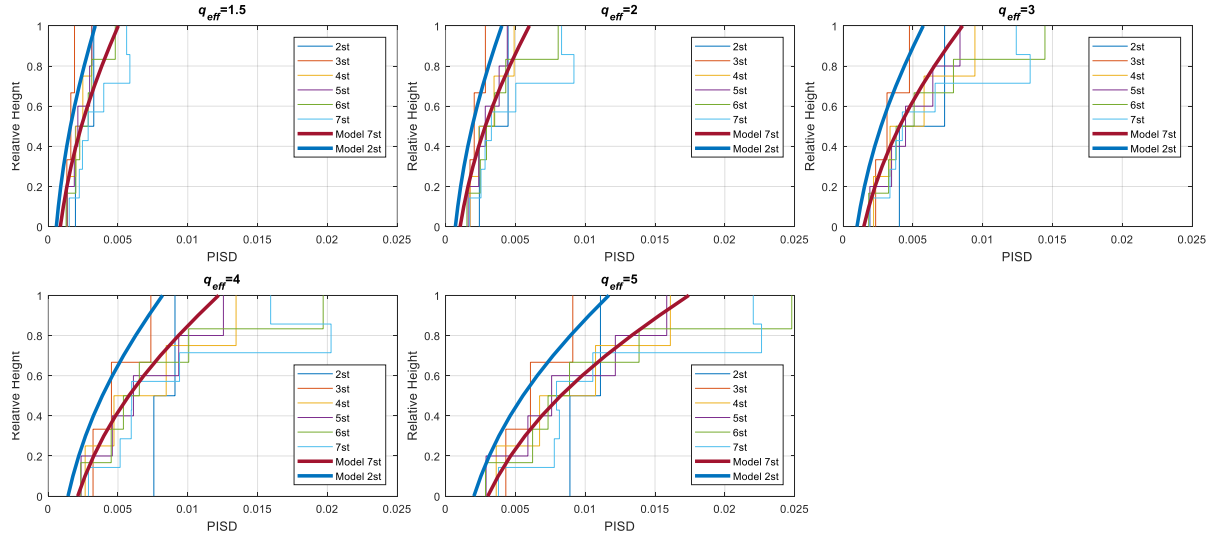


Figure 7-35 Illustration of how the model for PISD as a function of relative storey height struggles to capture soft-story type behaviour, particularly for higher q_{eff} values for taller structures

For the rapid performance assessments carried out later in this Chapter, it was decided to adopt the more conservative approach based using regression to predict peak response, and then distributing this over the frame height using the mode shape, i.e. using Equations 7.20 or 7.21. However, it is also possible to use Equation 7.23 to predict peak inter-storey drift.

7.4.3 Variation in δ_{mod} and θ_{mod}

As mentioned, for the FEMA P-58 simplified performance assessment procedure it is necessary to develop *EDP* distributions, meaning a value is required for a variation term, β_{SD} for drift response, in addition to the median value. This variation term is to account for uncertainty due to record-to-record variability, $\beta_{a\Delta}$, and modelling uncertainty, β_m . These two sources of uncertainty are combined using a SRSS approach (FEMA, 2012a):

$$\beta_{SD} = \sqrt{\beta_{a\Delta}^2 + \beta_m^2} \quad 7.24$$

Record-to record variation with respect to the model developed for inter-storey drift can be assessed by examining the model residuals. As mentioned, the models for δ_{mod} and ϑ_{mod} were developed by assuming a lognormal data distribution. The standard deviation of the model residuals for δ_{mod} is equal to 0.32. This is illustrated by red dashed lines in Figure 7-36. The impact of various parameters on the 16th and 84th percentile of the residuals, representing ± 1 standard deviation, was then examined. This was done by performing quantile regression to obtain first order polynomials representing the change in the 16th and 84th percentiles with each parameter. These polynomials are shown in blue dashed lines in Figure 7-36.

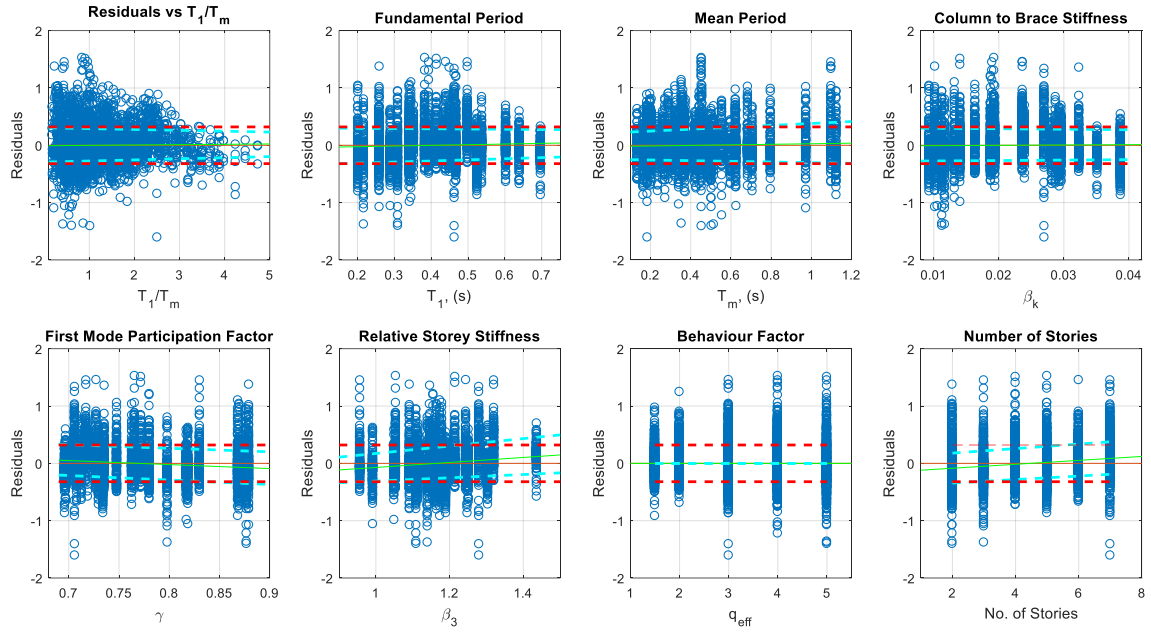


Figure 7-36 Change in standard deviation of model residuals for δ_{mod} with various parameters

There is no strong pattern in variation with any of the parameters. Therefore, it was decided to assume a lognormal standard deviation of 0.35 for δ_{mod} in all cases.

A similar procedure was adopted to analyse ϑ_{mod} , where the standard deviation of the residuals was calculated as 0.42, reflecting the greater level of scatter compared to δ_{mod} .

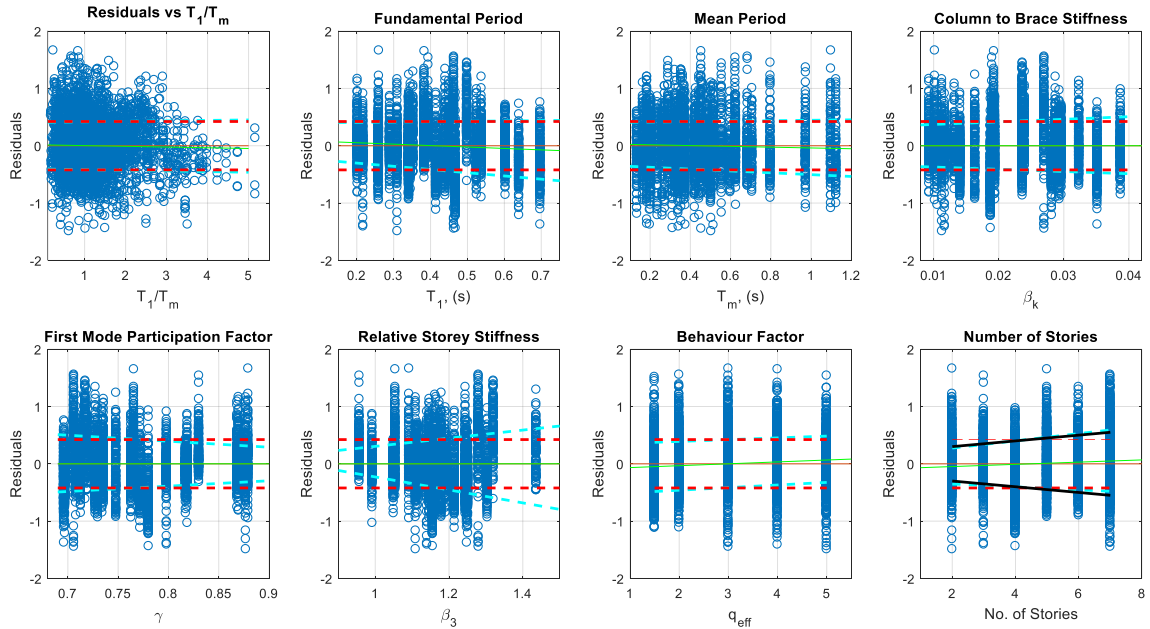


Figure 7-37 Change in standard deviation of model residuals for ϑ_{mod} with various parameters

From looking at the blue lines showing the quantile regressions in Figure 7-37, it can be seen that the average level of variation stays relatively constant with q and T_1/T_m . However, there is

evidence of increasing variation with increasing fundamental period, relative storey stiffness, column-to brace stiffness and with reducing first mode participation. All of these parameters are positively correlated with increasing number of stories. Hence it is reasonable to use the number of stories as a parameter to approximate variation in the data. A conservative estimate of the lognormal standard deviation can be made using the following expression:

$$\sigma_{\ln, \theta_{mod}} = 0.18 + 0.06NS \quad 7.25$$

Figure 7-38 shows how this expression compares with the fitted values for change in lognormal standard deviation with the number of stories.

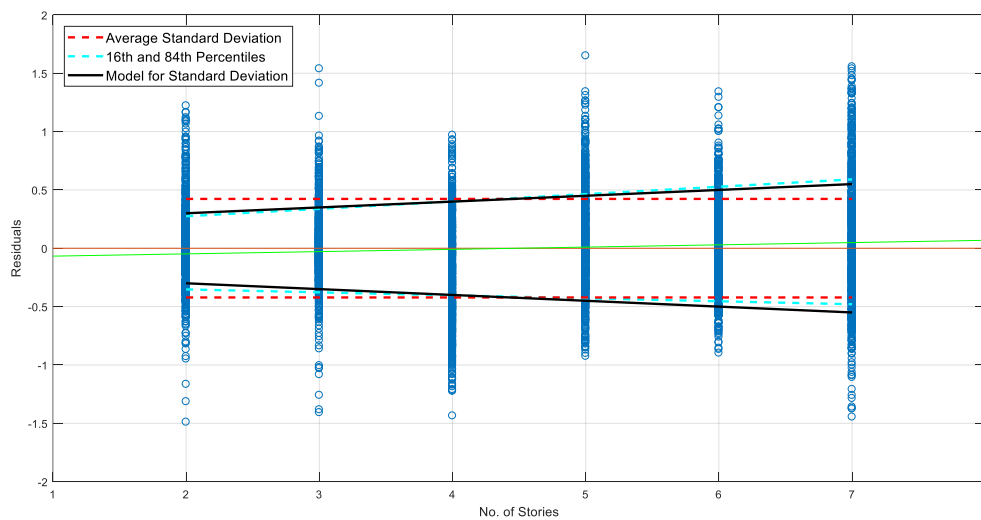


Figure 7-38 Variation in model residuals and fitted model for lognormal standard deviation with the number of stories

The standard deviations of δ_{mod} and ϑ_{mod} are estimated by assuming lognormal distributions. For performance assessment these parameters are converted to ϑ_{max} , i.e. inter-storey drift, using Equations 7.17 or 7.18. This involves multiplying by a constant ($\Delta_1 \times q_{eff}$ or $\vartheta_1 \times q_{eff}$). A lognormal distribution, X , can be described by a logarithmic mean μ , lognormal standard deviation, σ , and a standard normal variable, Z . As per Equation 7.27, it can be demonstrated that such a distribution multiplied by a positive constant, c , is also a lognormal distribution with a new median, $\bar{\mu}$, but with the same lognormal standard deviation as the original distribution:

$$X = e^{\mu + \sigma Z} \quad 7.26$$

$$cX = ce^{\mu + \sigma Z} = e^{\ln(c)} e^{\mu + \sigma Z} = e^{\mu + \ln(c) + \sigma Z} = e^{\bar{\mu} + \sigma Z} \quad 7.27$$

Therefore, this means that the lognormal standard deviations estimated for δ_{mod} and ϑ_{mod} are also the lognormal standard deviation for inter-storey drift and can be used in the FEMA P-58 performance assessment framework.

For application in performance assessment, a value of $\beta_m = 0.5$ was assumed for modelling uncertainty, in accordance with the FEMA P-58 recommendations. Therefore, the overall value for distribution used in performance assessment for peak inter-storey drift, β_{SD} , is given by:

$$\beta_{a\Delta} = 0.18 + 0.06NS \quad 7.28$$

$$\beta_m = 0.5 \quad 7.29$$

Hence if inter-storey drift is estimated based in roof drift using δ_{mod} :

$$\beta_{SD} = \sqrt{\beta_{a\Delta}^2 + \beta_m^2} = \sqrt{0.35^2 + 0.5^2} = 0.61 \quad 7.30$$

Alternatively, if inter-storey drift at each level is estimated based in peak inter-storey drift using ϑ_{mod} :

$$\beta_{SD} = \sqrt{\beta_{a\Delta}^2 + \beta_m^2} = \sqrt{(0.18 + 0.06NS)^2 + 0.5^2} \quad 7.31$$

In the broader context of applying probabilistic performance assessment as part of a performance-based design framework, the ability to explicitly consider uncertainty is an advantage over conventional design methods. Within present deterministic equivalent lateral load-based methods it is difficult to account for uncertainty outside of including a certain level of conservatism in the design process. Within the current EC8 design process for CBFs this is achieved for the structural modelling by ignoring the lateral stiffness provided by the compression diagonal, which means completed buildings can have significant levels of overstrength (Elghazouli, 2010). Similarly, for reasons outlined previously, the Eurocode 8 design spectrum is often conservative, meaning there is a level of conservatism in the representation of the seismic action. However, in performance-based design uncertainty can be included in the performance assessment by using factors like β_{SD} , which allows the levels of ground motion and structural modelling uncertainty to be quantified explicitly and varied depending on the perceived quality of the seismic hazard assessment and structural response prediction. However, further research is needed, particularly to quantify exactly the accuracy of typical structural response predictions and provide guidance on appropriate values of β_m .

7.5 Peak Floor Acceleration

7.5.1 Floor Acceleration Magnification

Figure 7-39 shows the variation in floor acceleration magnification with relative height for the five q_{eff} values in the dataset, while Figure 7-40 shows the median values for each frame height examined. As discussed, the saturation of PFA means that greater floor acceleration magnifications are observed for the lower q_{eff} values considered. Unlike inter-storey drift, peak floor accelerations for the frame heights examined do not tend to become concentrated at a single floor level, so it is reasonable to develop a regression equation based on the relative height. However, this may change for taller frames where studies (Reinoso and Miranda, 2005) of other frame types have shown a tendency for extreme values in the upper stories.

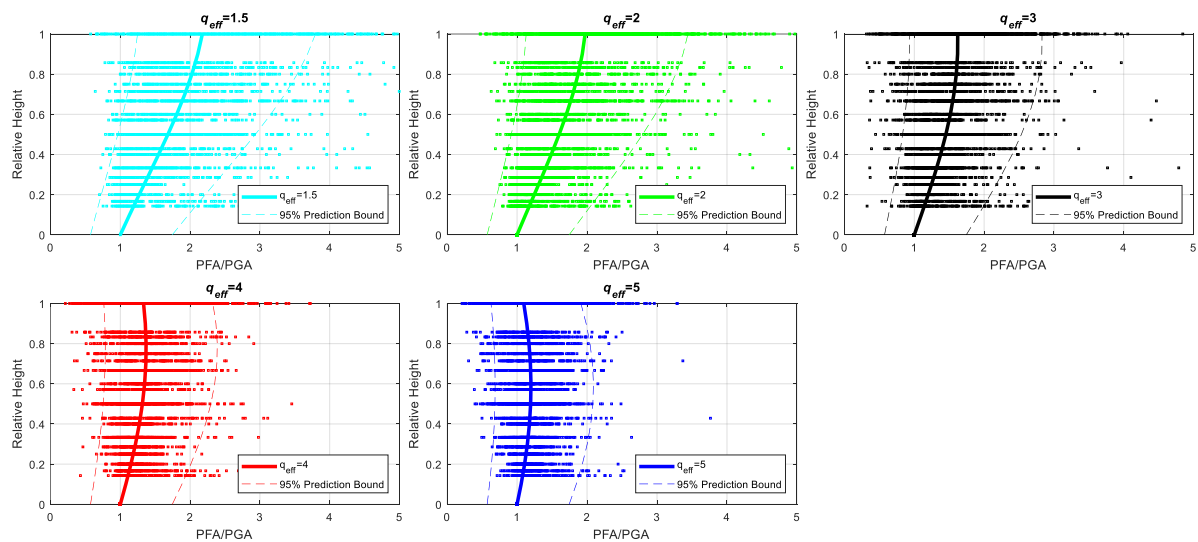


Figure 7-39 Data, fitted model and 95% prediction bounds for each q_{eff} examined

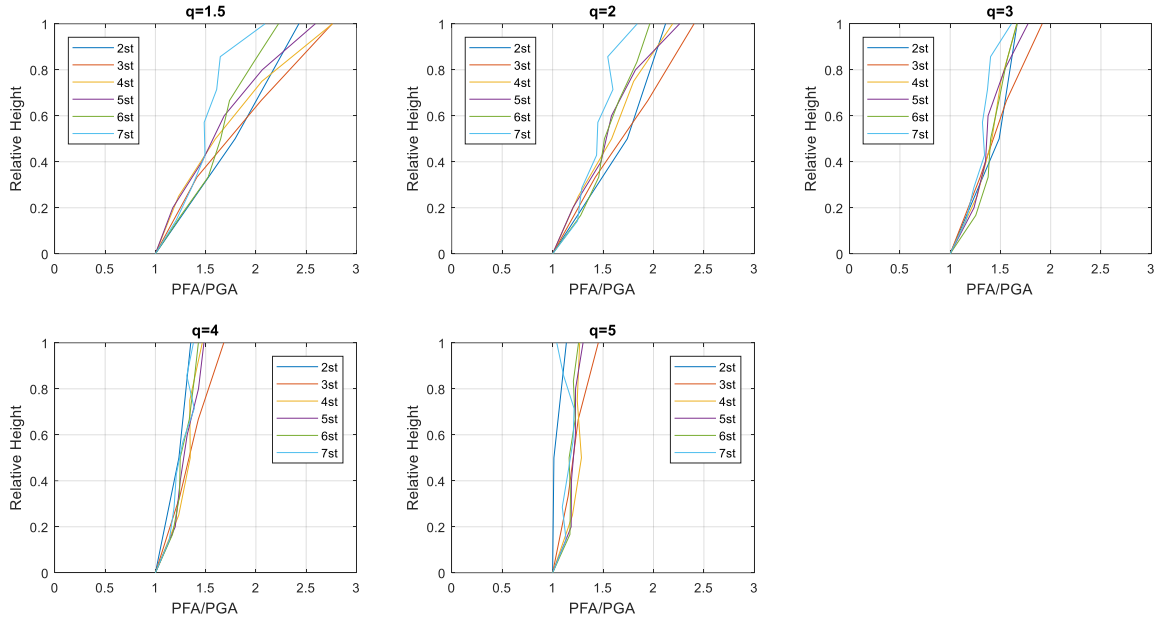


Figure 7-40 Median floor acceleration magnification for each frame height examined

A regression model was fitted to this data for use in rapid performance assessment. In order to develop a regression model the data was assumed to be lognormally distributed, as is generally assumed in performance assessment, including within FEMA P-58. This has been shown to be a valid assumption for floor acceleration magnification by Taghavi and Miranda (2012).

The equation proposed in FEMA P-58 to capture floor acceleration magnification is:

$$\ln \left(\frac{PFA}{PGA} \right) = c_0 + c_1 S + c_2 \frac{h_i}{H} + c_3 T_1 \quad 7.32$$

here S is a term that is equivalent to the q_{eff} term used here. Different coefficients are proposed for various frame types and heights. However, this equation does not capture the fundamental property that at a relative height of 0, i.e. at the ground level, the PFA must equal the PGA. In order to address this, the regression equation was initially adapted to:

$$\ln \left(\frac{PFA}{PGA} \right) = \frac{h_i}{H} \left(c_0 + c_1 q_{eff} + c_2 \frac{h_i}{H} + c_3 T_1 \right) \quad 7.33$$

Regression analysis was carried out using this equation. However, from examination of the mean squared error and model residuals it was decided to replace the fundamental period, T_1 , with ratio of fundamental to mean period, T_1/T_m , and also to include the number of stories, NS , as an extra predictor variable. Thus, the final regression equation was of the form:

$$\ln \left(\frac{PFA}{PGA} \right) = \frac{h_i}{H} \left(c_0 + c_1 q_{eff} + c_2 \frac{h_i}{H} + c_3 \frac{T_1}{T_m} + c_4 NS \right) \quad 7.34$$

he coefficients, and the associated 95% confidence intervals, describing the fitted equation are

presented in Table 7-8. The fitted model and the 95% prediction bounds are plotted in Figure 7-39 for each q_{eff} , using the average values of T_1/T_m and NS for the dataset.

Table 7-7 Coefficients and 95% confidence intervals for regression equation for Floor Acceleration Magnification, PFA/PGA

	Coefficient	95% Confidence Intervals	
C_0	1.846	1.811	1.881
C_1	-0.196	-0.200	-0.191
C_2	-0.488	-0.516	-0.459
C_3	-0.379	-0.387	-0.370
C_4	0.041	0.037	0.045

7.5.1.1 Influence of T_1/T_m and Number of Stories on Peak Acceleration

Figure 7-41 shows the impact of T_1/T_m on the peak acceleration response. It is clear that increased floor acceleration magnifications are experienced for T_1/T_m values slightly below unity. As with drift response discussed previously, this can be attributed to resonance between the ground motion and the elongated post-yield period of the structure. For T_1/T_m ratios greater than approximately 2, the floor acceleration magnification is approximately equal to unity, irrespective of the degree of nonlinearity.

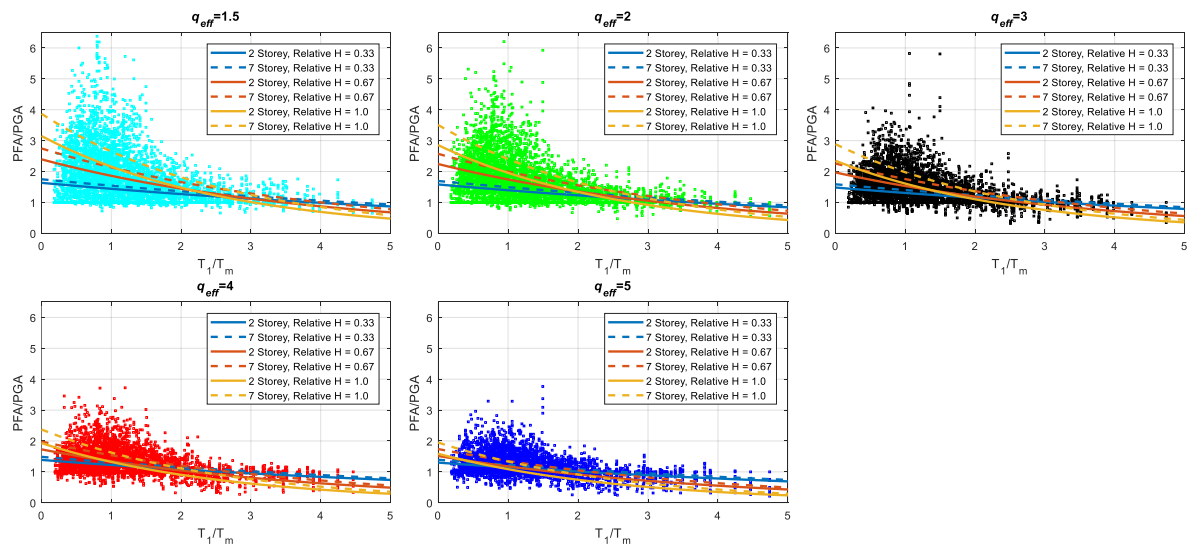


Figure 7-41 Variation in floor acceleration magnification with T_1/T_m

7.5.1.1.1 Variation with Number of Stories

Figure 7-43 shows the floor acceleration magnification predicted by the model for various T_1/T_m values for a 2 storey structure, while Figure 7-44 shows the same thing for a seven storey structure. Comparing Figures 7-42 and 7-43 allows the impact of the number of storeys and the

mean period on predicted acceleration response to be assessed. It can be seen that the model predicts greater acceleration magnification for the taller buildings. However this difference becomes less pronounced as the degree of nonlinearity increases.

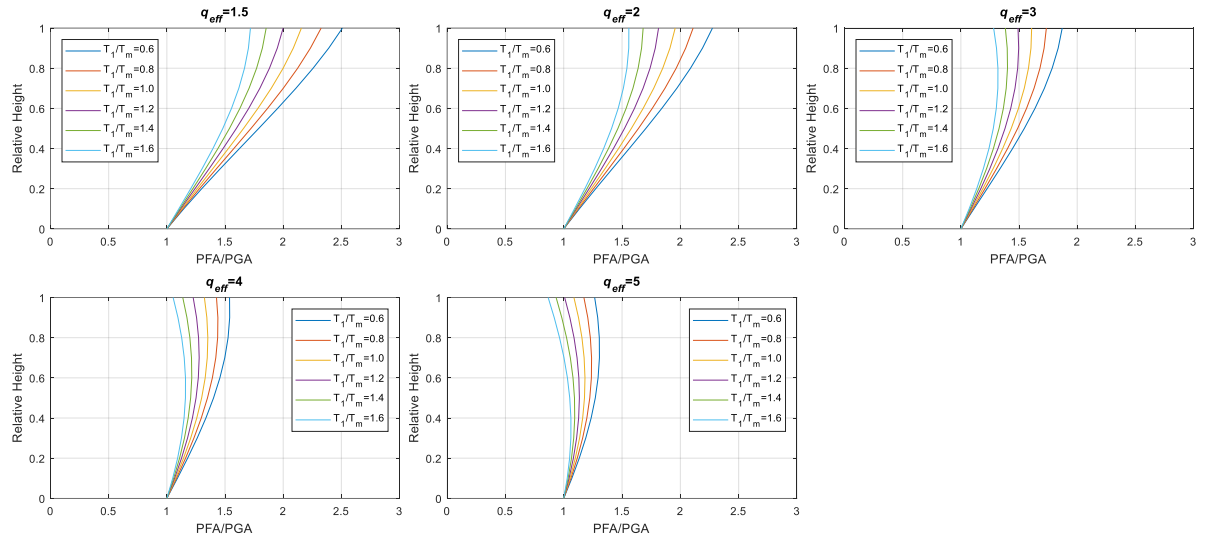


Figure 7-42 Floor acceleration magnification predicted by the model for various T_1/T_m values for 2 storey buildings

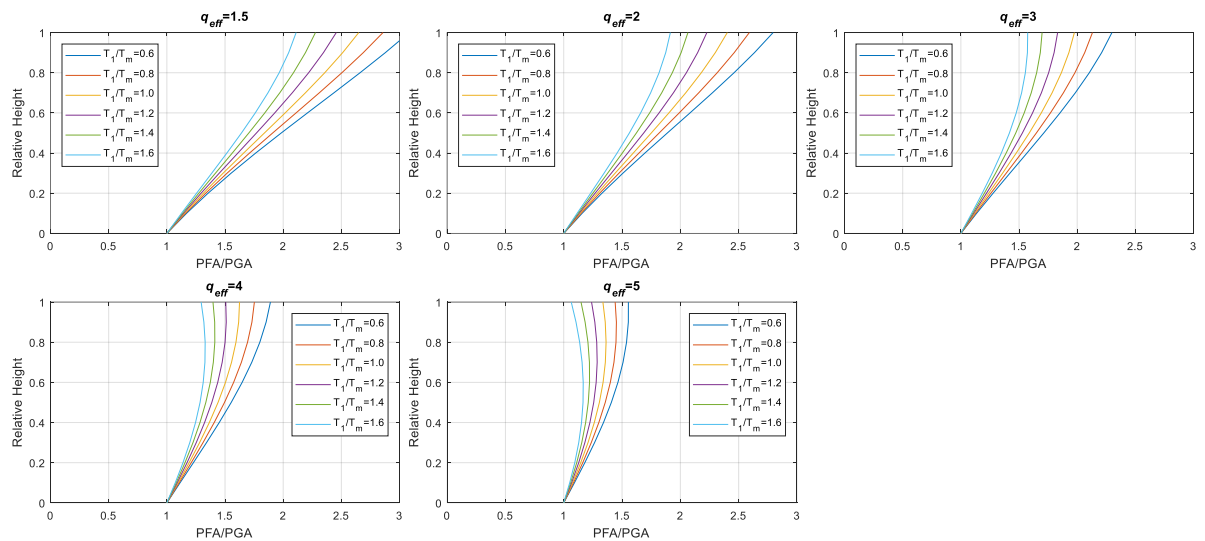


Figure 7-43 Floor acceleration magnification predicted by the model for various T_1/T_m values for 7 storey buildings

7.5.1.2 Examination of Model Residuals

As with the model for peak inter-storey drift, the impacts of a series of potential variables on the model residuals were examined in order to investigate the applicability of the fitted model. Residuals were calculated as the observed floor acceleration magnification less the value predicted by Equation 7.40 for the log transformed data. As shown in Figure 7-44, the average value of the residuals is always close to zeros and no clear trends are evident, suggesting that

the model accounts for the variation in acceleration response with various parameters reasonably well.

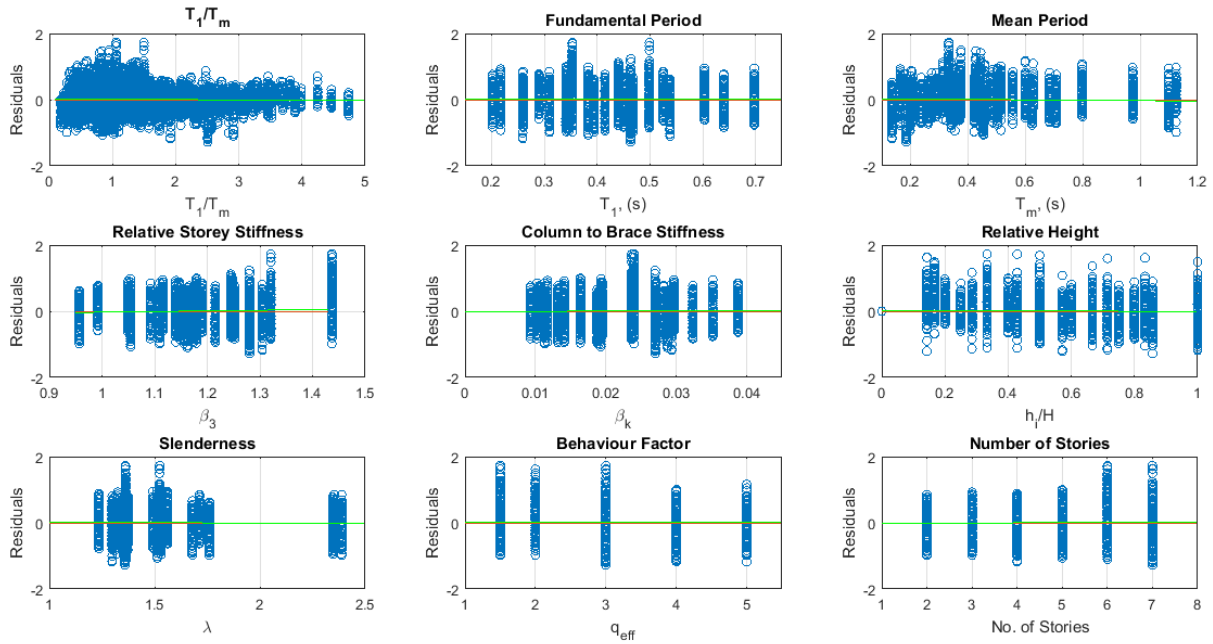


Figure 7-44 Impact of various parameters on model residuals for floor acceleration magnification

7.5.1.3 Comparison with other Models and Design Codes

Figure 7-45 compares the model for floor acceleration magnification developed in this study (Equation 7.34) with a number of other prediction methods: the model for braced frames proposed in the FEMA P-58 simplified analysis method (Equation 7.32), an approximation of the first mode reduced method developed by Rodriguez et al. (2002) (Equation 7.35) and the value proposed by Eurocode 8.

In Figure 7.43, average values across the databank for T_1/T_m and number of stories were used to calculate the values of floor acceleration magnification with the model developed in this study, while the average value for T_1 was used in the FEMA P-58 model.

As discussed in Chapter 4, the first mode reduced model (Rodriguez et al., 2002) involves combining the first r modal responses with the contribution of the first mode reduced to account for nonlinear behaviour, as described by Equation 7.35:

$$PFA = \sqrt{\left(\Gamma_1 \phi_{roof}^1 \frac{S_a(T_1)}{q_{eff}}\right)^2 + \left(\sum_{n=2}^r \Gamma_n \phi_{roof}^n S_a(T_n)\right)^2} \quad 7.35$$

For Figure 7-45, the value of this is approximated using the first and second modes only. The elastic spectral acceleration values proposed by Eurocode 8 for $S_a(T_1)$ and $S_a(T_2)$ are used for this purpose, assuming that the first and second structural periods lie in the constant acceleration region of the Eurocode 8 spectrum. The values of the modal participation terms, $\Gamma_1\varphi^1$ and $\Gamma_2\varphi^2$, used to develop the Figure are the values recommended by Rodriguez et al. (2002), 1.5 and 0.44 respectively.

Values for peak floor acceleration in Eurocode 8 are given for the design of non-structural components. A linearly increasing profile of floor acceleration along the height of the building is proposed, based on the assumption that the response is dominated by the fundamental mode of vibration, which can be approximated as linear for low and mid-rise buildings. Effectively, this formulation implicitly assumes that the floor acceleration magnification factor ranges linearly from a value of 1 at the base, i.e. the *PFA* is equal to the *PGA*, to 2.5 at the top of the structure.

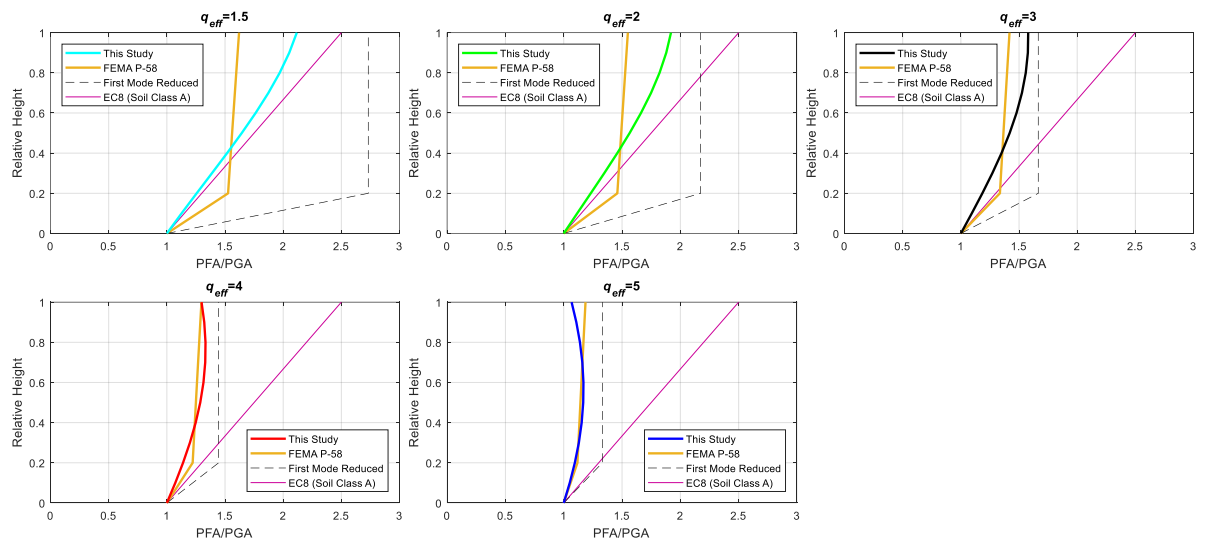


Figure 7-45 Comparison of various model for PFA/PGA

From Figure 7-45, it can be seen that for q_{eff} values above 3, the shape of the profile and the values predicted and by the model proposed here and the FEMA P-58 model are similar. However, at lower q_{eff} values the model becomes closer to the Eurocode 8 linear prediction. This suggests that regression equation employed here is more capable than the FEMA P-58 equivalent of capturing the transition between the linearly increasing and saturated response profiles observed with increasing q_{eff} . Eurocode 8 appears to overestimate peak acceleration response, supporting the conclusions of a series of other studies for different frame types (Wieser et al., 2013, Flores et al., 2015). The first mode reduced method also appears to overestimate floor acceleration magnification in all cases, assuming that the values proposed by

Equation 7.34 are accurate. For the lower q_{eff} values the extent of this overestimation is severe, however for $q_{eff} = 3$ and above the predication are reasonably similar to those predicted by the model developed here. While making this comparison the extent of the assumptions and simplifications in the development of the first mode reduced model for the dataset should be borne in mind.

7.5.1.4 Variation in Peak Floor Acceleration

As with peak inter-storey drift, for performance assessment through FEMA P-58 requires a distribution describing peak acceleration response. This distribution is described by a median value, obtained from Equation 7.34, and a distribution term β_{FA} . As with drift, this dispersion term used in performance assessment is calculated as a combination of uncertainty due to record-to-record variability and, β_{aa} , and modelling uncertainty, β_m , combined using a SRSS approach:

$$\beta_{FA} = \sqrt{\beta_{aa}^2 + \beta_m^2} \quad 7.36$$

In order to obtain a first estimate for a β_{aa} value the standard deviation of the model residuals was calculated as 0.29. Prior to this calculation, the peak floor accelerations at the ground level were removed from the dataset, as by the nature of the regression model there is no variation associated with these as it obeys the constraint that $PFA/PGA=1$. The impact of this on the distribution of the residuals is illustrated in Figure 7-46. From these histograms, it can be appreciated that including the data points with $h_i/H=0$ can distort analysis of the variation. It can also be seen that the distribution of the residuals is not entirely normal, however for large sample sizes this is particularly important for the estimation of regression coefficients (Gelman and Hill, 2006).

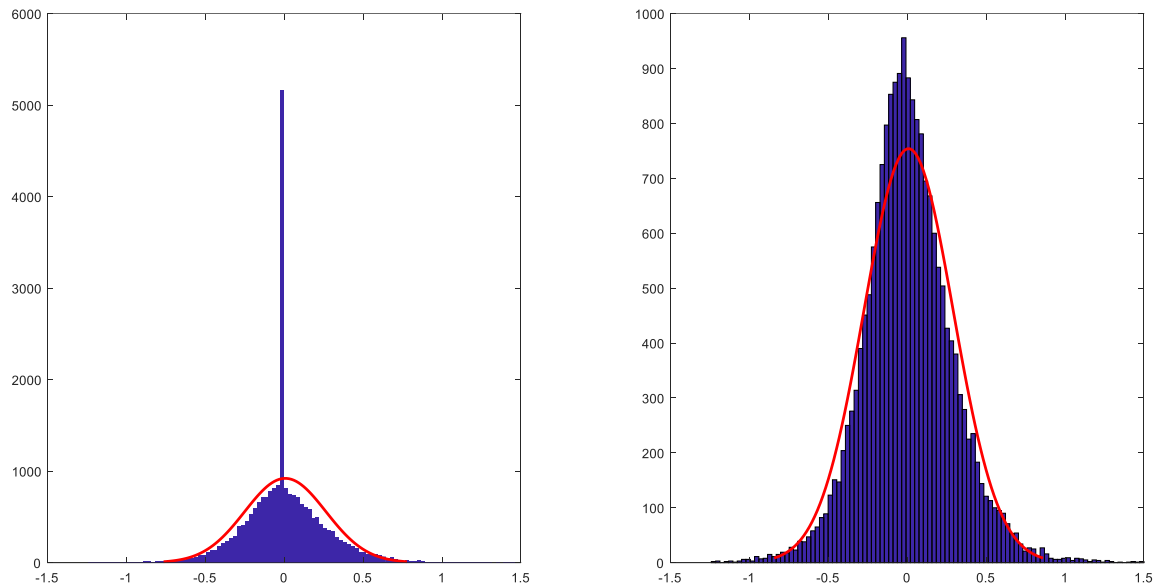


Figure 7-46 Distribution of residuals (a) including and (b) excluding the values at ground level

The impact of various potentially influential parameters on the 16th and 84th percentile of the residuals was then examined. As before, this was done by performing quantile regression to obtain first order polynomials representing the change in the 16th and 84th percentiles with various parameters. The results of these regressions are presented in dashed blue lines in Figure 7-47 and 7-48. Figure 7-47 also shows the value for the standard deviation across the dataset, in dashed red lines, and the individual residual data points for each analysis run. These data points are removed in Figure 7-48, which allows the variations in the 16th and 84th percentiles to be examined more clearly.

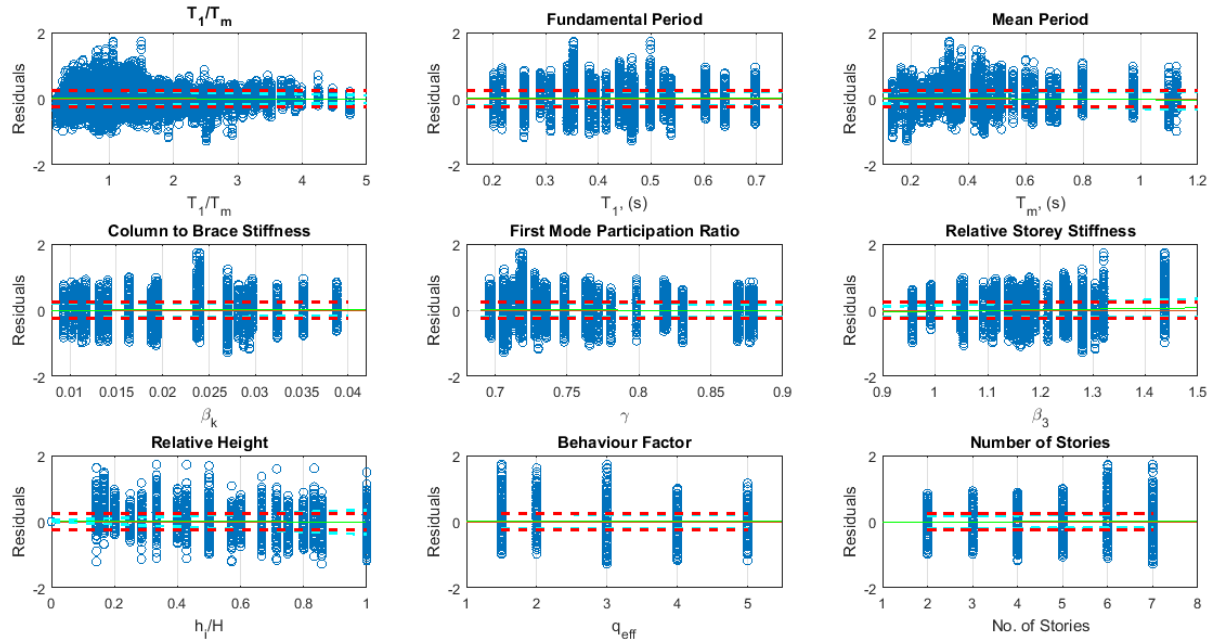


Figure 7-47 Model residuals and variation in model residuals with various parameters; standard deviation ($\pm\sigma$) across the dataset shown in red, variation in 16th and 84th percentiles (i.e. change in σ) with each parameter shown in blue

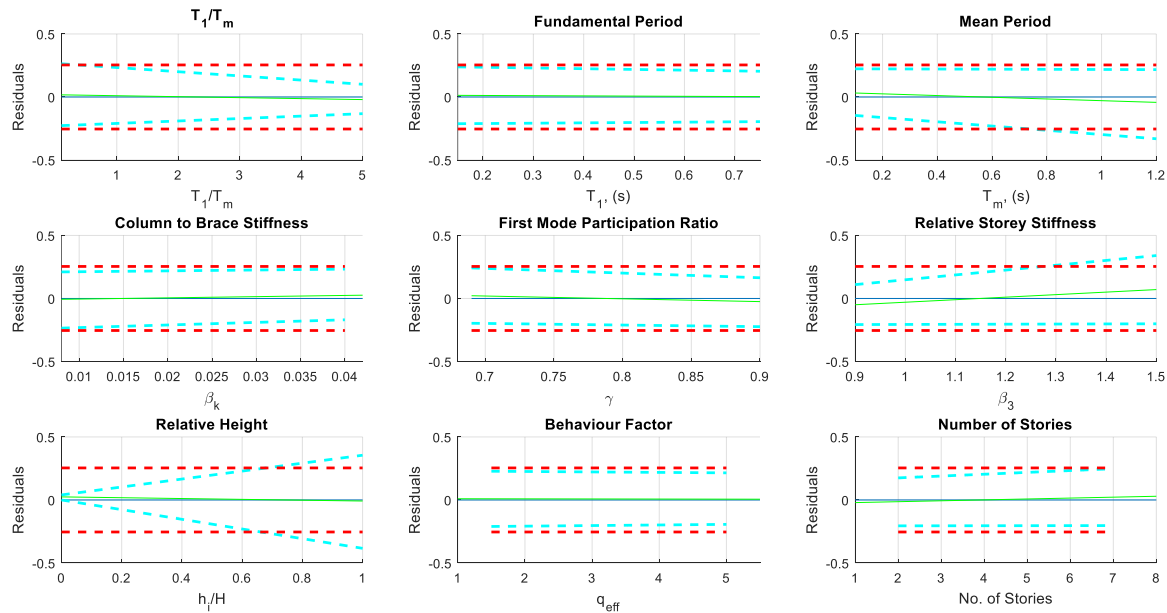


Figure 7-48 Variation in model residuals with various parameters; standard deviation ($\pm\sigma$) across the dataset shown in red, variation in 16th and 84th percentiles (i.e. change in σ) each parameter in blue

From these figures it can be seen that a change in variation with respect to T_1/T_m is evident; there is greater scatter at lower T_1/T_m values. However, this is partly because of the lower number of analyses carried out for higher T_1/T_m values. There is also a change in variability with relative height; a greater scatter is noted in the upper stories. However, it is not possible to

include this directly in the FEMA P-58 simplified methodology as implemented in PACT, as a single distribution parameter is used for all floor levels.

There is little variation in the distribution with respect to q_{eff} . Given that q_{eff} represents the ground motion intensity, this means that little difference is noted in the record-to-record variability as the ground motion intensity changes. Similarly, there is little variation in the fitted 16th or 84th percentiles as the number of stories changes.

Examination of these trends indicates that a constant value of 0.3 should be adopted for the lognormal standard deviation. Figure 7-49 shows how this value compares with the standard deviation of the residuals and the variation in the 16th and 84th percentiles with T_1/T_m and relative height; it can be appreciated how a value of 0.3 is a reasonable estimate for the standard deviation of the residuals.

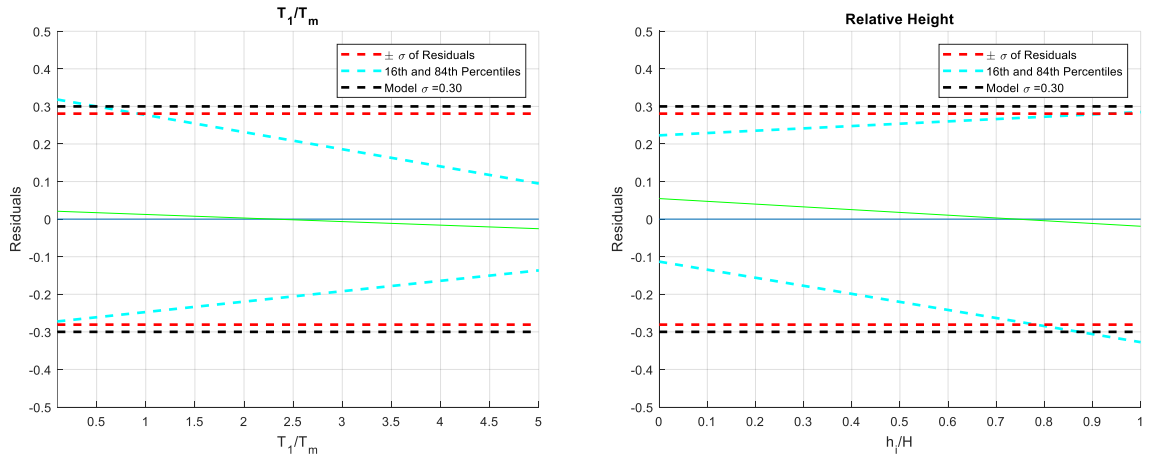


Figure 7-49 Comparison of the trends in variation with the final value for lognormal standard deviation of 0.3

As per the discussion in the previous section on the variation in δ_{mod} and ϑ_{mod} and inter-storey drift, the lognormal standard deviation for the lognormally distributed floor acceleration modification values can be shown to be equal to the lognormal standard deviation describing the distribution of peak floor acceleration. Hence, this value of 0.3 can be used for β_{aa} within the FEMA P-58 framework. As before, a value of $\beta_m = 0.5$ was assumed for modelling uncertainty, in accordance with the FEMA P-58 recommendations. Therefore, the overall value for distribution used in performance assessment for peak floor acceleration, β_{FA} , is given by:

$$\beta_{aa} = 0.3 \quad 7.37$$

$$\beta_m = 0.5 \quad 7.38$$

$$\beta_{FA} = \sqrt{\beta_{aa}^2 + \beta_m^2} = \sqrt{0.3^2 + 0.5^2} = 0.58 \quad 7.39$$

7.6 Rapid Performance Assessment Application

In order to assess the proposed ‘rapid’ performance assessment methodology, the 5 storey CBF designed using $q=4$ at the Oakland site and examined using conventional NLTHA-based performance assessment in Chapter 6 was re-examined using the proposed ‘rapid’ method. The equations developed in Sections 7.4 and 7.5 were applied to predict *EDPs*.

Performance assessment is normally carried out at a series of intensity levels, each of which is defined by a value of spectral acceleration, $S_a(T_1)$. As discussed, using Equation 7.16, a corresponding q_{eff} value can be calculated for each $S_a(T_1)$. Thus, a unique q_{eff} value represents every intensity level of interest for performance assessment. This allows corresponding *EDP* values to be calculated for each intensity level using Equations 7.17 or 7.18 and 7.34.

The steps for obtaining *EDPs* in the proposed Rapid performance assessment procedure are as follows:

1. Perform an Eigenvalue analysis
 - a. Obtain the fundamental period, T_1 , the first mode shape ϕ^1 , calculate the first mode mass participation ratio γ and the relative storey stiffness, β_3 .
2. Obtain the base shear and peak inter-storey drift at the point of yield, $V_{BS,yield}$ and ϑ_1 respectively
 - a. This can be achieved through a nonlinear static, i.e. pushover, analysis.
 - b. Alternatively, it is possible to perform linear static analysis, and use brace fragility functions developed elsewhere, for example by Lignos and Karamanci (2013), to predict the value at which nonlinear behaviour, i.e. brace buckling, is initiated, ϑ_1 . The base shear corresponding to this assumed value of yield in the linear analysis can be taken as $V_{BS,yield}$. This method of predicting avoids the need for nonlinear analysis and the development of a nonlinear model, but there is a corresponding trade off in thoroughness.
3. Obtain appropriate T_m values
 - a. There are at least three possible ways to do this:
 - i. Select a series of site-specific records to match, for example, the Conditional Spectrum, and calculate the mean T_m of these. This method was used in this study, using the records selected in Chapter 6.

- ii. Calculate an appropriate T_m value for the site, as a function of earthquake magnitude and site-source distance, using the expressions proposed by Rathje et al. (2004)
 - iii. Calculate an appropriate T_m value for the site, using the T_c value of the Eurocode 8 elastic spectrum, using Equation 7.13 (Kumar et al., 2011). Unlike two previous proposed methods, this does not require hazard deaggregation.
- 4. Calculate q_{eff}
 - a. Select intensity levels to carry out performance assessment and obtain $S_a(T_1)$ values for each.
 - b. Calculate q_{eff} using Equation 7.16.
- 5. Calculate inter-storey drift at each storey level
 - a. Calculate δ_{mod} or ϑ_{mod} from Equation 7.10 or 7.11
 - b. Calculate peak global drift, Δ_{max} , or peak inter-storey drift, ϑ_{max} , from Equation 7.17 or 7.18.
 - c. Calculate inter-storey drift at each floor level, $\vartheta_{max,i}$, using the mode shape, as per Equation 7.20 or 7.21
 - d. Calculate value for variation in inter-storey drift based on Equation 7.30 or 7.31
- 6. Calculate peak floor acceleration at each storey level
 - a. Use Equation 7.34 to obtain the floor acceleration magnification
 - b. Calculate peak floor acceleration by multiplying this by the PGA for the intensity level
 - i. Here, PGA is calculated as the average of a series of site specific records, in a similar manner to T_m
 - ii. Alternatively, PGA can be calculated directly from PSHA
 - c. Calculate value for variation in peak floor acceleration based on Equation 7.39
- 7. Once the median and variation of the *EDPs* have been calculated, performance metrics can be calculated using the FEMA P-58 simplified procedure implemented in PACT.

7.6.1 EDP Evaluation

7.6.1.1 Peak Inter-Storey Drift Prediction

Figures 7-50 compares the median peak inter-storey drift profiles obtained from NLTHA with those obtained using Equation 7.20, i.e. using δ_{mod} , and the FEMA P-58 method. There is a reasonably good match between the NLTHA results and the model developed here using δ_{mod} . In general the prediction of roof drift is very close, while the use of the mode shape to create an

inter-storey drift profile over the frame height appears to work well in most cases. There is only one case, at the second level for the 2% in 50 year intensity level, where the difference between predicated and median recorded inter-storey drift exceeds 0.1%. There is also a reasonably good match between the predicted spread of data, as indicated by the 16th and 84th percentile in dashed lines and the corresponding percentiles from NLTHA.

The FEMA P-58 method predicts greater values for inter-storey drift than both model developed here and the NLTHAs, particularly for the higher intensity levels. For the lower intensity levels where behaviour is generally linear, a closer match to the NLTHA median is obtained. However, it is still clear that the values predicted by the model developed here specifically for CBFs designed to Eurocode 8 appear to give a better prediction of the NLTHA results than the more generally applicable FEMA P-58 model.

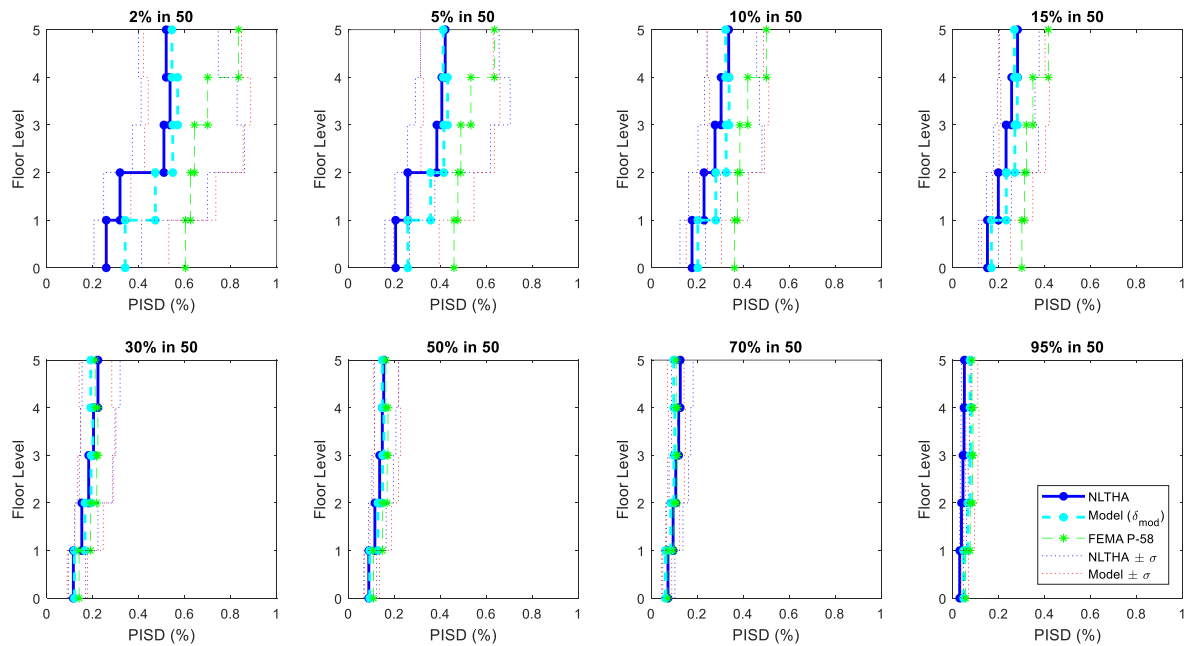


Figure 7-50 Comparison of median peak inter-storey drift profile obtained from NLTHA with values predicted by the model developed here using δ_{mod} and the FEMA P-58 simplified analysis method

Figure 7-51 shows the inter-storey drift profiles developed from ϑ_{mod} , i.e. the predicted peak inter-storey drift. The values obtained are generally conservative and almost always predict greater inter-storey drift than recorded in NLTHA. The spread of data is also observed to be greater than for the NLTHA results. However, the values obtained are still not as large as those predicted by the FEMA P-58 method, which substantially overestimates NLTHA values in many cases.

In general the match is better for the lower intensity levels, which are more important for lifetime performance assessment. The absolute value of the difference between the median peak inter-storey drifts is only greater than 0.1% in 3 cases, for the first and second level at the 2% in 50 year intensity level at the second level at the 5% in 50 year intensity level. Relatively, the largest differences occur at the 95% in 50 year intensity level, where peak inter-storey drift is on average approximately 60% greater for the model compared to the NLTHA median. However, as the values of drift at this level are very small, all less than 0.1%, will have little impact for damage, and subsequently performance, assessment.

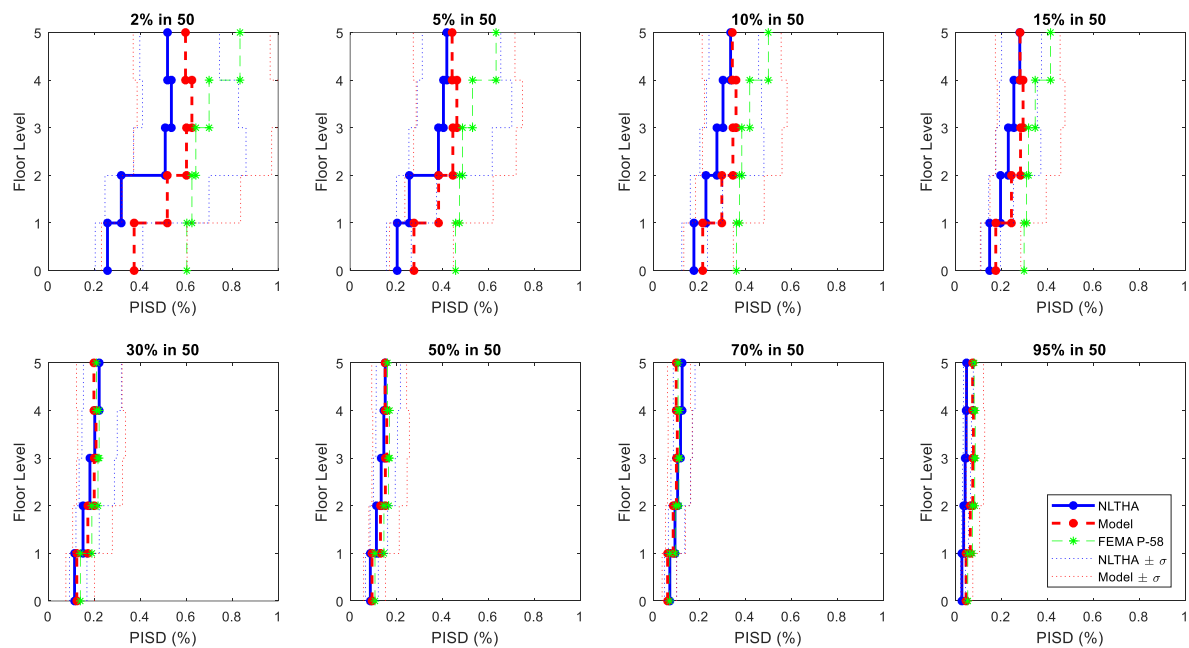


Figure 7-51 Comparison of median peak inter-storey drift profile obtained from NLTHA with values predicted by the model developed here using ϑ_{mod} and the FEMA P-58 simplified analysis method

Comparing Figures 7-50 and 7-51, it is clear that the inter-storey drift profile obtained from ϑ_{mod} is more conservative than the corresponding one obtained from δ_{mod} . This could be expected, given that ϑ_{mod} is intended to capture soft-storey type behaviour whereas δ_{mod} reflects roof drift. At the higher intensity levels, a better match with the NLTHA results, particularly in terms of the distributions, is obtained using the δ_{mod} based model. However, at the lower intensity levels both methods compare relatively well with NLTHA.

7.6.1.2 Peak Floor Acceleration

The Peak Floor Acceleration values obtained from Equation 7.34 are compared with the median values from NLTHA in Figure 7-52. In general, there is a good match between the two. The difference between the medians from the NLTHA and the model is less than $0.1g$ in the majority of cases and is less than $0.15g$ for all cases bar the first floor level at the 2% in 50 year intensity

level and the 4th floor level at the 15% in 50 year intensity. In general, the extent of the differences between the values are not large enough to significantly affect the expected levels of damage.

Values obtained using the expression for peak floor acceleration proposed in FEMA P-58 are also shown. In general, these values are further away from the median values obtained from NLTHA than the values predicted by Equation 7.34. The FEMA P-58 profile doesn't appear to capture the increase in accelerations over the frame height well; it tends to overestimate PFA at the lower stories, agree well (in some cases better than the model developed here) around mid-height and underestimates values at the roof level. This is true for all intensity levels, even if the scale of the plots presented in Figure 7-51 mean that this is difficult to see for the lower intensity levels. As with inter-storey drift, this suggests it is possible to get improved predictions of frame response when prediction methods are developed specifically for a particular design code or methodology.

The model developed here isn't necessarily conservative with the NLTHA acceleration response being both under and over predicted at different floor levels and intensities. In some cases, it fails to capture the increases in peak floor acceleration at the roof level; given that these become more common with increased frame height, the applicability of the model for taller frames is questionable. Like the medians, the 16th and 84th percentiles for the NLTHA and the model, as marked by the dashed lines, are generally quite similar.

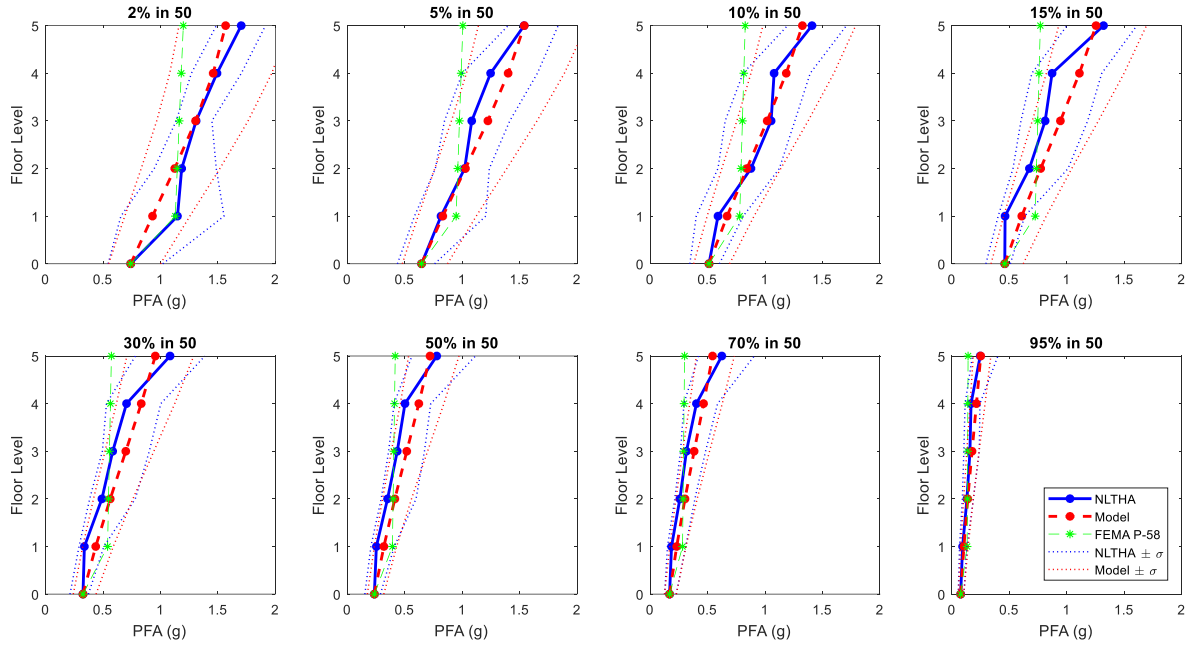


Figure 7-52 Comparison of median peak inter-storey drift profile obtained from NLTHA with values predicted by the model developed here and the FEMA P-58 simplified analysis method

7.6.2 Collapse & Excessive Residual Drift Estimate

Collapse probability can be estimated quickly without NLTHA using the Equations proposed in FEMA P-58. This is termed Judgement Based Collapse Fragility (FEMA, 2012a).

The median value of spectral acceleration defining the collapse fragility function, $S_a(T_1)_{collapse}$, is given by:

$$S_a(T_1)_{collapse} = 4S_d, \quad \text{for } T_1 \geq 0.6s \quad 7.40a$$

$$S_a(T_1)_{collapse} = 4S_d[5(1 - C_4)T_1 + 3C_4 - 0.2], \quad \text{for } 0.4s < T_1 < 0.6s \quad 7.40b$$

$$S_a(T_1)_{collapse} = 4C_4S_d, \quad \text{for } T_1 \leq 0.6s \quad 7.40c$$

Where:

$$C_4 = \frac{\sqrt{2q - 1}}{q} \quad 7.41$$

A value of 0.7 is advised for the lognormal standard deviation. Figure 7-53 compares the collapse fragility function defined using this method with the one developed in Chapter 6 using NLTHA for the 5 storey CBF at the Oakland site designed using $q=4$. It can be seen that for this particular structure the two fragility functions are very similar, with the FEMA P-58 method being slightly more conservative.

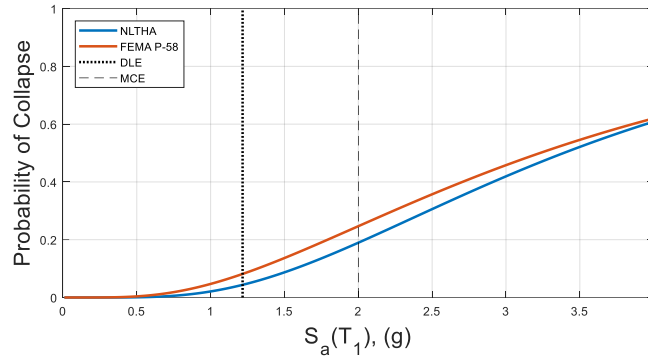


Figure 7-53 Comparison between the Collapse Fragility Functions developed through NLTHA, as discussed in Chapter 6, and using the FEMA P-58 Judgment Based method

Residual drift can be predicted based on peak inter-storey drift, as discussed in Chapter 6, using Equation 6.1. However, from a lifetime performance assessment perspective, the impact of collapse and residual drift is expected to be minimal.

7.6.3 Performance Metrics

Performance assessment was carried out using PACT with the *EDPs* evaluated in the previous section. Two calculations were performed, one using the inter-storey drift profile based on roof drift, i.e. δ_{mod} , as shown in Figure 7-50 and one based on peak inter-storey drift, i.e. ϑ_{mod} , shown in Figure 7-51. The building performance model, which lists the damageable components and their sensitivity to seismic action, used in performance calculation was the same as that used in Chapter 6 to carry out NLTHA-based assessment. As before, performance is represented by financial losses due to a combination of repair costs and downtime.

The expected losses calculated using the δ_{mod} -based drift profile are compared to the losses calculated using NLTHA in Figure 7-54. It can be seen that results are very similar; total expected losses differ by only 0.3% of initial cost. There is a slight overestimation of drift dependant losses and underestimation of acceleration dependant losses when the ‘rapid’ method is used, however these differences are minimal. Collapse dependant losses are approximately twice as large for the Rapid method, but the overall contribution to lifetime losses is still relatively minor.

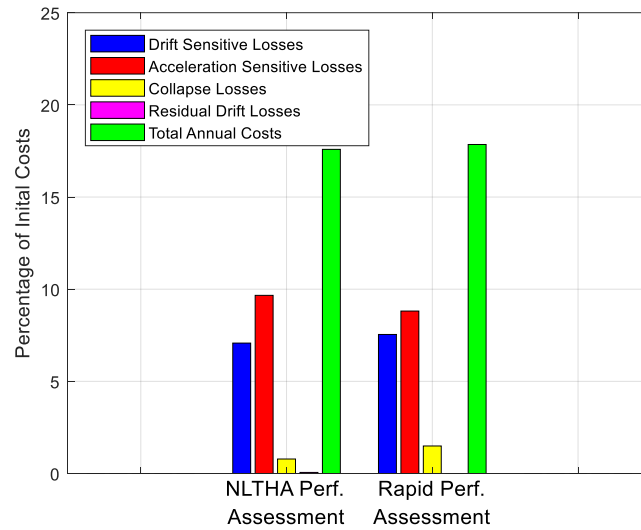


Figure 7-54 Expected losses calculated using $EDPs$ obtained through NLTHA and the rapid method using δ_{mod} for the $q=4, 5$ storey CBF at the Oakland site

Figure 7-55 shows the expected losses calculated using the more conservative ϑ_{mod} based drift profile. The rapid method predicts greater losses than the NLTHA. It can be seen from Figure 7-54 that this is primarily due to drift dependant losses, which are overestimated by approximately 2.5% of initial costs.

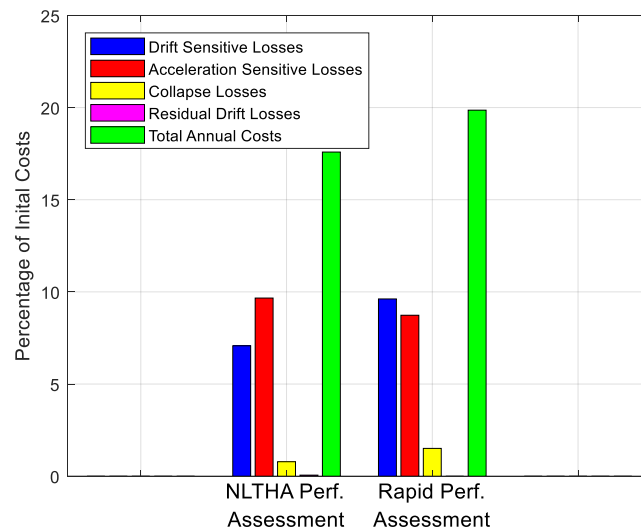


Figure 7-55 Expected losses calculated using $EDPs$ obtained through NLTHA and the rapid method using ϑ_{mod} for the $q=4, 5$ storey CBF at the Oakland site

To examine the results shown in Figure 7-54 in more detail, Figure 7-56 and 7-57 compare the intensity level specific performance metrics at the MCE, DLE and SLE calculated using the NLTHAs and the proposed rapid method. Drift and acceleration dependant losses are generally similar for the SLE and DLE. The most obvious difference between the two methods is in collapse

dependant losses, which are more prominent for the Rapid method. This is particularly true at the MCE, where the greater number of collapses also means slightly reduced drift and acceleration dependant losses are observed.

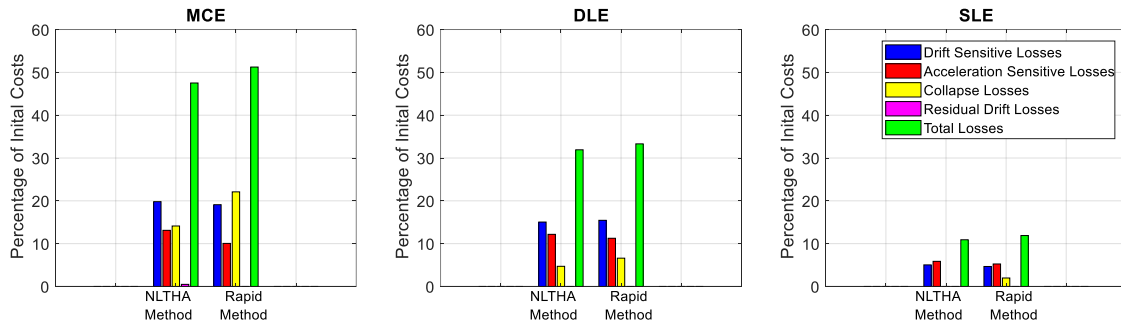


Figure 7-56 Comparison of intensity-specific losses calculated using δ_{mod} for the MCE, DLE and SLE for the $q=4$ design at the Oakland site

When ϑ_{mod} is used to predict inter-storey drift, as shown in Figure 7-57, losses are greater at each intensity level for the rapid method, primarily due to greater drift dependant losses. Otherwise, similar trends to the δ_{mod} method shown in Figure 7-56 are observed. Again, acceleration dependant losses are generally quite similar, with only a small difference noticeable at the MCE, while collapse dependant losses are more prominent for the ‘rapid’ method, particularly at the MCE.

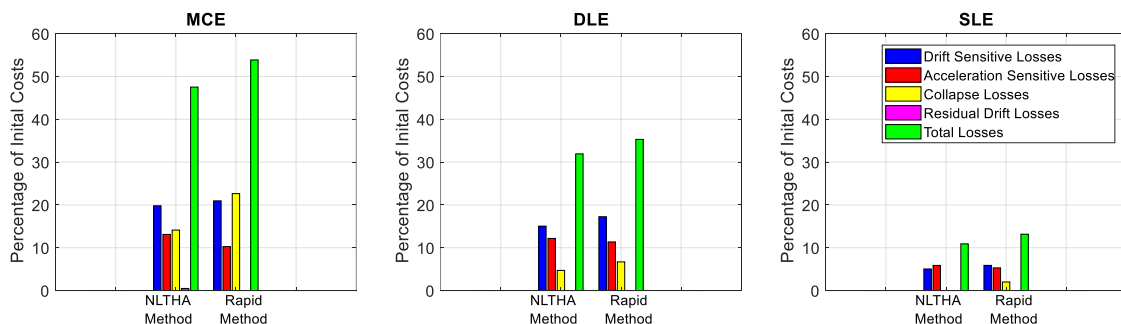


Figure 7-57 Comparison of intensity-specific losses calculated using ϑ_{mod} for the MCE, DLE and SLE for the $q=4$ design at the Oakland site

Comparison between the two performance assessment methods was also made for the 5 storey CBFs using the $q=2$ design at the Oakland site and the $q = 4$ design at the Seattle site. This allows the ability of the proposed method to capture the variations with q and seismic hazard, as noted in the NTLHA-based study, to be assessed. The results are presented in Figures 7-58 to 7-61. Compared to the NLTHA-based assessment, lifetime losses are estimated to within 2.5% of initial costs. This is primarily due to the overestimation of drift dependant losses. However, the general trends observed in NLTHA based analysis, namely the increase in lifetime losses with q and seismic hazard, are still observed. Table 7-8 summarizes the comparisons.

Table 7-8 Comparison between lifetime losses calculated by NLTHA and the proposed simplified method for three case study buildings

Site	q	NLTHA Losses (% of Initial Cost)	Drift Calculation Method	Rapid Losses (% of Initial Cost)	Difference (% of Initial Cost)	Percentage Difference
Oakland	4	17.6	δ_{mod}	17.9	0.3	2%
			θ_{mod}	19.9	2.3	13 %
Oakland	2	13.6	δ_{mod}	12.6	-1.0	-7%
			θ_{mod}	15.9	2.3	17 %
Seattle	4	8.2	δ_{mod}	7.4	-0.8	-9%
			θ_{mod}	10.3	2.1	25%

Considering the $q=2$ design at the Oakland site, Figures 7-58 and 7-59 show the comparison between the NLTHA and ‘rapid’ methods for drift response calculated using δ_{mod} and ϑ_{mod} respectively. The total lifetime losses calculated using δ_{mod} are underestimated compared to the NLTHA based results, whereas the losses calculated using ϑ_{mod} are overestimated. Clearly, in both cases this is attributable to differences in the drift dependant losses, which are underestimated using δ_{mod} and overestimated using ϑ_{mod} . The value for drift dependant and therefore overall losses obtained using δ_{mod} is closer to the NLTHA-based value, despite not being conservative. Therefore, it is recommended to use δ_{mod} and Equation 7.11 to predict drift for performance assessment. Acceleration and collapse dependant losses, which are independent of the drift prediction method, are predicted reasonably well.

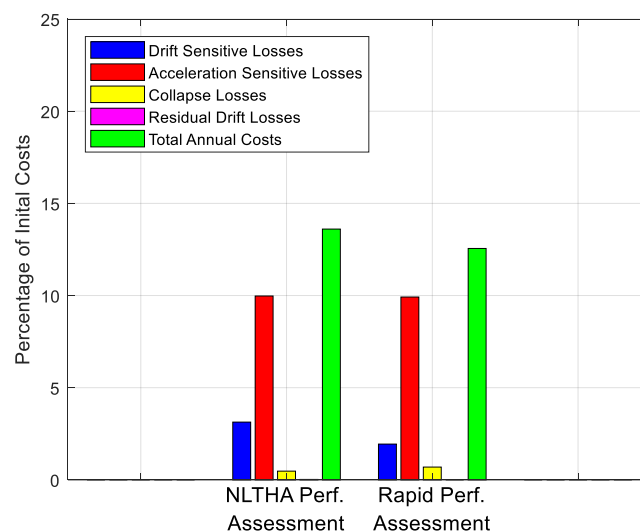


Figure 7-58 Expected losses calculated using EDPs obtained through NLTHA and the rapid method using δ_{mod} for the $q=2$, 5 storey CBF at the Oakland site

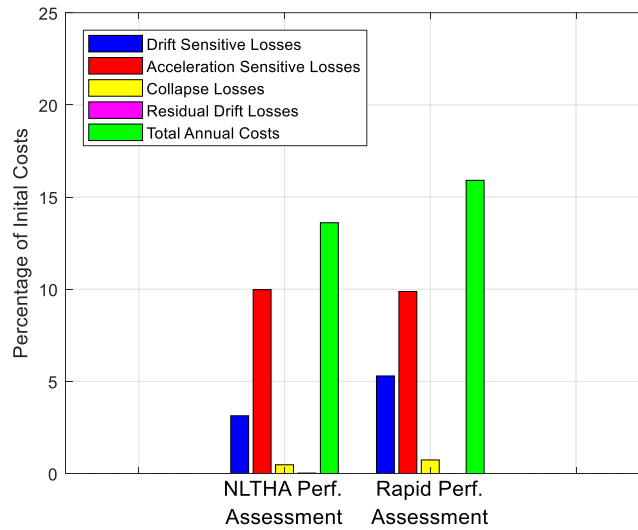


Figure 7-59 Expected losses calculated using $EDPs$ obtained through NTLHA and the rapid method using ϑ_{mod} for the $q=2, 5$ storey CBF at the Oakland site

Similar trends are observed for the Seattle $q=4$ design, as is shown in Figure 7-60 and 7-61. Acceleration and collapse dependant losses are predicted reasonably well, with drift dependant losses under predicted using δ_{mod} and over predicated using ϑ_{mod} . However, again the δ_{mod} based method gives a closer estimation of the NTLHA based value.

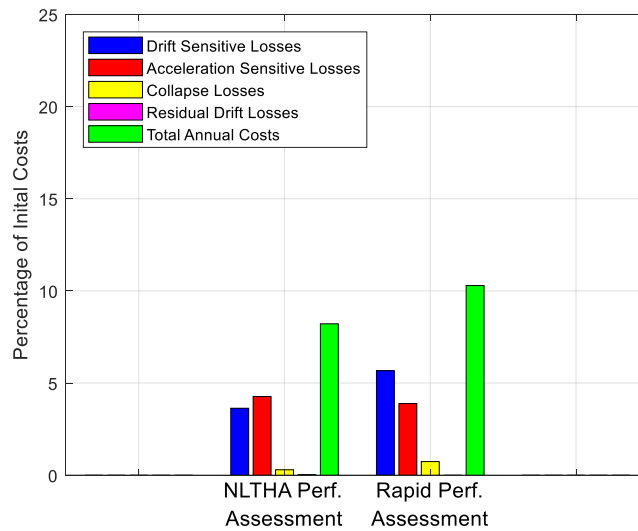


Figure 7-60 Expected losses calculated using $EDPs$ obtained through NTLHA and the rapid method for the $q=4, 5$ storey CBF at the Seattle site; with inter-storey drift at each level calculated based on predicted global drift, i.e. using δ_{mod}

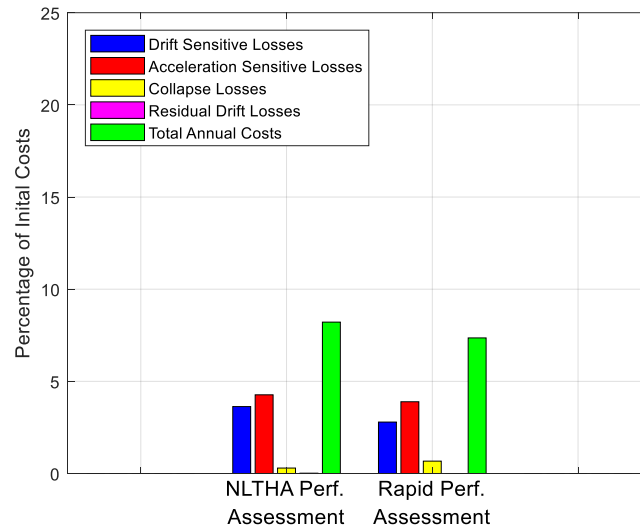


Figure 7-61 Expected losses calculated using $EDPs$ obtained through NLTHA and the rapid method for the $q=4, 5$ storey CBF at the Seattle site; with inter-storey drift at each level calculated based on predicted peak inter-storey drift, i.e. using ϑ_{mod}

Looking at these results, it can be concluded that, broadly speaking, the results compare reasonably well with NLTHA based methods and the ‘rapid’ method appears capable of capturing general performance trends. The acceleration dependant losses predicted are almost identical to those obtained through NLTHA. This suggests that the model for predicting PFA response is adequate for performance assessment for frames of this type. The proposed method has more difficulty accurately predicting drift dependant losses; these tend to be underestimated if δ_{mod} is used and overestimated using ϑ_{mod} . For the cases examined, the values obtained using δ_{mod} are closer to the values obtained from NLTHA, but are not always conservative. Having said that, when δ_{mod} is used, the method appears capable of predicting lifetime losses within $\pm 10\%$ of NLTHA, which given the many uncertainties associated with lifetime performance assessment in general, seems a reasonable outcome.

7.7 Conclusion

The Chapter proposes a method to carry out performance assessment of CBFs designed to Eurocode 8 without recourse to NLTHA, termed ‘rapid’ performance assessment. This is potentially beneficial as the complexity and computational expense of NLTHA make it, and therefore performance assessment, unattractive for practicing engineers.

Firstly, the drift response of CBF structures designed to Eurocode 8 was examined. Using results from a comprehensive set of time history analyses, expressions were developed to predict global and maximum drift modification factors, δ_{mod} and ϑ_{mod} respectively, for CBFs designed to EC8. Both δ_{mod} and ϑ_{mod} were shown to be dependent on the ratio of fundamental structural period

to ground motion mean period, T_1/T_m , as well as the ground motion intensity, which is represented by the effective behaviour factor, q_{eff} . Either of these terms, in conjunction with the mode shape, can be used to predict inter-storey drift at any level of a building, which in turn can be used for performance assessment.

Likewise, expressions to predict floor acceleration magnification were developed, using data obtained from an extensive number of NLTHAs. As with drift, T_1/T_m and q_{eff} , as well as the relative height of a storey level and the total number of stories, were identified as relevant parameters for acceleration prediction. For both drift and acceleration, improved response prediction was achieved using the Eurocode 8 specific equations developed here for CBFs designed to Eurocode 8 compared to the more generally applicable expressions proposed in FEMA P-58.

To assess the applicability of the developed equations, performance assessment was carried out for buildings which were previously assessed using conventional NLTHA methods. This process was performed using both δ_{mod} and ϑ_{mod} to calculate drift; a closer match to NLTHA results were obtained using δ_{mod} . The results of this process were reasonably successful; when δ_{mod} is used to predict drift response total lifetime losses were calculated within 10% of the value of those calculated using NLTHA.

8 Experimental Setup for Hybrid Simulation

8.1 Introduction

As discussed in Chapter 5, hybrid simulation is an experimental testing method that involves using actuator(s) to apply a calculated displacement for a particular timestep to an experimental structure before measuring some aspect of the response and using this in the calculation of the next target displacement. In substructured hybrid testing some parts of the structure, typically critical elements whose behaviour is not easily modelled, are tested experimentally, whilst components with more predictable behaviour are modelled numerically. Hybrid simulation has proven popular with earthquake engineers examining the seismic response of buildings, where substructuring allows full scale experiments to be performed whilst remaining cost effective in comparison to shake table tests. Typically, time-history analysis is performed using finite element software and the force required to impose the calculated displacement of the experimental substructure is measured at each time-step and used to solve the equations of motion at for the next timestep.

The goal of the experimental work performed in this study is to investigate the seismic response of a steel Concentrically Braced Frame (CBF) structure, with the ultimate ambition of improving lifetime assessment methods for this form of structure. The response of CBFs to large earthquakes is highly nonlinear and challenging to capture correctly in a purely numerical model (for example see Ryan et al. (2017)). Therefore, by alleviating the need to model some of the more complex components, substructured hybrid simulation can be a valuable tool for investigating the seismic performance of CBFs. The overall intended outcome of the tests is to assess and improve the capabilities of numerical models; which in turn will lead to improved ability to predict *EDPs* and then evaluate performance metrics of interest. This will be achieved through the application of a model updating algorithm to optimize modelling parameters.

This Chapter deals with the development of the experimental setup for a hybrid simulation testing program. Firstly, the structure to be tested, and the concepts underpinning its design, are introduced. Then the numerical modelling required for hybrid simulation is discussed. Once an adequate structural model was developed, preliminary testing was undertaken to verify the test method. Some of the issues that arose in this preliminary testing are discussed and the solutions used to resolve them are presented.

8.2 Experimental Setup

8.2.1 Experimental Facility

The structures laboratory at Trinity College Dublin (TCD) is equipped with an MTS real-time hybrid test system. The experimental hardware consists of a Series 111 MTS Accumulator and a 150kN capacity high speed linear hydraulic actuator with a 250mm (± 125 mm) stroke. The computer hardware comprises a Simulation Host PC, a Real-time Target PC, a Test PC and a two channel MTS 493 Real-time Controller. The Structural Test System (STS) software provides the user interface for PID (Proportional Integral Derivative) control and calibration of the actuator. For this study, the actuator was installed within two parallel internal reaction frames, as shown in Figure 8-2 and 8-3.

The test system has previously been used to carry out experimental testing on CBF structures (McCrum, 2012, Hunt, 2013). The beam and column specimens used in those tests were reused in this testing program.

8.2.2 Test Structure Design

The experimental substructure used for hybrid simulation in this test program is shown in Figure 8-1. It can be seen that this consists of a beam (305x165x54 UB) and column (2 No. 203x203x46) assembly, which in a full structure would primarily resist vertical gravity loads, along with one bracing member (40x40x4 SHS) which provides lateral stiffness. Gusset plates connect the bracing member to the beam and column. So called 'stub' sections (2 No. 203x203x60 UC) were used at the base of the column to approximate pinned supports.

The experimental structure was mounted on horizontal members spanning between the two reaction frames. Roller guides were mounted on the reaction frames, as shown in Figures 8-2 and 8-3, to prevent the structure from displacing laterally out-of-plane.

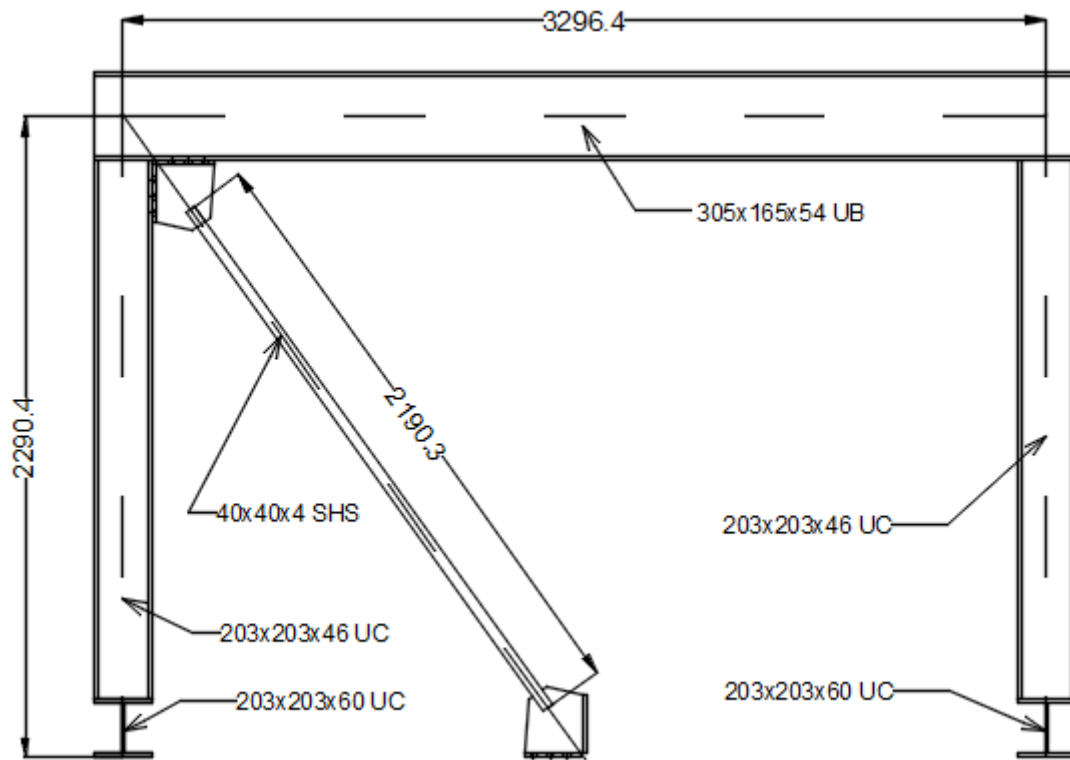


Figure 8-1 Experimental substructure

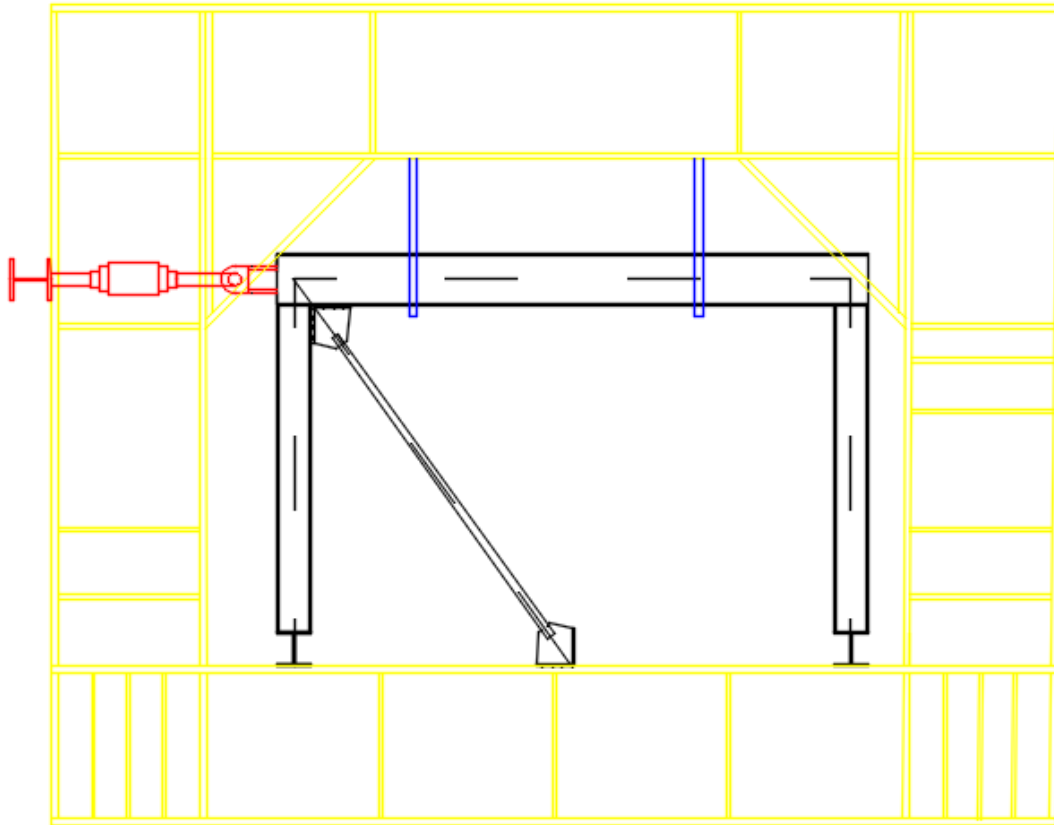


Figure 8-2 Schematic of test setup showing experimental substructure in black, reaction frames in yellow, actuator in red and roller guides in blue

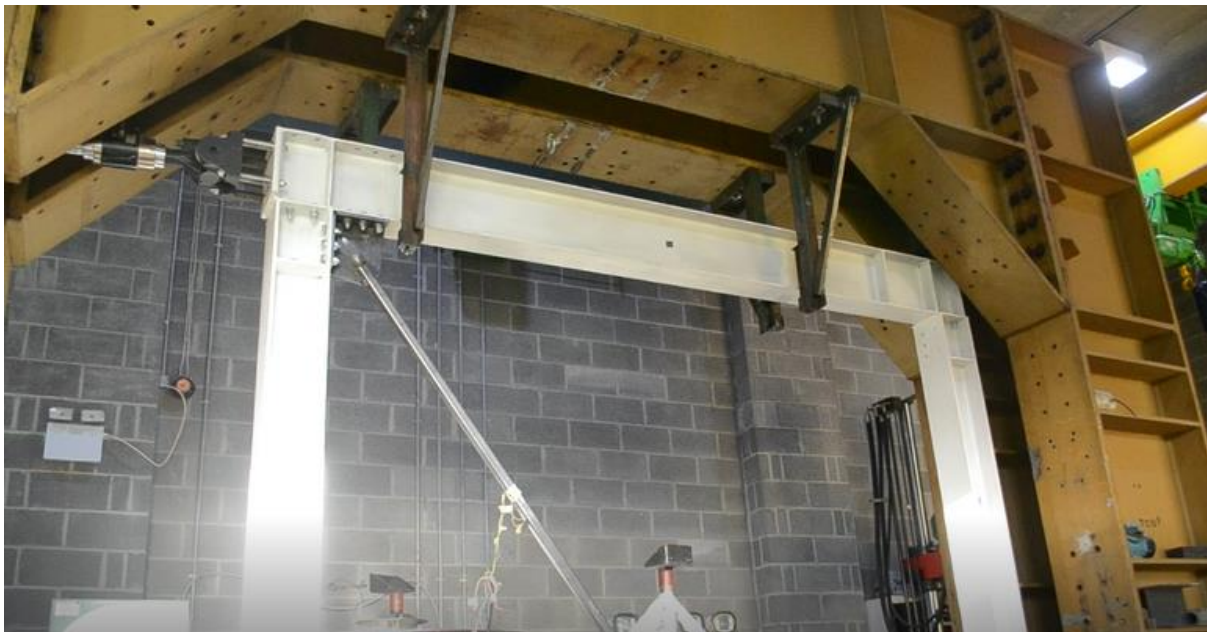


Figure 8-3 Photograph of the test setup at the TCD structure laboratory, showing experimental substructure, parallel reaction frames, roller guides and actuator

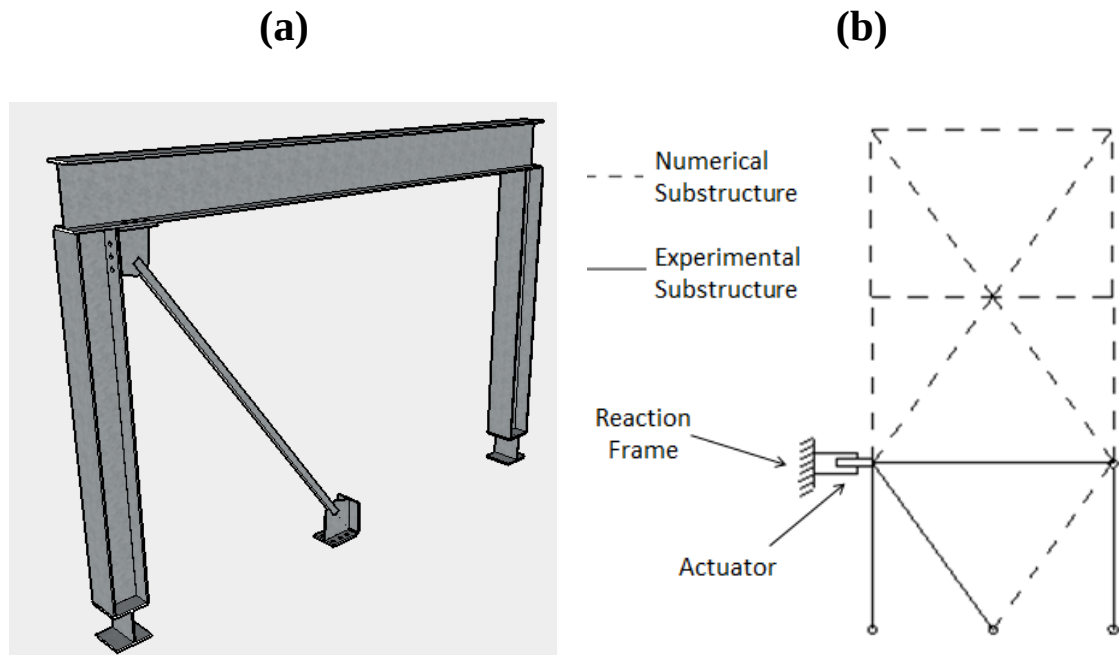


Figure 8-4 (a) Isometric view of the experimental substructure and (b) Experimental and numerical substructures

8.2.2.1 Brace and Beam and Column Design

This experimental test structure forms part of a larger three storey CBF model, in which the experimental substructure represents the ground storey as illustrated in Figure 8-4(b). The entire frame has been designed to Eurocode 8. The storey heights and bays width of the experimental substructure were dictated by the size of the reaction frames and the lengths of the beam and column sections. It was decided to keep the same frame geometry for the numerical storeys as for the experimental substructure. The guiding principles for the remainder of the frame design were as follows:

- 1) Given the force and displacement capacity of the actuator, $\pm 125\text{mm}$ and 150 kN respectively, it should be possible to reach the ultimate resistance of in the experimental bracing member. Given the frame geometry the maximum axial force that can be imposed on the brace is approximately 250kN .
- 2) Given the actuator capacities it should also be possible to reach the design deformation capacity of the experimental bracing member, calculated by Eurocode 8 as the product of the yield displacement and the design behaviour factor.
- 3) The experimental brace should obey the Eurocode 8 non-dimensional slenderness limitations; $1.3 \leq \bar{\lambda} \leq 2$.

- 4) The bracing members in the numerical substructure should obey the Eurocode 8 brace slenderness limitations; $1.3 \leq \bar{\lambda} \leq 2$.
- 5) The bracing members in both the numerical and physical substructures should obey the Eurocode 8 overstrength limitations; $\Omega_{\max} - \Omega_{\min} < 25\%$.
- 6) The bracing members in both the numerical and physical substructures should obey the Eurocode 8 design criteria; $N_{pl,Rd} > N_{Ed}$
- 7) The beams, columns and connections in both the numerical and physical substructures should obey Eurocode 8 capacity design criteria
- 8) The first brace to yield should be in the physical substructure

Principles 6) and 7) in the above list are controlled by the seismic load imposed on the structure. This is in turn controlled by the seismic mass, the design spectral acceleration, which is a function of the design ground acceleration, the soil type and the behaviour factor employed. In the design of a real world structure these parameters, possibly with the exception of the behaviour factor, are pre-defined and cannot be controlled by the designer. However, in this experimental scenario they can be varied so that a brace design which fits criteria 1) to 5) also meets seismic capacity design requirements. Hence, in essence the design of the experimental structure was a reversal of real-world design procedure.

Structural analysis, and subsequently design, was performed by assuming fully pinned connections, with lateral resistance assumed to be provided by the tension members only. A 40x40x4 SHS section fabricated from S235 steel was chosen as the bracing member for the experimental substructure. Table 8-1 shows how this fulfils requirements 1 to 3 outline above. Only one bracing member is included in the experimental setup. The actuator does not have sufficient force capacity to impose large displacements on a frame with two braces that are stocky enough to meet the slenderness criteria given the frame geometry. Testing only one brace also simplifies the interpretation of the experimental measurements and supports later model validation and updating.

Table 8-1 Design of the experimental substructure

Experimental Design Targets	Chosen	Required	Governing
Tensile Yield Force (kN)	76.73	<150	Actuator Force Capacity
Ultimate Force Capacity (kN)	117.54	<150	Actuator Force Capacity
Design Displacement ($q \times \delta_{yield}$) (mm)	6.45	<125	0.5 x Actuator Stroke
Brace Slenderness	1.81	$1.3 \leq \lambda \leq 2$	EC8 Limits

Table 8-2 demonstrates how all the bracing members in the test structure were selected to fulfil requirements 4) to 8) in the list above. The braces first numerical level are 40x40x4 SHS sections fabricated from S235 steel, while a S235 40x40x2 SHS section is used for the top level. This combination of material strength and section sizes allows the Eurocode 8 slenderness and overstrength criteria to be fulfilled. It can be seen from Table 8-2 that sections sizes have been chosen so that brace overstrength is much greater in the numerical levels; this is done in an attempt to ensure the brace in the experimental substructure experiences initial yield.

Table 8-2 Design details of test structure

Floor Level	Seismic Mass (Mg)	Brace Section	Overstrength (Ω_i)	Slenderness (λ)	Beam	Column
3	9	40x40x2	1.202	1.71	UB 305x165x54	UC 203x203x46
2	11.25	40x40x4	1.247	1.81	UB 305x165x54	UC 203x203x46
1	11.25	40x40x4	1.016	1.81	UB 305x165x54	UC 203x203x46

For the test program, it was decided to use the LA Bulk Mail site as a case study site. This site was also used as a case study site in the BRACED test program (SERIES, 2013, Salawdeh et al., 2017). The ground acceleration with a 10% in 50 year probability of exceedance on type A ground at the site, $0.4g$, was obtained from the USGS online tools (United States Geological Survey, 2017) and was used as the design ground acceleration value, a_g , in the reverse capacity design process. Type C ground was assumed for this process.

Using this design ground acceleration value and the previously selected bracing members, a reverse capacity design procedure was performed to select the seismic mass. This was done using an elastic analysis in which the design seismic base shear was distributed as horizontal forces at each floor level over the height of the structure based on the mass and height of each level; i.e. using the Eurocode 8 lateral force method of analysis for regular structures. The design seismic load was calculated using a behaviour factor of $q = 4$. In this analysis, all connections were assumed to be fully pinned and lateral resistance was assumed to be provided by the tension diagonals only. From this process, the masses at each floor level given in Table 8-2 were selected as they gave overstrength values slightly above 1 for the bracing members. Checks

were then performed on the beams and columns to ensure they met the axial load requirements for the combination of gravity and seismic loads stipulated by the code and to ensure dissipative behaviour is limited to the bracing members.

The bracing configuration of the numerical substructure is slightly unusual; generally for a structure with a V-braced frame, like the experimental substructure at the bottom level, the configuration of the bottom storey would be repeated at each level. However, here a split level X configuration was chosen as this avoids placing an unbalanced load at the centre of the experimental beam after the compression brace buckles in the level above. Such a load would be difficult to impose on the experimental substructure. Hence the chosen bracing configuration allows the experimental substructure to better capture the boundary conditions of the system. Representing the boundary conditions accurately has been identified (McCrum and Williams, 2016) as an important element in successful sub-structured hybrid tests.

8.2.2.2 Gusset Plate & Connection Design

The gusset plates and connections were designed to ensure yielding is limited to the bracing members. As can be seen from Figure 8-5, the design action for the gusset plates was calculated as the greater of the ultimate plastic axial resistance of the brace multiplied by a partial factor of 1.25 and the value stipulated by EC8 for non-dissipative members.

Brace			
Depth	d	40	mm
Breath	b	40	mm
Thickness	t	4	mm
Brace Capacity Calculation			
Brace Area	A	559	mm ²
Brace Yield Strength	$f_{ult, Brace}$	360	N/mm ²
Brace Axial Capacity	$N_{Pl, Rd, Brace}$	201.24	kN
			$A \times f_{y, Brace}$
Overstrength Factor	γ_{ov}	1.25	
Maximum Tension Capacity	$\gamma_{ov} N_{Pl, Rd, Brace}$	251.55	kN
			$\gamma_{ov} \times N_{Pl, Rd, Brace}$
EC8 Cl. 6.7.4 Design Action for Non-Dissipative members			
Seismic Action	$E_{d, E}$	129.2859	kN
Overstrength Factor	γ_{ov}	1.25	
Minimum Brace Overstrength	Ω	1.066886	
Capacity Design Seismic Action	E_d	189.6582	kN
			$1.1 \gamma_{ov} \Omega E_{d, E}$
Minimum Required Gusset Capacity		251.55	kN
			$\max(\gamma_{ov} N_{Pl, Rd, Brace}, 1.1 \gamma_{ov} \Omega E_{d, E})$

Figure 8-5 Gusset plate design action calculation

The gusset plates were then designed using a standard linear clearance approach. The Whitmore (Whitmore, 1952) section concept was used to calculate the effective area of the gusset plate for

tensile resistance. Dimensions of the plates were selected to have sufficient axial capacity in tension and buckling capacity in compression. Buckling resistance was calculated using the method proposed by Thornton (1991), which involves treating the gusset plate as a strut of length equal to the average of L_1 , L_2 and L_3 as shown in Figure 8-9(b). A conservative effective length factor of $K = 1.2$ was adopted in line with the reasoning proposed by Hunt (2013) and shown in Figure 8-6. This value assumes fixed rotation and free translation at the brace end of the gusset plate to account for the large out of plane deformation that may occur under high drift demands. As a result of this designs process 12mm gusset plates, with dimensions as shown in Figure 8-9(a), were selected for use at both ends of the brace in all experimental tests.

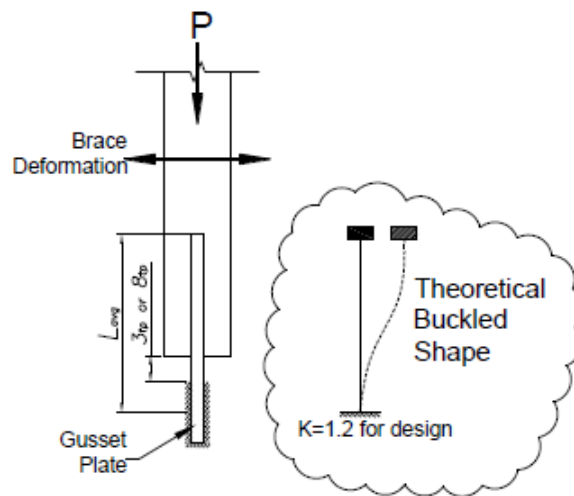


Figure 8-6 Theoretical buckled shape of gusset plate and effective length factor of $K = 1.2$, as proposed by Hunt (2013)

Tensile Capacity				
Plate Thickness	t_p	12	mm	
Weld Length		65	mm	
Brace Width	b	40	mm	
Whitmore Width	b_{ww}	116.9	mm	
Gusset Yield Strength	$f_{y,Gusset}$	235	N/mm^2	
Gusset Capacity	$N_{pl,Rd, Gusset}$	329.658	kN	$f_{y,Gusset} \times b_{ww} \times t_p$

Figure 8-7 Design of gusset plate for tensile resistance

Thornton Method Buckling Check				
L_1		159.4	mm	
L_2		201.4	mm	
L_3		130.2	mm	
L_{ave}		163.6667	mm	$(L_1+L_2+L_3)/3$
Effective Area	A_{ww}	1402.8	mm ²	$b_{ww} t_p$
Euler Buckling Load				
Effective Length	K	1.2		Thornton 0.65, Hunt 1.2
Effective Length	L_{crit}	196.4	mm	$K*L_{ave}$
Second Moment of Area	I_z	16833.6	mm ⁴	$b_{ww} t_p^3 / 12$
Elastic Modulus	E	210000	N/mm ²	
Euler Buckling Load	N_{Cr}	904.5094	kN	$\pi^2 EI / L_{crit}^2$
EC3 Buckling Load				
Radius of Gyration	i	3.464102	mm	$(I/A_{ww})^{0.5}$
	λ	56.6958		L_{crit}/i
	λ_1	93.9		93.9E
Non-Dimensional Slenderness	$\bar{\lambda}$	0.603789		λ/λ_1
Imperfection Factor	α	0.49		Buckling Curve C
	φ	0.781209		6.3.1.2 EC3
Reduction Factor	χ	0.783136		6.3.1.2 EC3
Buckling Resistance	$N_{b,Rd}$	258.1672	kN	$\chi \times f_{y,Gusset} \times b_{ww} \times t_p$

Figure 8-8 Gusset plate design for buckling resistance

The welds between the brace and the gusset and the between columns were designed according to Eurocode 3 Part 1-8. The gusset plates were welded to reusable flange plates that were then bolted to the beams and columns, as illustrated in Figure 8-10. This allowed a quick changeover of brace specimens between tests, as the gusset and brace could be prepared outside the frame while tests on other specimens were ongoing and then bolted in place once needed. The welds between the gusset plates and these end plates, and the bolted connection between the plates and the beam and column, were also designed according the EC3. Details of each of these design checks, and further drawings of specimens, are given in Appendix.

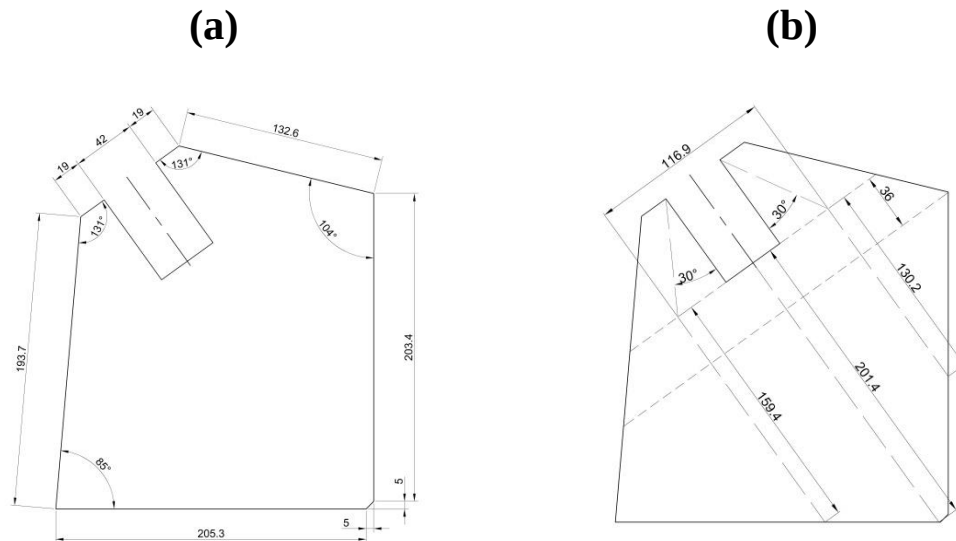


Figure 8-9 (a) Gusset plate dimensions, (b) gusset plate showing yield line, Whitmore width and dimensions for Thornton buckling model

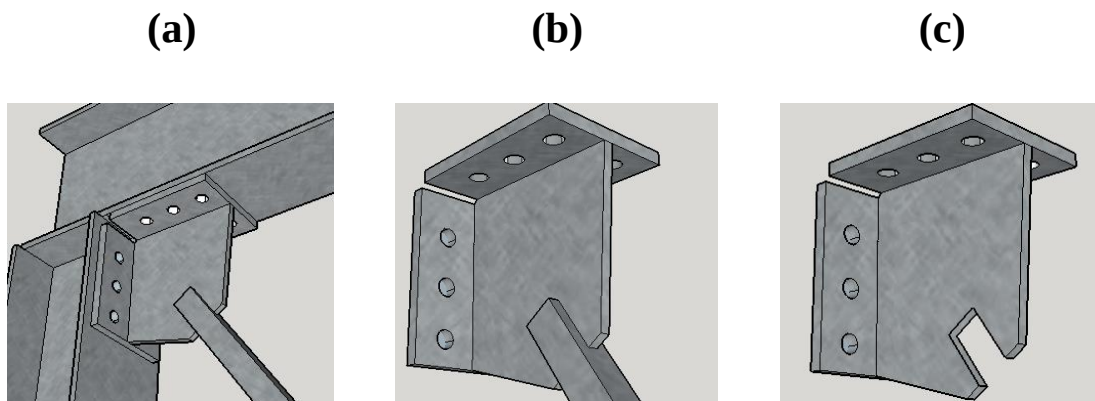


Figure 8-10 Gusset and end plate assembly for efficient interchange of brace samples

8.2.3 Instrumentation

The internal load cell and LVDT in the actuator record the force and displacement. The data is fed back from the actuator to the Test PC at the Scramnet frequency of 1/1024s. Data recorded on the Host PC consists of displacement and force feedback from the STS PC at larger time steps equal to the integration step for the particular earthquake record e.g. typically 0.01s.

The displacement imposed on the experimental substructure is measured by an internal LVDT (Linear Variable Differential Transformer) in the actuator. Likewise, the resisting force is measured by the actuator using an internal load cell.

8.3 Material Testing

All steel used in design was specified as S235. Material coupon test were carried out to determine the actual yield strength, ultimate strength and elastic modulus of the bracing

members. Six coupons were machined from untested 40x40x4 SHS sections. These tests were carried out in accordance with the European Standard IS EN ISO 6892-1:201 (ISO, 2016); Metallic materials - Tensile testing -Part 1: Method of test at room temperature

From the testing process a mean yield strength of 294.25 N/mm², with a standard deviation of 10.8 N/mm², was obtained. This mean value was used as the yield strength of the material in all of the finite element models.

8.4 Hybrid Simulation Setup at TCD

In hybrid simulation a finite element model runs concurrently with the experimental analysis. The experimental work described here has been performed using OpenSees 2.5.2 and OpenFresco 2.7 as the computational driver and middleware respectively. The experiment is controlled by the PID servo-control system in the laboratory. An intermediate Predictor-Corrector algorithm was implemented in Simulink to synchronise the computational driver and the control system, which perform their tasks at different timescales. Hence, a so-called three loop hardware architecture is employed in the tests, as illustrated in Figure 8-11. Each of these tasks is run on a separate processor, with data transferred via the ScramNET GT150 reflective shared memory.

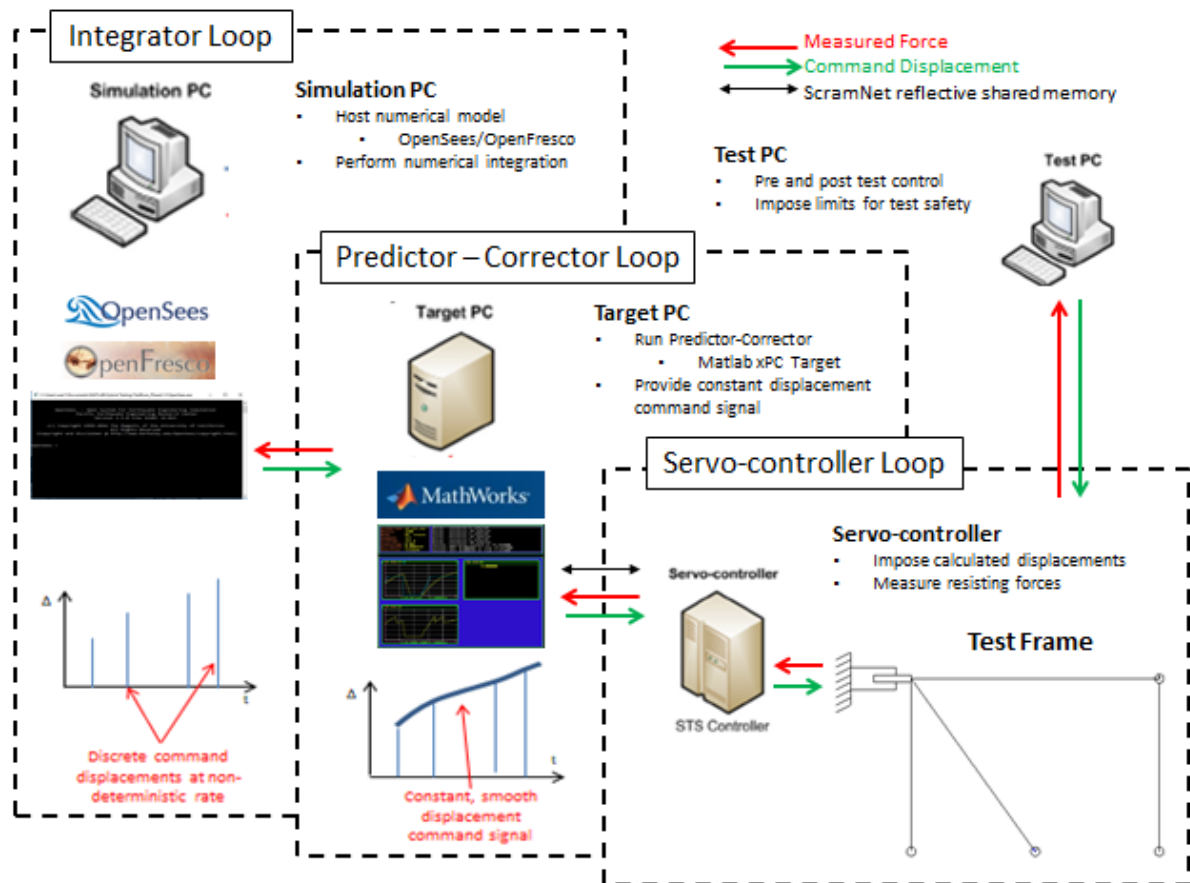


Figure 8-11 Schematic of three-loop hardware architecture at TCD Structures Laboratory

8.5 Finite Element Model

8.5.1 Numerical Substructure

The numerical substructure is modelled in OpenSees. As with the numerical models used in previous chapters, the bracing elements were modelled following the recommendations of Uriz et al. (2008) using two force-based elements, an initial camber displacement of 0.1% of the brace length at the midpoint and three integration points with the recommended cross section discretization. Again as before, the gusset plate model proposed by Hsiao et al. (2012) using nonlinear rotational spring elements to simulate the end restraint imposed on the braces was employed, while the Steel02 material was used for all nonlinear components.

The beams and columns were modelled as elastic elements to reduce the computational burden, after a preliminary investigation confirmed the validity of this simplification. Entirely numerical analyses were performed firstly using elastic elements, and then with distributed plasticity elements, to model the beams and columns. This analysis was carried out using five ground motion records scaled to an intensity level with a 2% probability of exceedance in 50 years for the LA Bulk Mail site, the selection of which is discussed in the next Chapter and which are the

strongest to be employed in the hybrid simulation program. No significant difference was found between these modelling approaches, as illustrated in Figure 8-12 for one of the ground motions. Table 8-3 shows that for the *EDPs* of interest for performance assessment, namely peak inter-storey drift and peak floor acceleration, the difference between the two modelling approaches is less than 1% for all the cases examined.

Table 8-3 Impact of using elastic elements compared to distributed plasticity elements for modelling beams and columns of the test structure

Ground Motion	1	2	3	4	5
Elastic Beams & Columns					
Peak Inter-storey Drift (%)	1.300	1.177	1.205	0.544	0.696
Peak Floor Acceleration (g)	1.588	1.712	1.435	1.346	1.436
Distributed Plasticity Beams and Columns (7 integration points per element)					
Peak Inter-storey Drift (%)	1.300	1.186	1.206	0.544	0.695
Peak Floor Acceleration (g)	1.586	1.716	1.436	1.345	1.436
Percentage Difference					
Peak Inter-storey Drift	0.03%	0.76%	0.07%	-0.05%	-0.07%
Peak Floor Acceleration	-0.12%	0.25%	0.07%	-0.08%	-0.01%

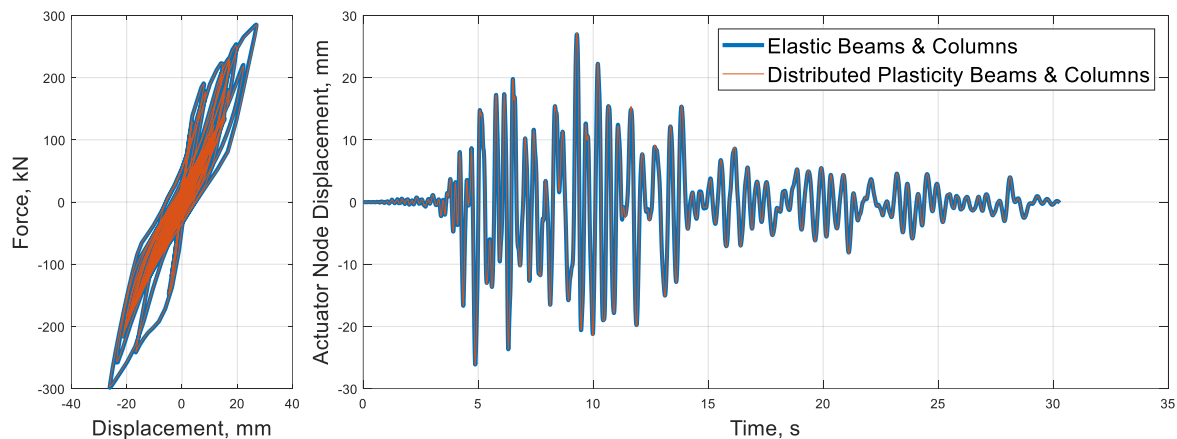


Figure 8-12 Influence of using elastic or distributed plasticity beams & columns in finite element model on response

8.5.2 Integration Algorithm

Numerical integration methods are used to solve the equations of motion at each time-step during hybrid simulation. The use of the Operator-Splitting and Generalized Operator-Splitting (the *alphaOS* and *alphaOSGeneralized* objects in *OpenFresco*) integration algorithms were investigated. As these algorithms are explicit, there is no need to iterate within a calculation step, and hence the integration can be performed quickly. In preliminary hybrid simulations these were found to work well for low intensity level earthquakes, allowing for quicker tests than the implicit Newmark algorithm. However, when buckling occurred in the numerical substructure the system tended to become unstable and give excessively large displacements.

Instability at geometric nonlinearities is a known weakness of these alphaOS algorithms (Schellenberg et al., 2009c). It is possible that faster, potentially real time, hybrid simulations could be performed in the future using operator splitting integration methods in OpenSees and OpenFresco for Moment Resisting Frame type structures, where there is no buckling and minimal geometric nonlinearities.

The use of other conditionally stable explicit integration algorithms is not appropriate for the test structure as the relatively large number of degrees of freedom, resulting from the number of nodes required to model the connections and brace members, leads to a large highest natural frequency and consequently an impractically small maximum timestep. Given this, and the unsuitability of the operator-splitter methods, it was decided to use an implicit Newmark algorithm specially developed for hybrid simulation with a fixed number of iterations at each timestep, for all tests. This is the same as the algorithm that has been used in previous tests at TCD (McCrum, 2012), and is very suitable for slow hybrid testing (Mosalam, 2013).

Prior to testing, the number of iterations necessary at each timestep was investigated using numerical modelling. As before, time history analysis of the test structure was carried out using a number of different ground motion records scaled to represent earthquake intensity with probabilities of exceedance of 2% in 50 years at the LA Bulk Mail site, as detailed in Chapter 9. Firstly a benchmark analysis was performed using the regular implicit Newmark algorithm with the number of iterations at each step depending on a displacement increment convergence test, as in a conventional numerical analysis. Analysis was then repeated using the implicit Newmark algorithm with the number of iterations fixed at 2, 5 and 10. The key *EDPs* of peak inter-storey drift and peak floor accelerations obtained from this process are given in Table 8-4. In general, the peak drift and acceleration response are similar for each analysis runs and there is little benefit to increasing the number of iterations. However, in some cases when only 2 iterations are employed the analysis becomes unstable. This can be seen in the example shown in Figure 8-13, where there is no visible difference between the results for different number of iterations, with the exception of the 2 iterations per step run becoming unstable. As a result of this process it was decided to set the number of iterations at each timestep to 5 for hybrid simulation.

Table 8-4 Influence of the number of iterations at each time step on the Peak Inter-storey Drift and Peak Floor Acceleration

Ground Motion	1	2	3	4	5
Peak Inter-storey Drift (%)					
Convergence Test	1.300	1.177	1.205	0.544	0.696
2 Iterations	Unstable	Unstable	Unstable	0.544	0.696
5 Iterations	1.299	1.180	1.199	0.544	0.696
10 Iterations	1.300	1.177	1.205	0.544	0.696
Peak Floor Acceleration (g)					
Convergence Test	1.588	1.712	1.435	1.346	1.436
2 Iterations	Unstable	Unstable	Unstable	1.346	1.437
5 Iterations	1.588	1.707	1.457	1.346	1.436
10 Iterations	1.588	1.725	1.434	1.346	1.436

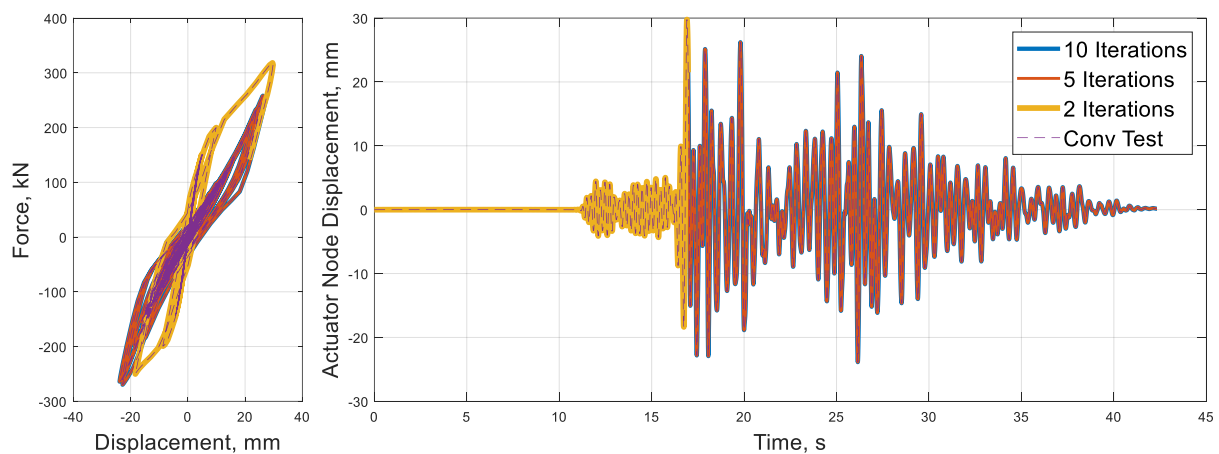


Figure 8-13 Influence of number of iterations at each time step in the integration algorithm on response, compared to a regular Newmark algorithm where the number of iterations is decided by a convergence test based on the displacement increment at iteration.

8.5.3 Representation of Experimental Substructure

One challenge in developing an OpenSees – OpenFresco model for hybrid simulation is to correctly represent the experimental substructure within the finite element model. When a single element is tested using one actuator this is relatively straight forward. However, for a more complex experimental substructure, like the one under examination here, tested using only one actuator, this becomes challenging.

It would be possible to avoid this problem by testing a single element, most probably the bracing member. However to fully understand the frame response, it is necessary to test the entire brace, gusset plate, beam and column assembly, i.e. the full range of structural components providing seismic resistance at a storey level.

The test setup illustrated in Figure 8-1 behaves like a single degree of freedom system under seismic excitation and therefore it is valid to test it using only a single actuator. This means that

the displacement command and force feedback are only available for a single degree of freedom, i.e. the degree of freedom connected to the actuator. However, because the numerical substructure is connected to different nodes of the experimental substructure, the experimental system cannot simply be modelled as a single degree of freedom system in OpenSees. For example, using the numbering convention set out in Figure 8-15, the numerical model is connected to the experimental model at degrees of freedom 7, 8, 9, 10, 11, 12 and 15. Hence, in order to model the numerical substructure correctly, the experimental substructure needs to be represented in the finite element model in such a way that it incorporates each of these interface degrees of freedom.

In the tests performed by McCrum (2012) the experimental substructure was represented in OpenFresco by two beamColumn experimental elements. OpenFresco was programmed to view the experimental setup as two separate tests, each consisting of a fixed-base column connected to an actuator, as illustrated in Figure 8-14. However, only one of these assemblies corresponded to the actual physical experiment; the other actuator-column setup was entirely fictitious with the purpose of representing degrees of freedom not connected to the actuator in the finite element model. The response of this fictitious test was simulated using the SimUniaxialMaterials experimental control object in OpenFresco. The measured stiffness of the column–actuator assembly that represented the real test structure was set to equal to 99% of the calculated lateral stiffness of the whole experimental substructure. The displacement calculated here was imposed on the experimental substructure at each timestep and the resisting force measured and fed back for use in the solution to the equations of motion for the following step. The simulated experiment provided only 1% of the lateral stiffness of the experimental substructure, in order to limit its influence on the overall outcome. A numerical beam element joined the real and simulated experimental columns to ensure they moved together horizontally, as the two degrees of freedom would in the experimental test. Both of the experimental elements had an axial stiffness equal to that of the columns used in the test. Thus the setup provided realistic vertical and horizontal displacements at the degrees of freedom interfacing with the numerical substructure.

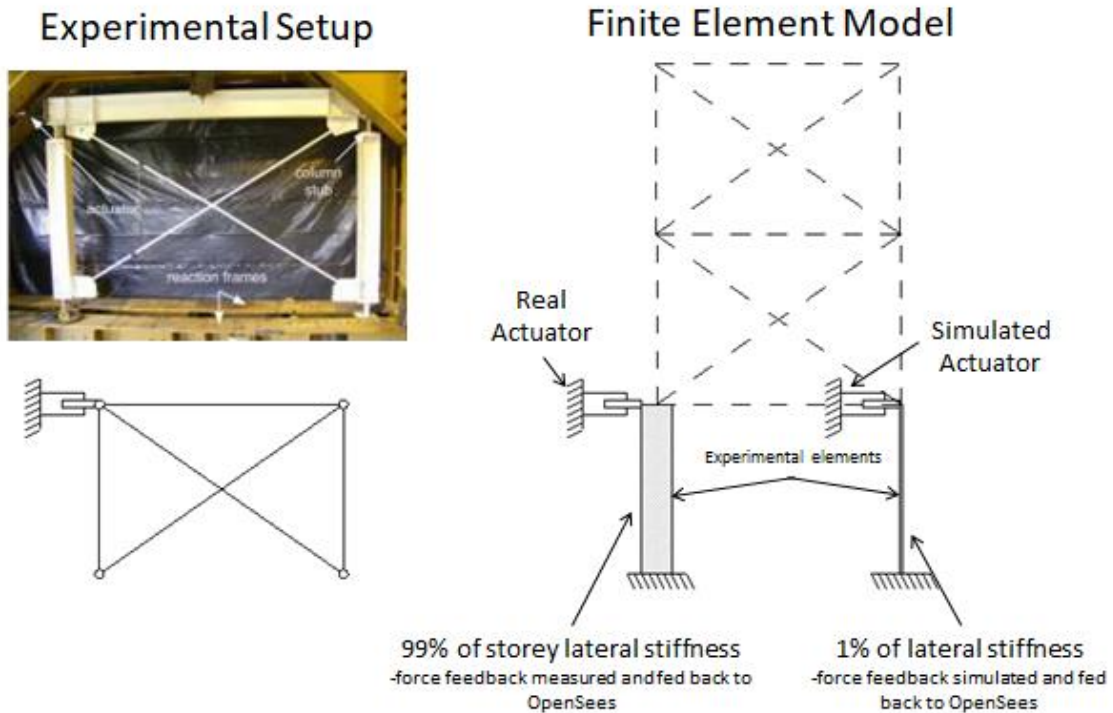


Figure 8-14 Experimental setup and finite element model from (McCrum, 2012); the experimental element is represented by two fixed-base columns, one of which is simulated

While this simulated experiment approach was found to work reasonably well in terms of accuracy, a new approach was adopted here to more faithfully represent of the test setup. The generic experimental element object in OpenFresco was used to model the experimental substructure. This method has been employed in previous research elsewhere to test experimental substitutes using only horizontal actuators, for example by Lin et al. (2012). In this approach a macro-element, which connects any number of nodes and has an associated stiffness matrix, is created to represent the experimental setup. How exactly the nodes are connected and the properties of the individual experimental elements are not specified; the assemblage of element stiffness matrices into a global matrix is performed outside OpenFresco. This global stiffness matrix for the macro experimental element is then added to the overall global stiffness matrix in the global finite element model representing the combined numerical and experimental substructures.

In order to develop the specific experimental substructure stiffness matrix required for these tests, the experimental setup was modelled as illustrated in Figure 8-15. With restrained degrees of freedom removed, the stiffness matrix for this macro-element, which was developed using basic finite element principles and assembled prior to being inputted into OpenFresco, , can be shown to be:

$$K = \begin{bmatrix} \frac{4EI_{col}}{L_{col}} & 0 & \frac{6EI_{col}}{L_{col}^2} & 0 & \frac{2EI_{col}}{L_{col}} & 0 & 0 & 0 & 0 \\ 0 & \frac{4EI_{col}}{L_{col}} & 0 & 0 & 0 & \frac{6EI_{col}}{L_{col}^2} & 0 & \frac{2EI_{col}}{L_{col}} & 0 \\ \frac{6EI_{col}}{L_{col}^2} & 0 & \frac{EA_{beam}}{L_{beam}} + \frac{12EI_{col}}{L_{col}^3} + \frac{EA_{brace}}{L_{brace}} \cos^2 \vartheta & \frac{-EA_{brace}}{L_{brace}} \cos \vartheta \sin \vartheta & \frac{6EI_{col}}{L_{col}^2} & -\frac{EA_{beam}}{L_{beam}} & 0 & 0 & 0 \\ 0 & 0 & \frac{-EA_{brace}}{L_{brace}} \cos \vartheta \sin \vartheta & \frac{EA_{col}}{L_{col}} + \frac{12EI_{beam}}{L_{beam}^3} + \frac{EA_{brace}}{L_{brace}} \sin^2 \vartheta & \frac{6EI_{beam}}{L_{beam}^2} & 0 & \frac{-12EI_{beam}}{L_{beam}^3} & \frac{6EI_{beam}}{L_{beam}^2} & 0 \\ \frac{2EI_{col}}{L_{col}} & 0 & \frac{6EI_{col}}{L_{col}^2} & \frac{6EI_{beam}}{L_{beam}^2} & \frac{4EI_{col}}{L_{col}} + \frac{4EI_{beam}}{L_{beam}} & 0 & \frac{6EI_{beam}}{L_{beam}^2} & \frac{2EI_{beam}}{L_{beam}} & 0 \\ 0 & \frac{6EI_{col}}{L_{col}^2} & -\frac{EA_{beam}}{L_{beam}} & 0 & 0 & \frac{EA_{beam}}{L_{beam}} + \frac{12EI_{col}}{L_{col}^3} & 0 & \frac{6EI_{col}}{L_{col}^2} & 0 \\ 0 & 0 & 0 & \frac{-12EI_{beam}}{L_{beam}^3} & \frac{6EI_{beam}}{L_{beam}^2} & 0 & \frac{EA_{col}}{L_{col}} + \frac{12EI_{beam}}{L_{beam}^3} & \frac{-6EI_{beam}}{L_{beam}^2} & 0 \\ 0 & \frac{2EI_{col}}{L_{col}} & 0 & \frac{6EI_{beam}}{L_{beam}^2} & \frac{2EI_{beam}}{L_{beam}} & \frac{6EI_{col}}{L_{col}^2} & \frac{-6EI_{beam}}{L_{beam}^2} & \frac{4EI_{col}}{L_{col}} + \frac{4EI_{beam}}{L_{beam}} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Where E is the modulus of elasticity of steel, I is the second moment of area of the member in question, A is the area of the element, L is the member length and ϑ is the angle between the brace member and the horizontal.

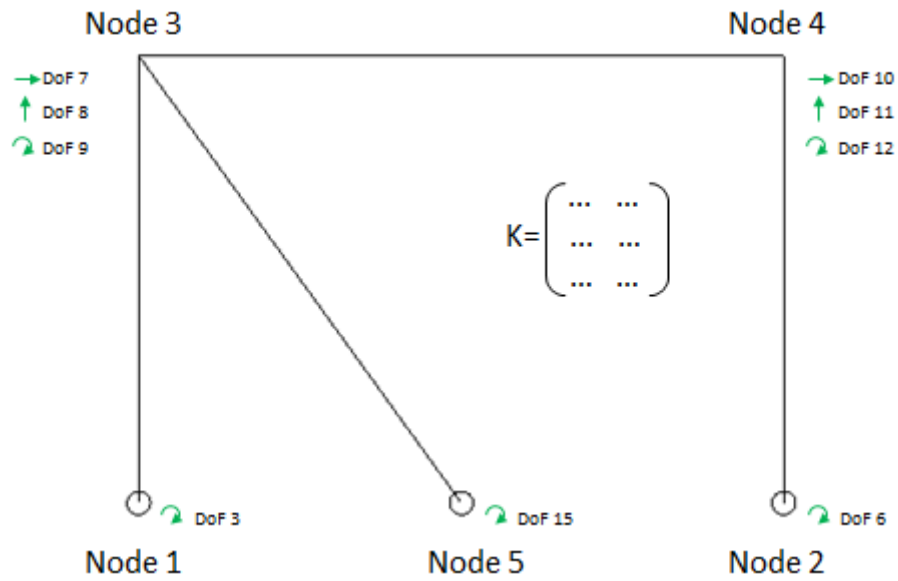


Figure 8-15 Model of the experimental substructure, with nodes and active degrees of freedom labelled, used to develop the macro-element stiffness matrix for hybrid simulation

Specifying the initial stiffness matrix of the experimental substructure correctly is important in hybrid simulation as typically it is not updated for nonlinear behaviour during the testing procedure (Schellenberg et al., 2009c). This is in contrast to numerical elements where tangent stiffness matrices are used at each calculation step.

Adopting this generic element approach allows the experimental substructure to be tested without the need for the extra simulated experimental elements employed in previous tests at

TCD. It also provides greater freedom to the modeller to adapt the rotational stiffness of the connections, although fully fixed and pinned connections were assumed here.

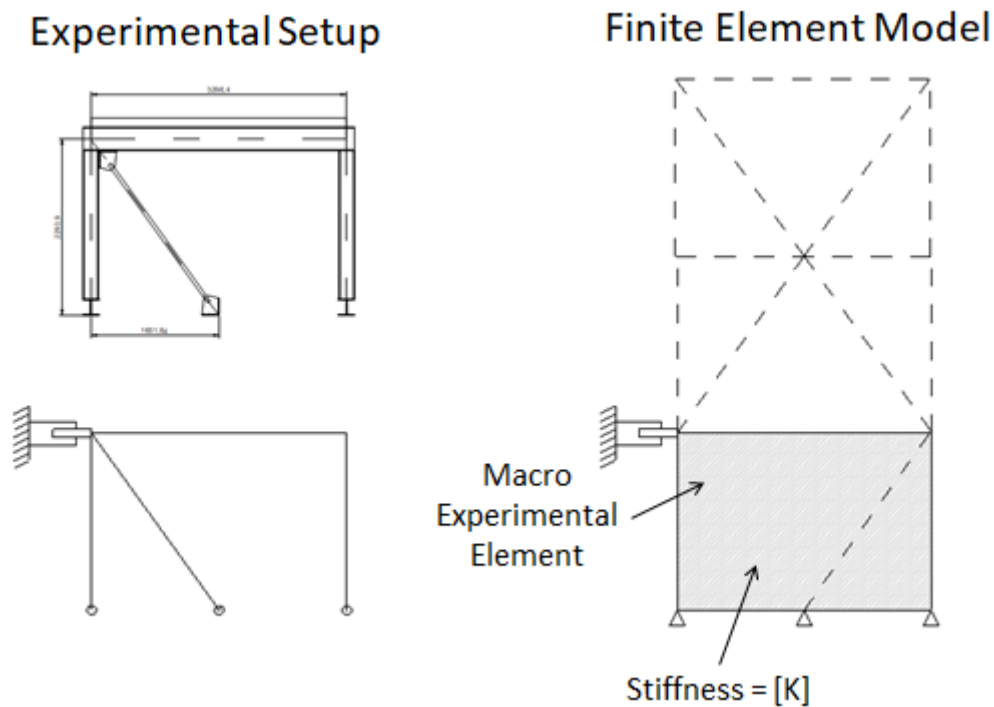


Figure 8-16 Experimental setup and finite element model adopted for this test; experimental substructure modelled as macro-element

8.5.4 Validation of Experimental Substructure Modelling

A fully numerical model of the test structure was developed in OpenSees. This model was similar to the finite element model employed in hybrid simulation, except both the experimental and numerical substructure were modelled numerically. Analysis was performed using a regular implicit Newmark integration algorithm.

Figure 8-17 compares the displacement time history response of the degree of freedom connected to the actuator (DOF 7 in Figure 8-15) calculated using the purely numerical model with the values recorded from experimental tests using the 99%-1% strategy and the macro element discussed above. It can be seen that the difference between the three is minimal, but the macro element arguably achieves a slightly closer match with the purely numerical model, particularly later on in the test.

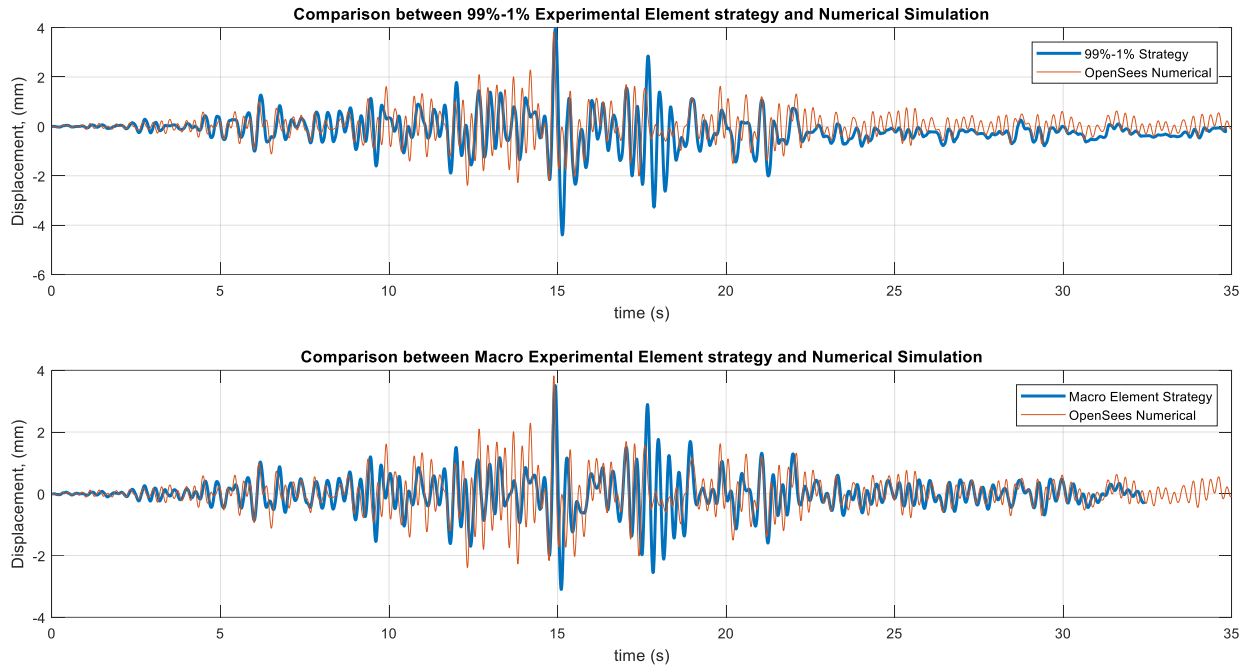


Figure 8-17 Comparison between actuator node displacement for purely numerical mode and experimental test run with 99% - 1% lateral stiffness distribution and generic macro-element approach (Taiwan40 ground motion record, scale factor = 10.0, PGA = 0.15g)

Figures 8-18 and 8-19 present further comparison between the purely numerical model and experimental tests performed using the macro-element strategy. A plot of first storey drift versus base shear is presented in Figure 8-18. In Figure 8-19 the displacement of the actuator node and the roof displacement of the numerical substructure are compared with their equivalents from the purely numerical model. In this test run the frame was found to behave almost entirely linearly and it can be seen that a very close match is achieved between the experimental and numerical results, particularly for displacement. This illustrates the suitability of the macro-element strategy for testing a complex experimental substructure using a single actuator. A more comprehensive comparison of experimental and numerical response simulations is presented in Chapter 9.

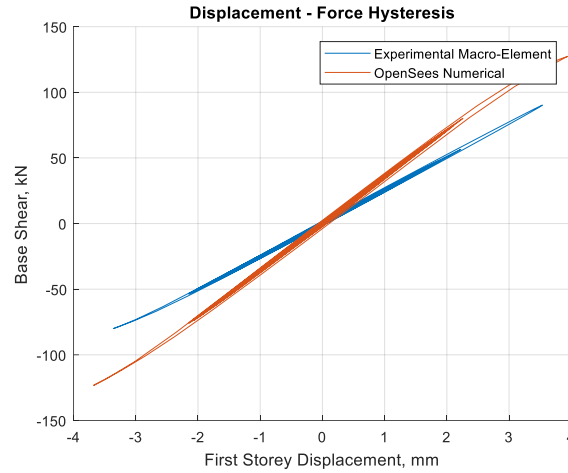


Figure 8-18 First storey displacement – base shear hysteresis comparison between purely numerical and experimental for a sample ground motion (Coalinga 01 (PEER ID 357) Ground Motion Record, Scale Factor = 1.24, PGA = 0.19)

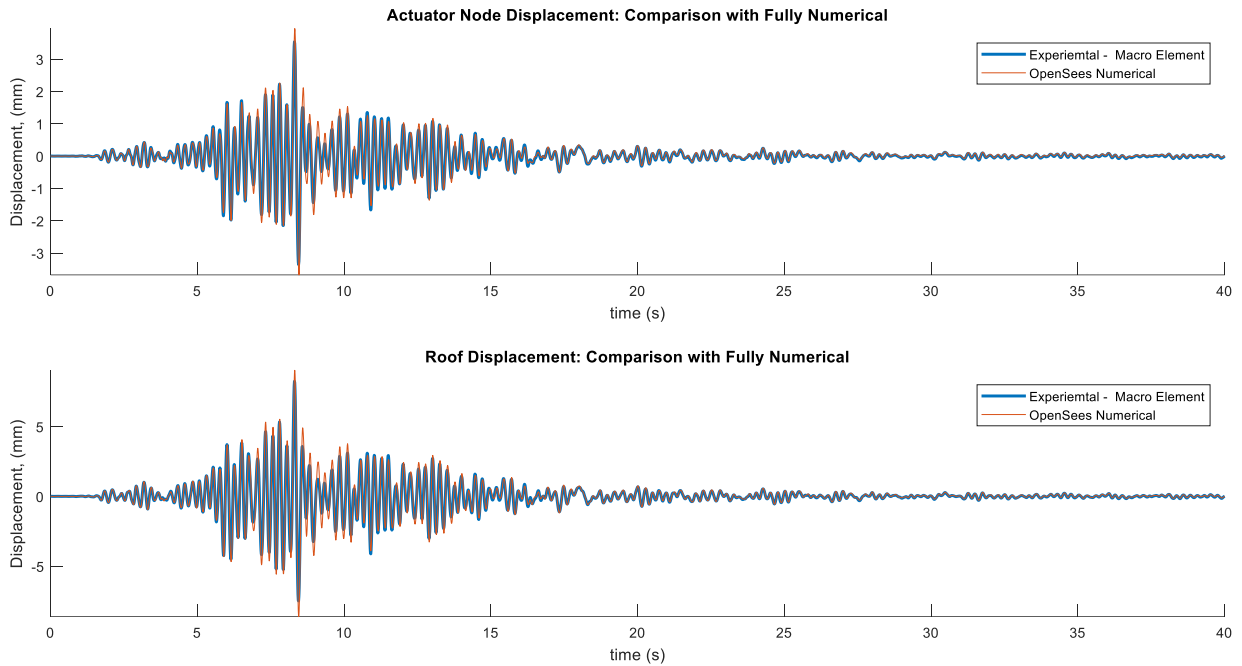


Figure 8-19 Comparison of actuator and roof displacement time histories from purely numerical OpenSees model and hybrid simulation using macro-experimental element for a sample ground motion (Coalinga 01 (PEER ID 357) Ground Motion Record, Scale Factor = 1.24, PGA = 0.19)

8.6 Preliminary Testing

Once an adequate finite element model was developed a series of preliminary hybrid simulations were performed. The purpose of these was to become familiar with the test system, to validate the finite element models and control strategies and to correct any other problems. During these tests a number of issues that required addressing arose. These are discussed in the following sections.

8.6.1 Random Long Delays

In previous tests at TCD long delays frequently occurred during the tests. For the hybrid test setup employed here using a Newmark integration algorithm with the number of iterations fixed at 5, OpenSees is capable of executing calculations approximately 6 timestep calculations per second, although this is not a constant number and varies with the available computational power, model complexity, strength of the ground motion and other factors. However, on occasions, timesteps took much longer to complete, either in the order of some minutes or on occasions did not complete at all. These delays were not repeatable; the same test could be run twice but would not experience the same delayed timesteps.

It should be noted that to the best of the authors' knowledge, such delays have not been reported before in literature. It is possible that the problems discussed and solutions presented below are local issues and only of relevance to the system at TCD. However, it is equally possible that the problem, or a variant thereof, is experienced in other hybrid testing facilities and therefore the following discussion of the causes and the solution adopted here is potentially valuable for future tests elsewhere.

8.6.1.1 Identification of the Problem Region

An extensive debugging program was undertaken to establish the cause of these delays using the OpenSees source code. A special debugging integrator object was developed to allow the exact point of the delays to be identified. The new transient integrator was developed from the existing NewmarkHSFixedNumIter integrator, with a series of 'print' commands added to allow the user to identify where exactly in the algorithm the delays were occurring. From this, it was shown that long delays occurred while OpenFresco attempted to read in the feedback signal from the test structure as the integrator attempted to execute the `formElementResidual` method for the experimental element.

Based on this knowledge, the force read-in method was further examined. The interface between OpenFresco and the Predictor-Corrector model is in the experimental control class in OpenFresco. For test setups using xPC Target, like the one in TCD, the measured resisting force and measured displacement are obtained in the 'acquire' method within the `ECxPCTarget` experimental control object. Pseudo-code for 'acquire' is as follows:

```

atTargetID = 0
while (atTargetID != 1)                                //Loop to check the value of the atTarget
signal
    (
        atTarget = xPCGetSignal (atTargetID)            //check the value of the atTarget
        signal
    )
    //Progress once atTarget == 1
MeasuredForce = xPCGetSignals(daqSignal)                //get the value of feedback signals

```

atTargetID is a signal in the Simulink Predictor-Corrector model that normally equals 0, but equals 1 when the target displacement is reached by the actuator. Looking at the pseudo code, it can be seen that the ‘acquire’ method waits indefinitely until the target displacement is reached, i.e. until atTargetID =1, and then reads in the measured resisting force. On occasion, it was noted that the atTargetID signal did not equal 1 when expected, thus causing the system to wait indefinitely, resulting in long delays. In order to explain why this is the case, and how the problem was solved, it is necessary to understand the workings of the Predictor-Corrector model. Therefore, a detailed explanation of the algorithm is provided in the following section.

8.6.1.2 Summary of the Simulink Predictor-Corrector Model

As illustrated in Figure 8-11, the Predictor–Corrector model is the middle piece of the three-loop hardware architecture and links OpenFresco and the structural test system. The model used here originates from the one provided in the OpenFresco 2.6 Examples Manual, which is based on the event driven strategy proposed by Mosqueda et al. (2005) and further refined by Schellenberg et al. (2009c). However it has been developed for compatibility with the local test system at TCD and for use with OpenFresco 2.7. The predictor-corrector model is built in Simulink using the xPCTarget environment, and runs on the so-called Target PC. Its role is to provide a constant command displacement signal to the actuator. OpenSees runs at a non-deterministic rate, while the test system runs at a constant rate of 1024 commands a second. Hence the role of the Predictor-Corrector model is to convert the irregular displacement commands from OpenSees into a constant, smooth command displacement signal for the actuator.

As the name suggests, this is done by both predicting and correcting the displacement command. At the start of a calculation timestep, before the command displacement has been received, the displacement is predicted based on the previous displacement commands. The actuator moves according to this prediction. Once the command displacement is received the actuator movement is corrected so that it reaches the calculated command displacement at the end of the step. This concept of predicting and correcting is illustrated in Figure 8-20.

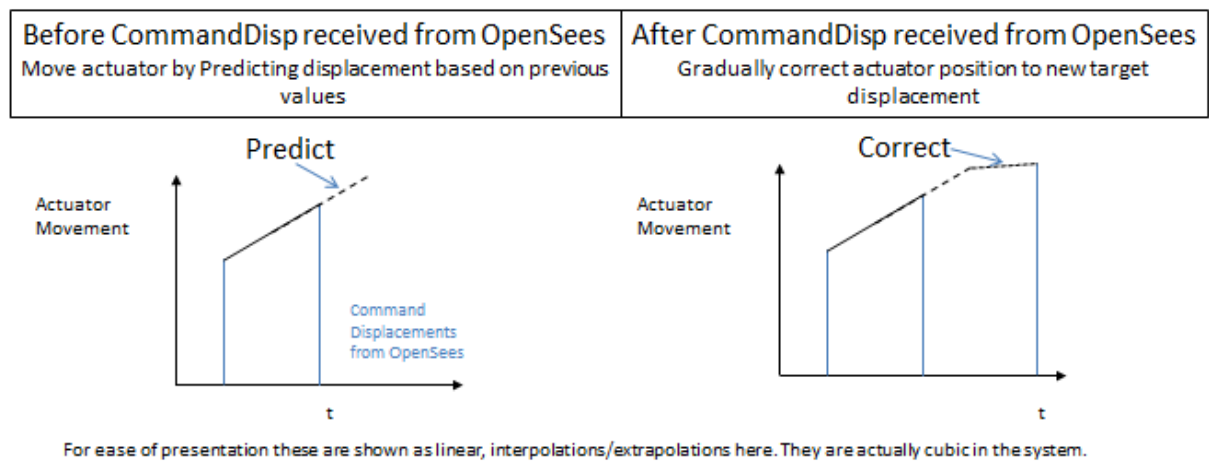


Figure 8-20 Illustration of the Predictor-Corrector model controlling actuator movement by predicting before receiving command displacement and correcting after receiving command displacement

8.6.1.2.1 Counter

These prediction and corrections work using a counter, denoted 'i' in the code. For this test setup, the counter was set to run between 1 and 20, normally in steps of 1. The position of the actuator within a timestep is determined by the value of the counter; it moves so that the displacement is equal to the command displacement when the counter equals 20. At intermediate values, the value of displacement signal sent to the actuator is determined by the value of the counter. For example, when the counter equals 10 and the model is still predicting (i.e. the command displacement for the step has not yet been received), the actuator will be half way (10/20) between the displacement at the end of the last step and the predicted displacement for the end of the present step. This concept is illustrated for a hypothetical example step in Figure 8-21.

- Example Time step, step 'i' (Assuming Linear interpolation/extrapolation for simplicity)
 - Say: $U_{i-2} = 8\text{mm}$; $U_{i-1} = 9\text{mm}$; hence predict $U_i = 10\text{mm}$

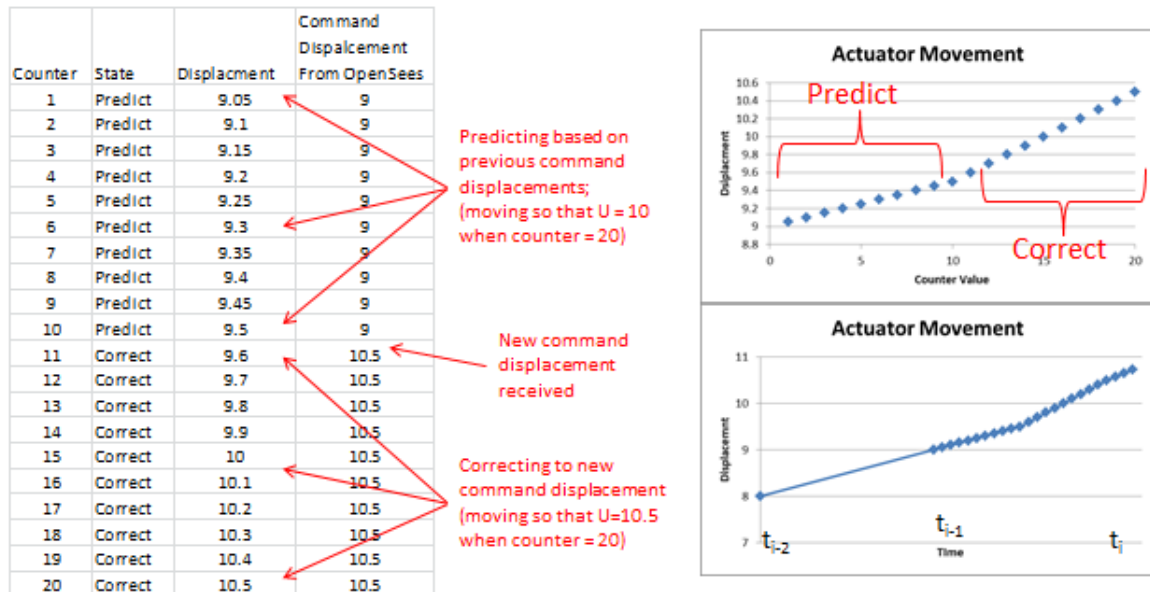


Figure 8-21 Example timestep, showing the workings of the counter within the Predictor-Corrector model

The predictor corrector also contains a 'slow-down' state for timesteps where the command displacement isn't received quickly enough. For these tests, if the command displacement has not been received by the time the counter reaches 12 the 'slow-down' state is triggered. In 'slow-down' the model continues to predict based on the previous displacements, but the rate of increase of the counter becomes continuously smaller and hence the actuator movement becomes gradually slower. This provides extra time for OpenSees to generate the next command displacement. When this value is received the model returns to 'Correcting' as normal.

8.6.1.2.2 atTargetID Signal

atTargetID is a Boolean signal within the Predictor-Corrector model that indicates whether the target displacement has been reached. Generally, the value of atTargetID is 0. However, when the counter reaches 20, and the displacement of the actuator is equal to the target displacement, the value of the atTargetID signal becomes 1. It stays at 1 until the next command displacement is received from OpenSees. This is illustrated in Figure 8-22.

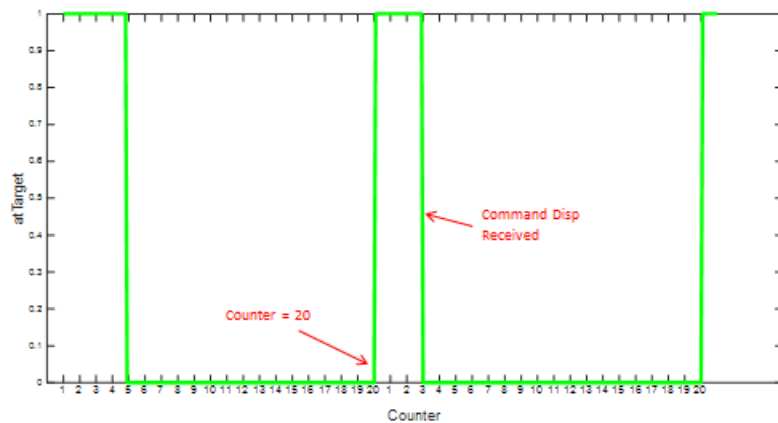


Figure 8-22 Variation of the atTargetID signal with the counter

As discussed, the atTargetID signal is monitored by the 'acquire' method within the ECxPCTarget class in OpenFresco. OpenFresco waits until atTargetID equals 1 before reading in force feedback. If the value of atTargetID remains at zero, OpenFresco will wait indefinitely.

8.6.1.2.3 Erratic Counter Misbehaviour

The problems that arose in the system at TCD were due to erratic counter misbehaviour. On occasion, the counter jumped and didn't equal 20 within a timestep. This means atTargetID didn't go to 1, and hence OpenFresco is delayed indefinitely in the 'acquire' method.

The explanation for this erratic counter behaviour is not known. Figure 8-23 shows an example of the counter behaving as intended; the counter reaches 20 at the end of each step. Two examples of observed erratic misbehaviour resulting in delays are shown in Figure 8-24. In these it can be seen that the counter fails to reach 20 at the end of a timestep. In Figure 8-24(a) the counter jumps from 19 to 21, while in Figure 8-24(b) it resets from 5 to 0. Both of these errors result in a long delay in the following step as OpenFresco waits to read in the force feedback.

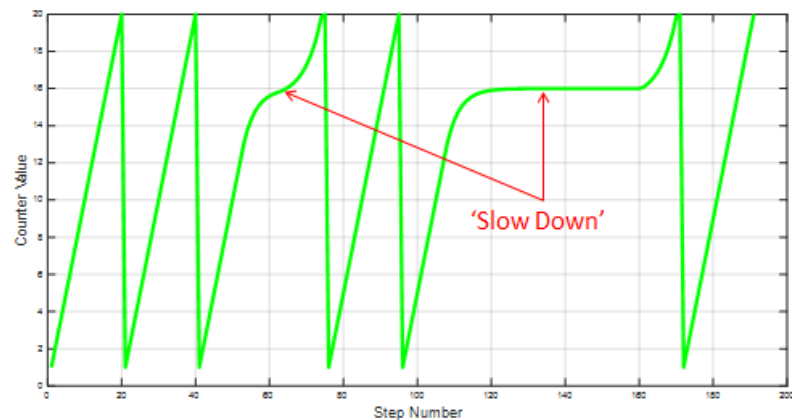


Figure 8-23 Example of correct counter behaviour, including some 'slow down' states

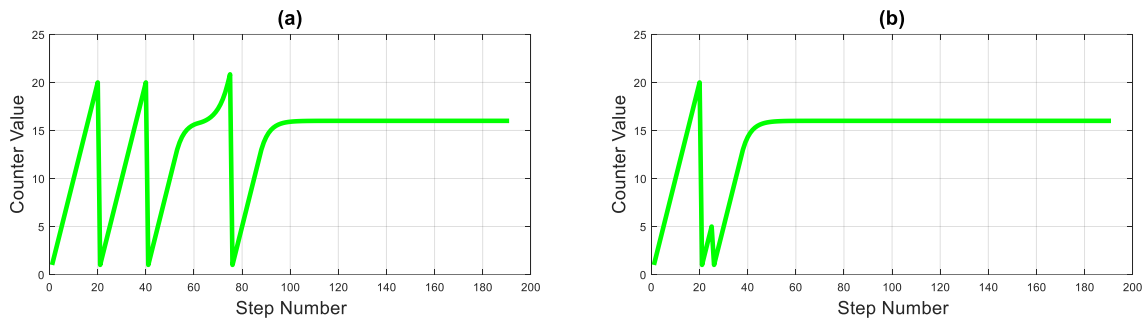


Figure 8-24 Examples of erratic counter misbehaviour; in both cases the counter fails to equal 20 within a timestep resulting in a delay at the next step due to (a) counter ‘overshoot’ and (b) counter ‘undershoot’

8.6.1.2.4 Solution

The ideal solution to the issue would have been to correct the issues causing the erratic misbehaviour of the counter. However, as these were not understood, this was not possible. Instead, the solution to the problem involved changing the way the atTargetID signal was triggered. Instead of changing value when the counter equals 20, the control of the atTargetID signal was changed to the ‘State’ variable. ‘State’ records whether the Predictor-Corrector is predicting, correcting or in ‘slow down’. In the modified method, atTargetID was set to equal 1 when the model is predicting. Therefore, in terms of the counter, this is equivalent to setting the atTargetID to change when the counter equals 1. Or, more generally, the new ‘State Controlled’ method measures the force feedback at the beginning of a timestep, whereas the original ‘Counter Controlled’ method measures it at the end of the previous step.

‘State’ appears to be much more stable than ‘counter’ and doesn’t experience any of the associated erratic misbehaviour. Hence, the new method is more robust and doesn’t suffer from delays. Figure 8-25 illustrates the differences between the old and new control methods for a normal step, while Figure 8-26 shows a step where ‘counter’ jumps from 19 to 21, i.e. a typical example of erratic misbehaviour. It can be seen that the new ‘State Controlled’ method deals with the counter misbehaviour and avoids a delay, unlike the original ‘Counter Controlled’ method.

Reached Target	Old - 'Counter Controlled'			New - 'State Controlled'			Reached Target
	Counter	State	atTarget	Counter	State	atTarget	
	19	Correct	0	19	Correct	0	
	20	Correct	1	20	Correct	0	
	1	Predict	1	1	Predict	1	
	2	Predict	1	2	Predict	1	
	3	Predict	1	3	Predict	1	
Next Command Received	4	Correct	0	4	Predict	1	Next Command Received
	5	Correct	0	5	Correct	0	

Figure 8-25 'Counter controlled' versus 'State controlled' atTargetID for a normal step

Counter not equal to 20	Old - 'Counter Controlled'			New - 'State Controlled'			Counter not equal to 20
	Counter	State	atTarget	Counter	State	atTarget	
	19	Correct	0	19	Correct	0	
	21	Correct	0	21	Correct	0	
	1	Predict	0	1	Predict	1	
	2	Predict	0	2	Predict	1	
	3	Predict	0	3	Predict	1	
	4	Correct	0	4	Predict	1	
	5	Correct	0	5	Correct	0	
			atTarget stuck at 0; delay			atTarget still OK	
						Next Command Received	

Figure 8-26 'Counter controlled' versus 'State controlled' atTargetID for a step containing counter misbehaviour

Given that the 'State Controlled' method measures the force feedback at the beginning of a timestep, as opposed to the 'Counter Controlled' method measuring it at the end of the previous step, in theory the 'State Controlled' method is measuring the force slightly after the target displacement has been reached. However, the change in position, and consequently resisting force, between the end of one step and the beginning of the next is minimal. Also, in theory, the 'State Controlled' method slows down the calculation of next command displacement, as OpenSees is receiving the measured force value slightly later, something which could become important in Real Time Hybrid Testing. However, again this is negligible, while also being impossible to quantify given that the two methods cannot be run simultaneously or parametrically.

8.6.2 NaN Errors

On occasion the predictor corrector Simulink model can return Not-a-Number (NaN) values for the force or displacement feedback. As with the counter misbehaviour, this is not repeatable, i.e. appears to occur randomly, and the cause is not understood. OpenFresco can read in these values, but OpenSees is not able to use them in the calculation of the next target displacement.

An error such as “Cannot get compatible forces and deformations for element 10: dW = #QNAN” is typically returned when OpenSees tries to execute a timestep using a NaN feedback from the previous step.

This problem was addressed by developing a filter in the Simulink model that blocks NaN values from reaching the measSignal, which is read by OpenFresco. If a NaN value is present the last non-NaN value is sent to the finite element model. This Simulink block diagrams that implement the filter are shown in Figures 8-27 and 8-28, with pseudo code given below:


```

measSig                                %Signal containing force and displacement
feedbacks
if (measSig (1) == NaN or measSig (2) == NaN)    %Check if either feedback value is NaN
    u1 = 1                                        %If it does set u1=1
else
    u1 = 0                                        %Else set u1 =0
if (u1 ==0)
    measSignal = measSig                        %allow signal through filter

```

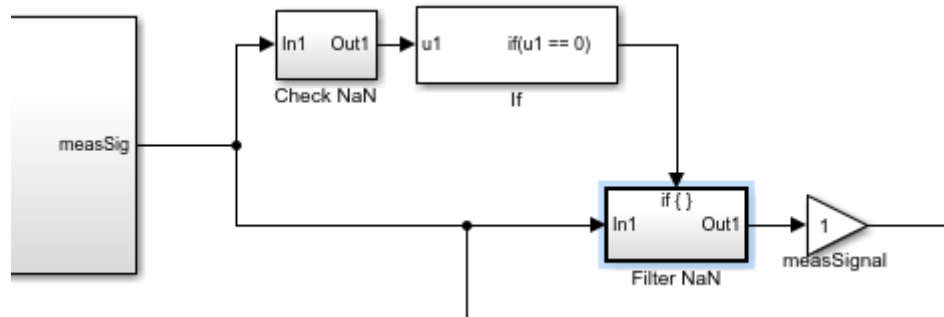


Figure 8-27 Subsystem to prevent NaN values reaching measSignal, which is read in by OpenFresco

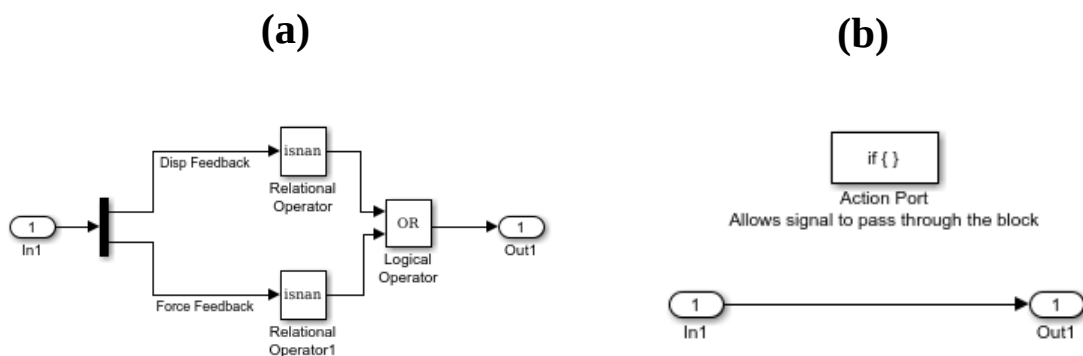


Figure 8-28 (a)The CheckNaN block which outputs a 1 if either the force or displacement signal value is NaN and (b) the Filter Nan block which

Despite this work, a number of issues are still present in the test system. Firstly, on occasion the predictor-corrector algorithm doesn't exit the 'slow-down' phase correctly after the command displacement is received. Instead of gradually return to increments of 1, as is supposed to happen, the counter stays incrementing at the 'slow down' rate until the end of the analysis step. This results in a very slow calculation step and an apparent pause in the test. However, the delay is not fatal, and the counter returns to incrementing correctly at the initiation of the next step.

Secondly, and more problematically, the integrator sometimes fails to update the model after the completion of a numerical integration step. This causes the test to stop. However, this occurs quite rarely, from experience approximately 1 on every 20 simulations. The reason for

this is not understood, and because of its rare occurrence, it is difficult to debug properly and fix the issue.

8.6.3 Other Issues

As well as dealing with the long delay and NAN errors, some further points of interest that are deserving of comment arose in preliminary simulations.

The test system at TCD works in units of kN for force and in mm for displacement. Therefore the OpenSees finite element models were set up to work in these units. Alternatively, the Experimental Control class in OpenFresco could have been used to convert units, by multiplying or dividing by an appropriate factor as the signals moved between the finite element model and the test system.

It is important that the versions of OpenSees and OpenFresco used are compatible with one another. For example, OpenSees 2.5 is not fully compatible with OpenFresco 2.6.2 and in some cases if these are used together unexpected response may be observed. For example, here it was noted that when the macro element representation of the experimental substructure described above was implemented with these two non-compatible versions, the command displacement reset to zero at the beginning of each timestep. This problem was solved by changing to compatible versions of OpenSees and OpenFresco.

Further to this, there is a slight difference in how older versions of OpenFresco and OpenFresco 2.7.0 manage the displacement and force feedback signals. OpenFresco 2.7.0 tries to read in these measurements in a two column matrix called measSignal. In older versions they are read in as 2 separate vectors, called measDisp and measForce. In order to deal with this, predictor corrector Simulink models developed for older OpenFresco versions need to be changed when moving between the two versions; a mux block can be used to combine the two separate signals into one. There are also some slight differences in syntax used to set up the experimental control in OpenFresco depending on the version used.

With the exception of some preliminary analyses, all the hybrid simulations detailed here were performed using OpenSees 2.5.0 and OpenFresco 2.7.0, which were the latest releases at the time of testing.

8.7 Conclusion

This chapter dealt with the development of a hybrid simulation test procedure for the purpose of examining the seismic response of a CBF structure.

Firstly the design of the structure to be tested was detailed. A three storey CBF designed to EC8, with the bottom storey being the experimental substructure for a substructured hybrid simulation, was developed. This experimental substructure is designed so as it should be possible to achieve post-elastic behaviour given the available test facilities.

The development of a finite element model for hybrid simulation is then discussed. The modelling of beams and columns as elastic elements and choice of integration algorithm, an implicit Newmark scheme with a fixed number of iterations, are justified through numerical analyses. The issue of representing the experimental substructure in the finite element model for compatibility with the numerical substructure is then addressed. A macro-experimental element was developed in OpenFresco in order to allow a complex experimental substructure to be tested with only one actuator as a SDOF system. Comparisons with purely numerical models showed this method worked well confirming the potential of the test method for investigating the seismic response of CBF structures on a storey level.

After this, the chapter details a number of technical challenges encountered in preliminary hybrid simulations on the test structure. Firstly, the treatment of apparently random delays in the test system is addressed. The predictor-corrector algorithm and the causes of the delays are discussed, before the alterations made to alleviate the issue are presented. Similarly, the method used to deal with NaN values in the feedback from the Simulink model is given. The latter two issues are possibly unique to the test facility at TCD, but the discussion provided and solutions presented may be of use for others addressing similar local problems elsewhere. Finally, some important points relating to the test setup that arose during preliminary testing are briefly mentioned.

In short, the Chapter demonstrates that a suitable test structure has been designed and a working hybrid simulation algorithm, including the finite element model and the experimental control, has been developed. These are employed in the following Chapters to examine the seismic response of CBFs.

9 Results of Hybrid Simulations

9.1 Introduction

This chapter presents the results of a hybrid simulation testing programme for a CBF structure. Chapter 8 detailed the design of the structure for testing, and the development and improvement of the test method at TCD. Using the developed test system, an experimental campaign was undertaken to assess the response of the test structure, and the ability of analytical models to capture this response, under ground motions of different intensities.

The test programme involved performing hybrid simulations using ground motion records with 50% and 10% probability of exceedance at a case study site. Five simulations were performed at each intensity level. The selection of the ground motion records is detailed at the beginning of the chapter. Initially, it was also planned to include ground motion records with a 2% in 50 year probability of exceedance, however due to damage to a component of the test system this was ultimately not possible.

The results of the hybrid simulations are then presented. Given that this research is focussed on performance assessment; peak inter-storey drift and peak floor acceleration are the primary *EDPs* examined. In each case, the *EDPs* are compared to the results obtained using purely numerical, analytical models, thus allowing the accuracy of these models to be assessed.

9.2 Experimental Programme

The design of the test structure for hybrid simulation was discussed in Chapter 8. As mentioned, the structure was designed for the Bulk Mail site in Los Angeles, the same site as was used in the BRACED shake table tests (Salawdeh et al., 2017). In order to assess frame response, and the performance of analytical models, under different intensity ground motion intensities, it was decided to perform hybrid simulation using ground motion records with 2%, 10% and 50% probabilities of exceedance in 50 years for the site.

9.2.1 Selection of Ground Motion Records for Experimental Testing

Ground motion records were selected to match the CMS at the site at 50%, 10% and 2% probability of exceedance in 50 years ground motion intensity levels. A Multiple Stripe Analysis (MSA) approach was adopted for the test programme, meaning different ground motion records were selected at each intensity level, as opposed to an Incremental Dynamic Analysis (IDA) approach where the same records with increasing scaling factors are used. As discussed in

Chapter 4, this means the records selected are consistent with the seismic hazard at the site at each intensity level.

The conditional mean spectra used to select the records was developed based from hazard deaggregation data obtained from the USGS online tool (United States Geological Survey, 2017). The conditional period, T^* , was taken as 0.25s, the fundamental period of the test structure calculated through an eigenvalue analysis performed using a numerical model of the test structure in OpenSees. As previously, MATLAB code, available online (https://web.stanford.edu/~bakerjw/gm_selection_old.html) (Jayaram et al., 2011b) was used to calculate the CMS and scale and select ground motion records to match it from the PEER NGA database (Chiou et al., 2008). Three ground motion records were selected at each desired intensity level.

Figure 9-1 shows the selected spectra as well as the CMS for each intensity level. Details of these ground motion records are given in Table 9-1, where they are identified as SS (site-specific) records.

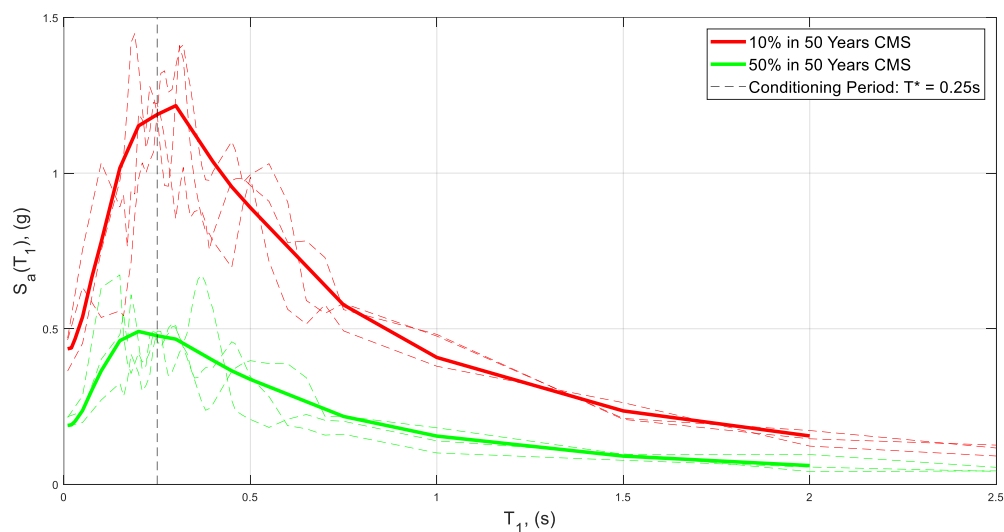


Figure 9-1 Spectra of Site Specific (SS) ground motion records selected to match the CMS at the LA Bulk Mail site at 2%, 10% and 50% probability of exceedance in 50 years intensity levels

Further to this, in order to investigate the impact of ground motion mean period (Rathje et al., 1998), on peak drift response, two more records were selected to broaden the range of T_m values used in the tests. T_m is dependent on the site conditions and ground motion records that are selected to be site specific tend to have similar T_m values, as can be seen in Table 9-1. Hence to broaden the range of T_m values, two non-site specific records were also selected. This was done by using the empirical relationships proposed by Rathje et al. (2004) to obtain

combinations of earthquake magnitude, source-site distance and site conditions corresponding to the desired T_m values. These combinations were then used to search the PEER database, and from this records with appropriate T_m values were selected. These records were then incrementally scaled to match spectral acceleration at the conditioning period, $S_a(T^*)$, at each of the three intensity levels. Figure 9-2 shows the scaled spectra of the records, where it can be appreciated that the non-site specific records differ significantly from the CMS at the site. More details are given in Table 9-1, where the records are identified as NSS (non-site specific) records.

Figure 9-3 shows the position of all the selected ground motion records on the T_1/T_m axis for a plot showing the predicted variation in global drift modification factor, as developed in Chapter 7. From this it can be appreciated why the non-site specific records were selected to broaden the range of T_1/T_m values used in the study.

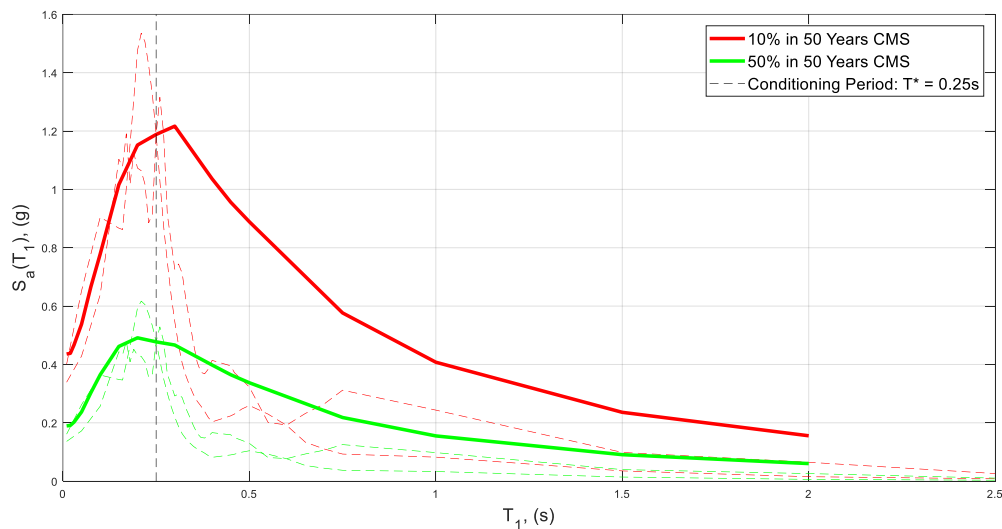


Figure 9-2 Spectra of non-Site Specific (NSS) ground motion records, selected to broaden the range of T_m values examined

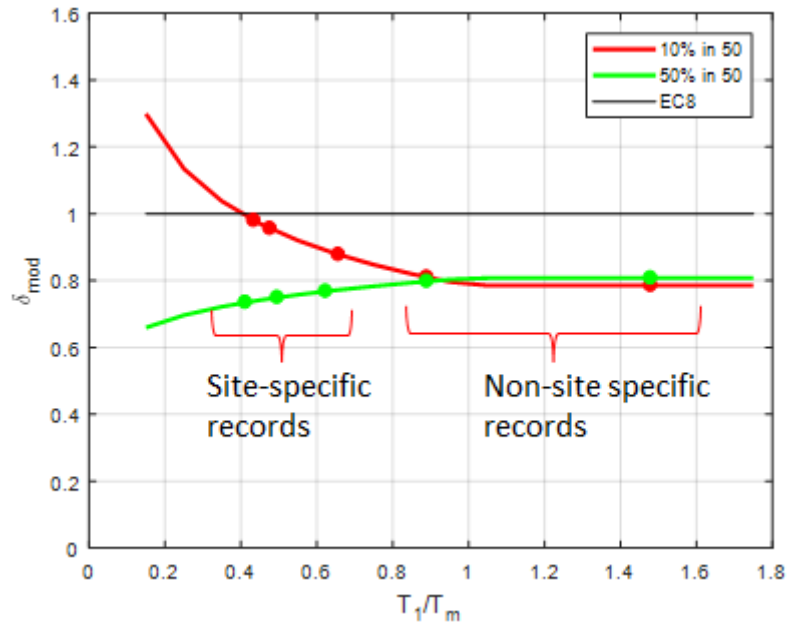


Figure 9-3 Location of selected records on T_1/T_m plot

Table 9-1 Details of selected ground motion records

ID	PEER ID	Earthquake Name	Year	Station Name	Magnitude	R_{rup} (km)	$V_{s,30}$ (m/s)	Scaling	PGA (g)	T_m (s)
50% in 50 Years										
SS_GM1_50	457	Morgan Hill	1984	Gilroy Array #3	6.19	13.02	349.85	1.08	0.22	0.59
SS_GM2_50	357	Coalinga-01	1983	Parkfield - Stone Corral 3E	6.36	34.00	565.08	1.24	0.19	0.49
SS_GM3_50	594	Whittier Narrows-01	1987	Baldwin Park - N Holly	5.99	16.72	544.68	1.61	0.21	0.39
NSS_GM4_50	80	San Fernando	1971	Pasadena - Old Seismo Lab	6.61	21.5	969.07	1.43	0.14	0.27
NSS_GM5_50	531	N. Palm Springs	1986	Puerta La Cruz	6.06	67.38	442.7	2.10	0.16	0.16
10% in 50 Years										
SS_GM1_10	1602	Duzce_ Turkey	1999	Bolu	7.14	12.04	293.57	0.63	0.46	0.56
SS_GM2_10	1082	Northridge-01	1994	Sun Valley - Roscoe Blvd	6.69	10.05	320.93	1.32	0.37	0.51
SS_GM3_10	1615	Duzce_ Turkey	1999	Lamont 1062	7.14	9.14	338.00	3.96	0.47	0.37
NSS_GM4_10	80	San Fernando	1971	Pasadena - Old Seismo Lab	6.61	21.5	969.07	3.56	0.34	0.27
NSS_GM5_10	531	N. Palm Springs	1986	Puerta La Cruz	6.06	67.38	442.7	5.22	0.40	0.16

Hybrid simulations were performed using each of these ground motion records. The three-loop experimental setup discussed and validated in Chapter 8 was employed. Hence, the experimental substructure was represented using the generic macro-element approach in OpenFresco and the numerical substructure modelled in OpenSees using previously discussed modelling practices. The predictor-corrector routine developed by Schellenberg et al. (2009c), and altered slightly to avoid the local errors discussed in Chapter 8, was implemented in Simulink to synchronise displacement commands from OpenSees with the servo-controller.

9.3 Experimental Results & Discussion

9.3.1 Test Results and Observations

9.3.1.1 50% in 50 Year Simulations

9.3.1.1.1 SS-GM1-50

Figure 9-4 shows the experimental response of the structure for the SS-GM1-50 test run. The upper force displacement hysteresis plots shows response of the experimental substructure, obtained from the force and displacement feedback measured by the actuator. The data is plotted so that displacement in the negative direction corresponds to compression in the experimental brace. The lower plot shows the force-displacement response using the value of base shear calculated by the OpenSees-OpenFresco model. This is different from the force applied by the actuator as it accounts for the numerical as well as the experimental substructure.

The response for the experimental substructure appears almost perfectly linear with a lateral stiffness of approximately 10.8kN/mm (see Table 9-4). There was no visible evidence of brace buckling or yielding during the test. This is despite the peak force in the negative direction, which corresponds to compression in the tested brace, being greater than the horizontal force that results in a compression force in the brace equal to the buckling capacity, calculated according to Eurocode 3 (CEN, 2005), $N_{b,Rd}$. This apparent underestimation of the predicted buckling load can be mainly attributed to simplifications in the calculation; pinned-pinned end conditions were assumed for the brace while no contribution to lateral stiffness from the beam-column assembly was considered. Furthermore, it should be noted that when buckling curve C is used, as is specified for cold formed hollow steel sections, the Eurocode 3 buckling load is based on an assumed geometric imperfection of $L/200$, or 0.5% of the brace length. This is higher than the conventionally assumed value of 0.1% used in numerical models to capture brace buckling and may be excessive for cyclic or seismic type loading where the brace is straightened when during the tension part of the loading cycle. The peak force is not near the calculated horizontal force necessary to cause tensile brace yielding, i.e. the horizontal component of $N_{pl,Rd}$, and no evidence of yielding is displayed in the force-displacement plot.

The hysteresis plot including the numerical substructure displays some nonlinear behaviour when displacement is positive. This corresponds to buckling behaviour in the numerical brace at the first level. This is despite the peak force being similar in both the positive and negative directions, suggesting the buckling load of the numerically modelled brace is lower than the

experimentally tested brace. This trend is evident throughout the experimental programme, particularly at higher intensity levels.

The drift and acceleration time histories at the location of the actuator and at roof level are also shown. The drift time history at the actuator node shows both the target and measured drifts; these are practically identical showing that the actuator applies the calculated displacements as intended. Peak inter-storey drift, which occurs at the experimentally tested ground floor, is less than 0.2%, meaning from a performance assessment viewpoint little or no drift dependant structural damage is likely to be experienced in the event of such an earthquake scenario. Brace buckling in CBFs is generally expected at drifts of approximately 0.35% (FEMA, 2012a), although clearly the exact value is dependent on the individual characteristics of the brace member. However, some minor drift dependent non-structural damage, for example to partition walls, is possible at these drifts. The peak floor acceleration, approximately 0.6g, occurs at the roof level, as is generally the case for linear response. Such levels of floor acceleration would be expected to cause some non-structural damage to acceleration sensitive components, particularly if components were not specifically designed for earthquake scenarios. It should be pointed out that the acceleration time histories shown are the values calculated by the OpenSees model, as opposed to values actually measured during the test, as these tests were not run in real time.

The test was aborted after just less than 10 seconds of the ground motion. This was because of the existing error in the test system that causes the integrator to stop, as discussed in Chapter 8. Comparison with numerical simulations, as shown in Figure 9-24, suggests that the peak response has already been achieved at this point in the test.

SS-GM1-50

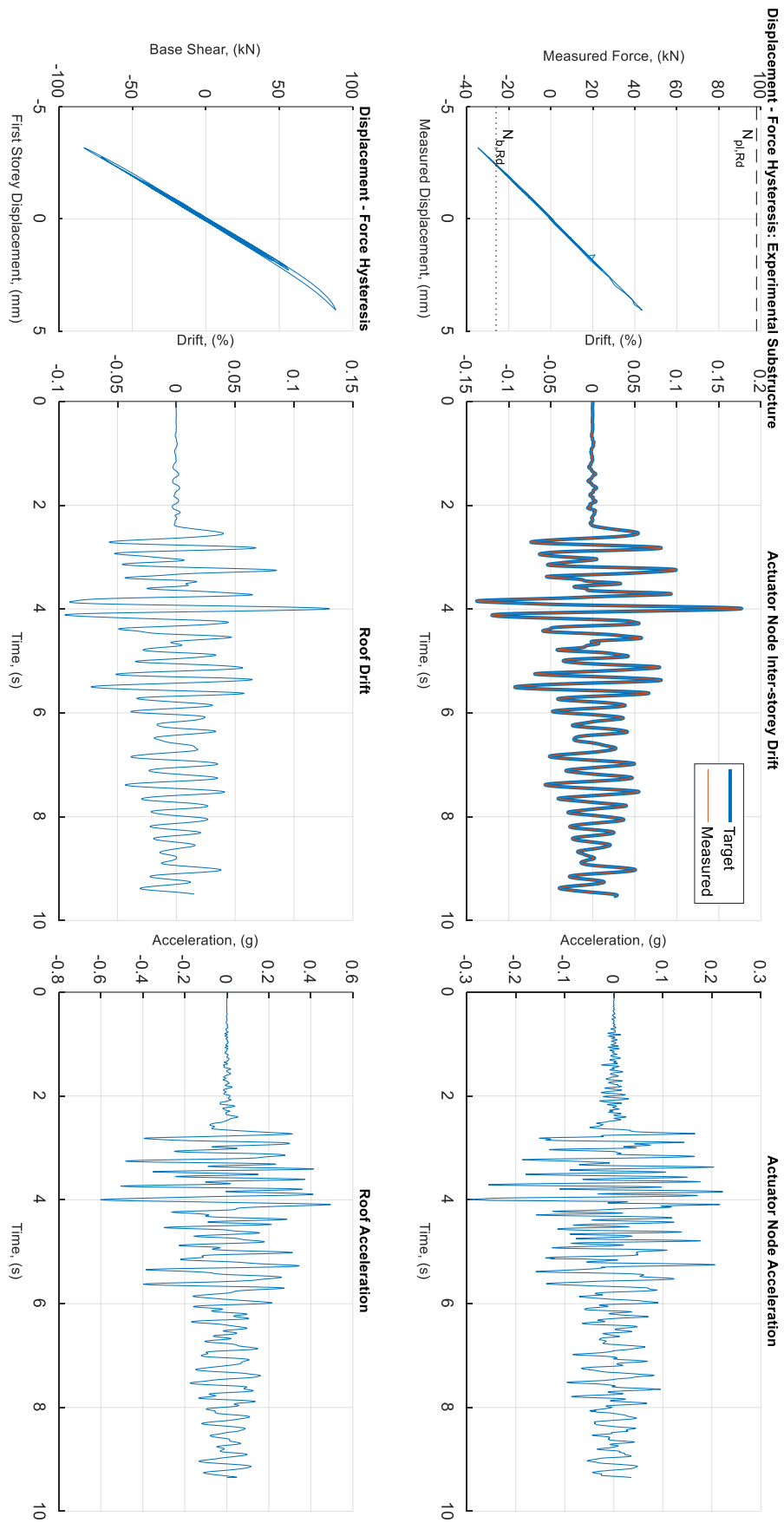


Figure 9-4 Experimental response for the SS-GM1-50 simulation

Figure 9-5 compares the drift and acceleration response at the roof level to those of a linear single degree of freedom system with the same fundamental period as the test substructure, $T = 0.25\text{s}$. The SDOF response was calculated using a linear Newmark routine implemented in Matlab. It can be seen that the drift response in particular is very similar to that of the SDOF structure, showing that for this low intensity earthquake the drift response is dominated by the first mode, which is as expected for a relatively low-rise structure like this with a first mode participation factor of 0.89. While still dominated by the first mode, there is some more evidence of higher mode effects in the acceleration response, where some higher frequency oscillations can be noted in the experimental data. The impact of higher modes on response is expected to be more evident in the acceleration response compared to drift.

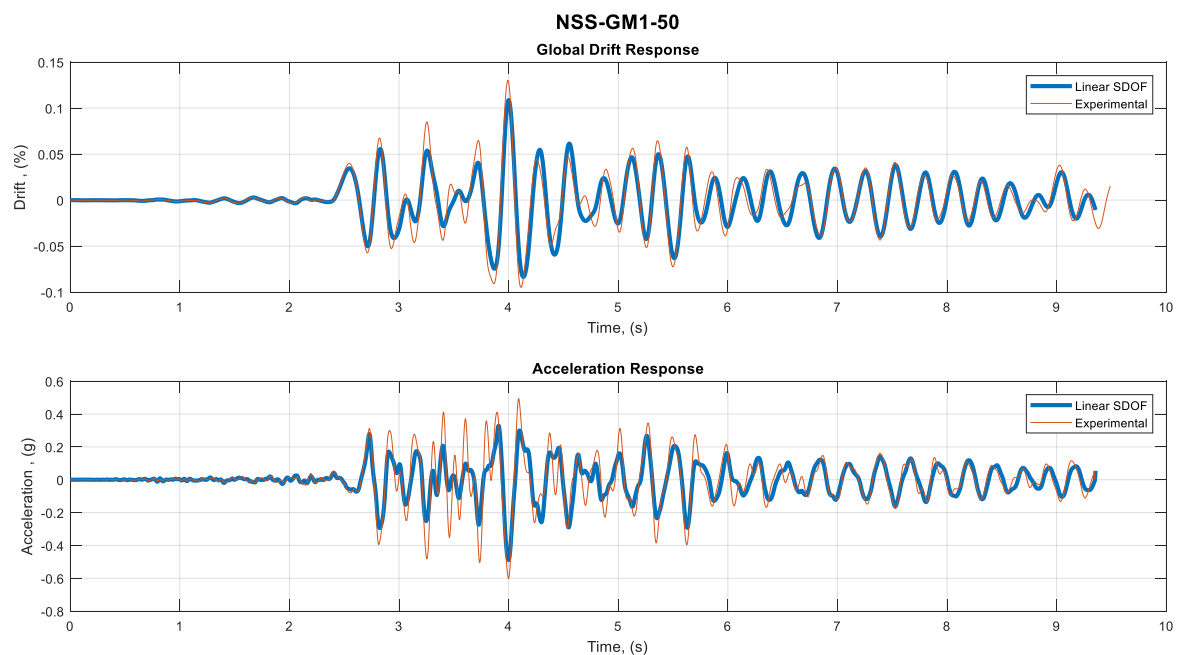


Figure 9-5 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system

In general these trends are repeated for each of the 50% in 50 year simulations, as shown in Figures 9-7, 9-10, 9-12 and 9-14. Table 9-2 gives the value of the peak roof drift and acceleration, and it can be seen that these are generally greater than the values suggested by the SDOF response. It should be noted that as all the ground motions are selected and scaled such that the spectral acceleration at $T = 0.25\text{s}$ is constant, the same peak response values are predicted for each simulation.

Table 9-2 Comparison of peak roof drift and acceleration with SDOF system response for the 50% in 50 year hybrid simulations

Test ID	Peak Roof Drift (%)		Peak Roof Acceleration (g)	
	SDOF	Hybrid Sim	SDOF	Hybrid Sim
SS-GM1-50	0.11	0.13	0.48	0.60
SS-GM2-50	0.11	0.12	0.48	0.46
SS-GM3-50	0.11	0.13	0.48	0.60
NSS-GM4-50	0.11	0.10	0.48	0.52
NSS-GM5-50	0.11	0.14	0.48	0.74

9.3.1.1.2 SS-GM2-50

Figure 9-6 shows the experimental response for SS-GM2-50. Many of the trends are similar to SS-GM1-50 discussed above. The behaviour of the experimental substructure appears almost perfectly linear despite exceeding the estimated buckling load, 26.05kN. However, as before there is evidence of a small amount of buckling in the numerical substructure.

The peak response is slightly less than for SS-GM1-50, with peak inter-storey drift of approximately 0.15% and peak floor acceleration of less than 0.5g. Uniquely for the 50% in 50 year tests, the peak roof acceleration is less than the value for the equivalent linear SDOF.

SS-GM2-50

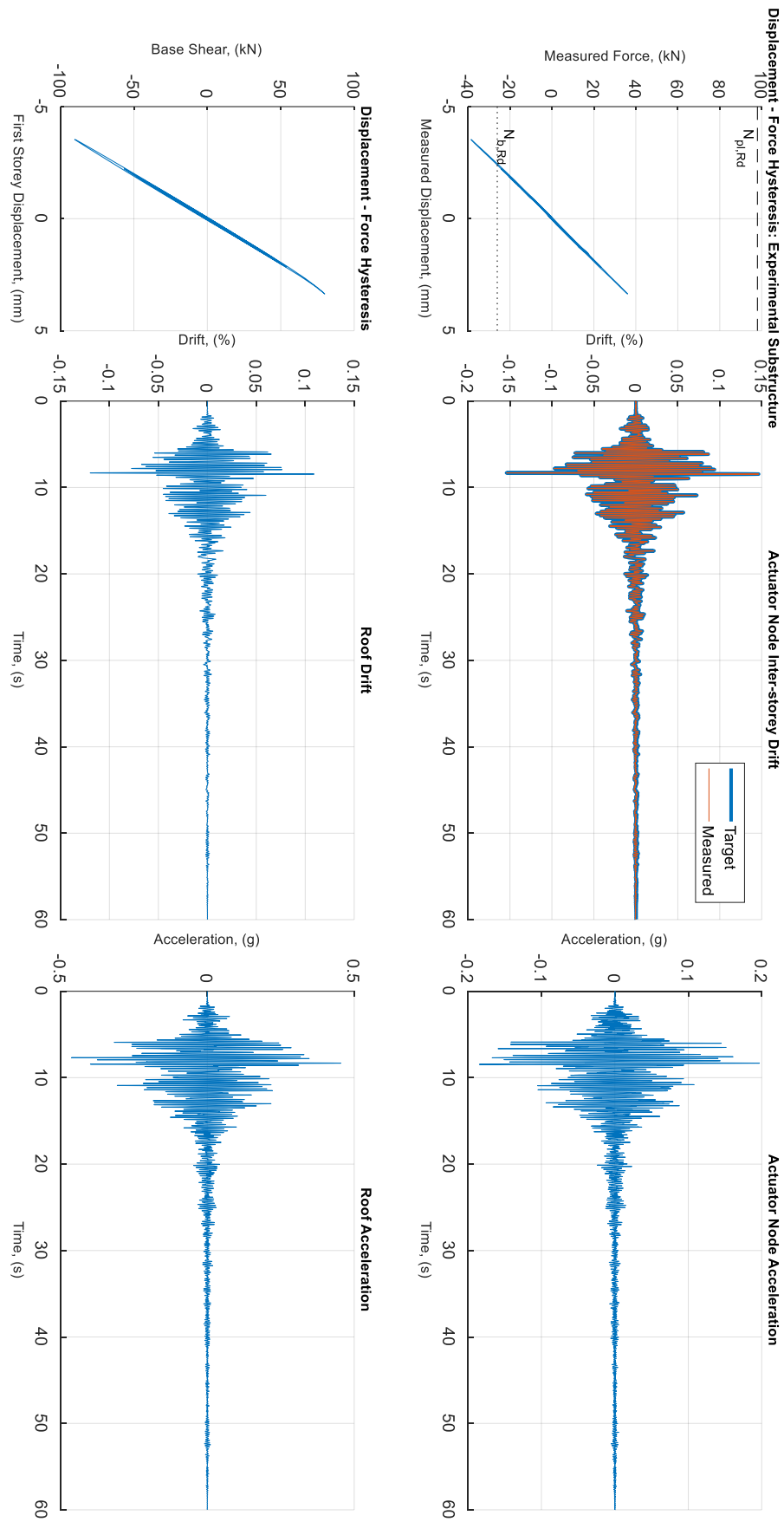


Figure 9-6 Experimental response for the SS-GM2-50 simulation

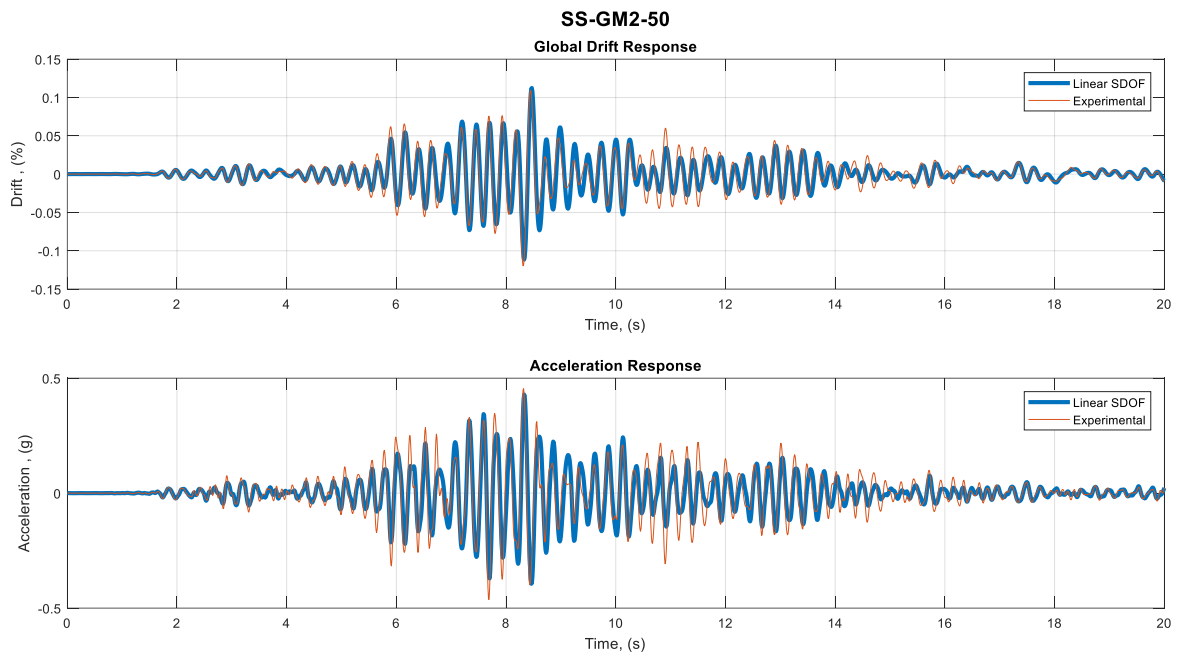


Figure 9-7 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for SS-GM2-50 over the first 20 seconds of the simulation

9.3.1.1.3 SS-GM3-50

The experimental results from SS-GM3-50 are shown in Figure 9-8. Again, broadly similar trends to the earlier tests are observed. The experimental substructure exhibits linear response with an approximate horizontal stiffness of 10.8kN/mm, with no buckling behaviour observed. However, some buckling appears to occur in the numerical substructure. The level of peak inter-storey drift and peak floor acceleration are broadly similar to previous tests, with both drift and acceleration response at roof level being greater than the corresponding SDOF response.

An error was observed in the measured displacement signal, which deviated momentarily from the target signal by approximately -2mm on two separate occasions. This can be seen in both the hysteresis plot for the experimental substructure and in the time history, at approximately 3 seconds. This portion of the time history, as well as the corresponding measured force and force-displacement plots, are shown in more detail in Figure 9-9. It can be seen that the measured restoring force does not appear to be affected. Firstly, this suggests that this was just a measurement error and the displacement spikes were not actually applied to the experimental substructure, as there would be a corresponding spike in the force measurement if this was the case. Secondly, as only the measured force is used in the hybrid simulation calculation to solve the equations of motion, these spurious displacement measurements do not affect the time stepping routine.

SS-GM3-50

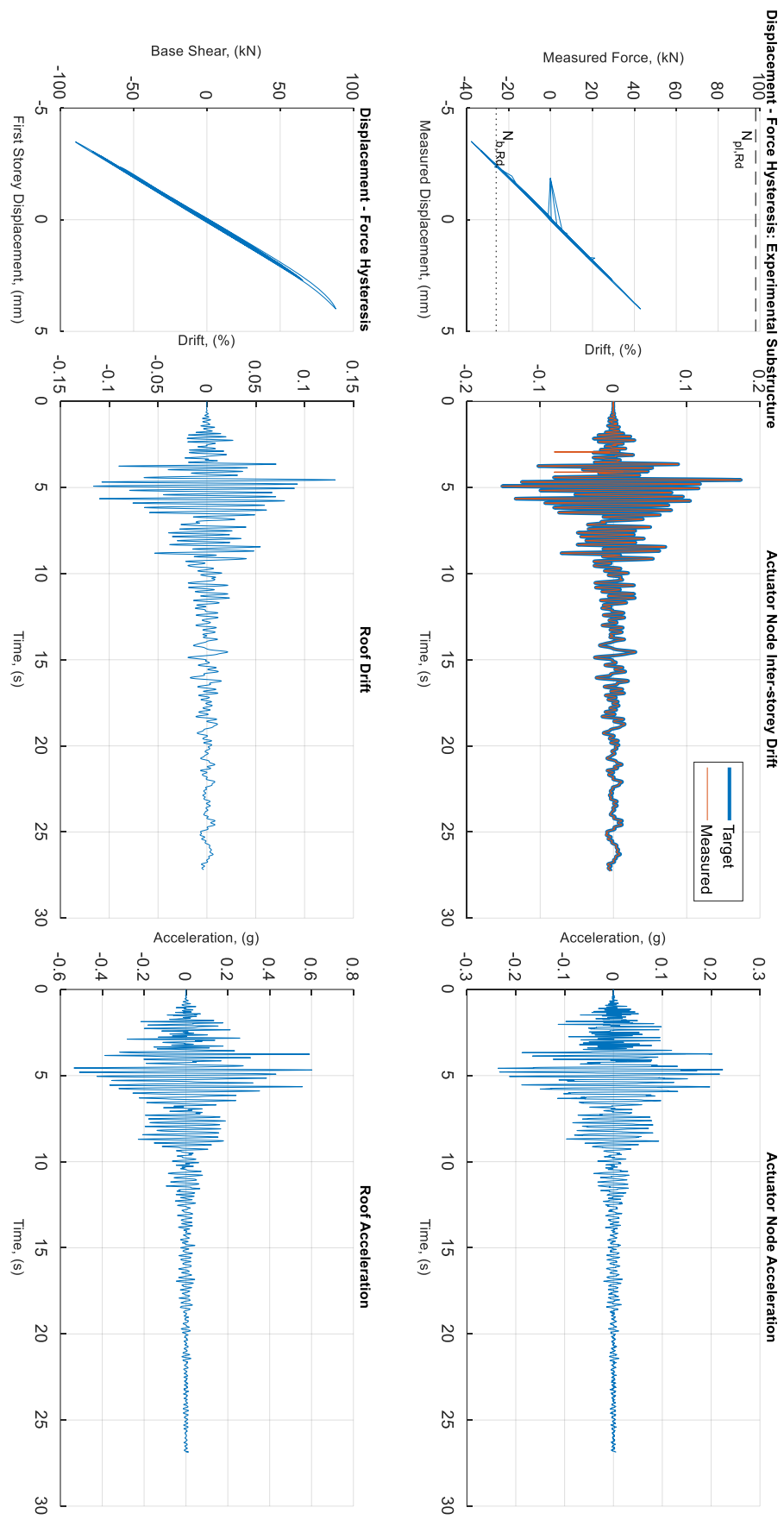


Figure 9-8 Experimental response for the SS-GM3-50 simulation

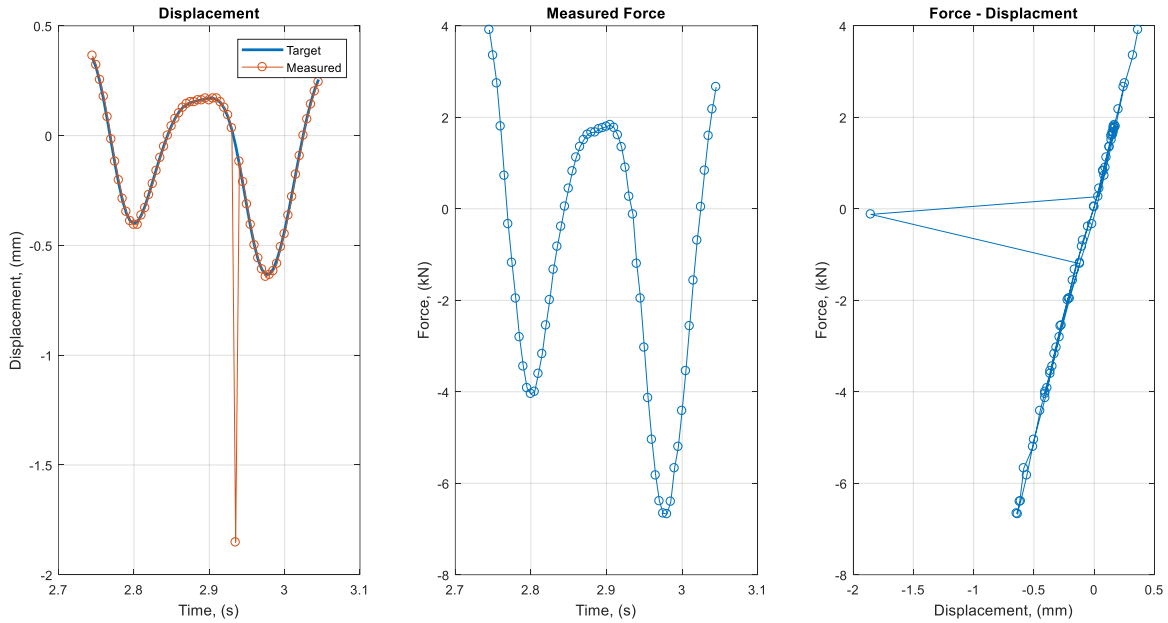


Figure 9-9 Example of a spike in the displacement measurement and its impact on force-displacement response

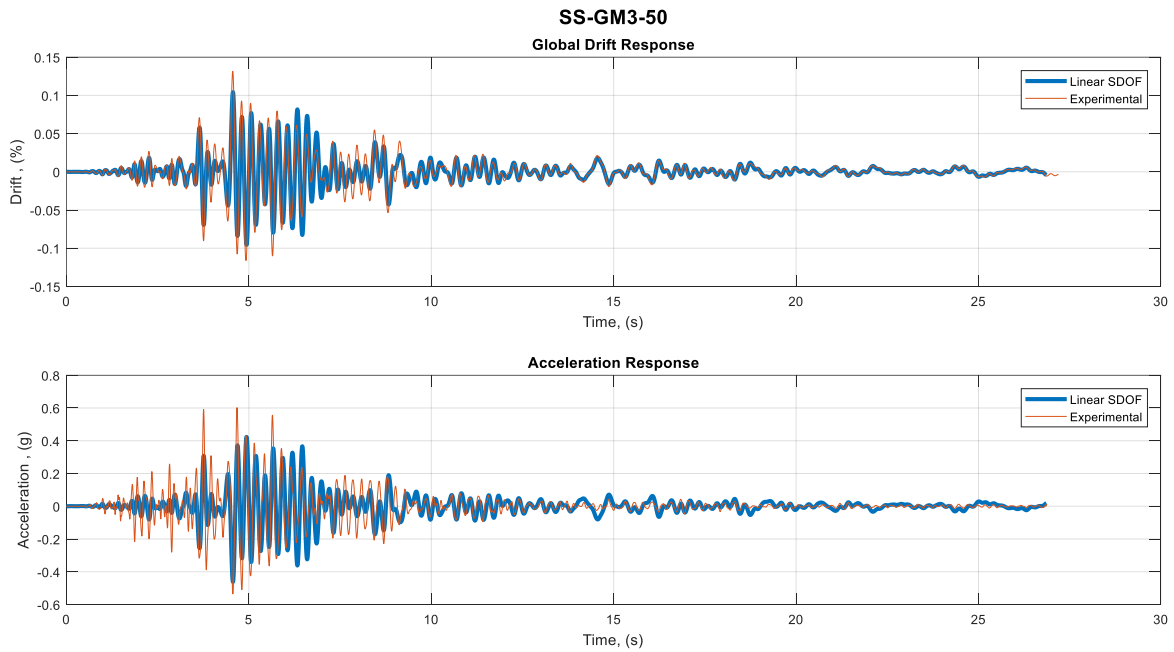


Figure 9-10 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM3-50 test run

9.3.1.1.4 NSS-GM4-50

Figure 9-11 shows the experimental response for NSS-GM4-50. The level of peak response and the general trends are similar to those discussed previously. The hysteresis plots both indicate that response is entirely linear, with no evidence of nonlinear behaviour in the numerical substructure. As with SS-GM3-50, a spurious spike in the displacement feedback after approximately 2.5s is noted.

NSS-GM4-50

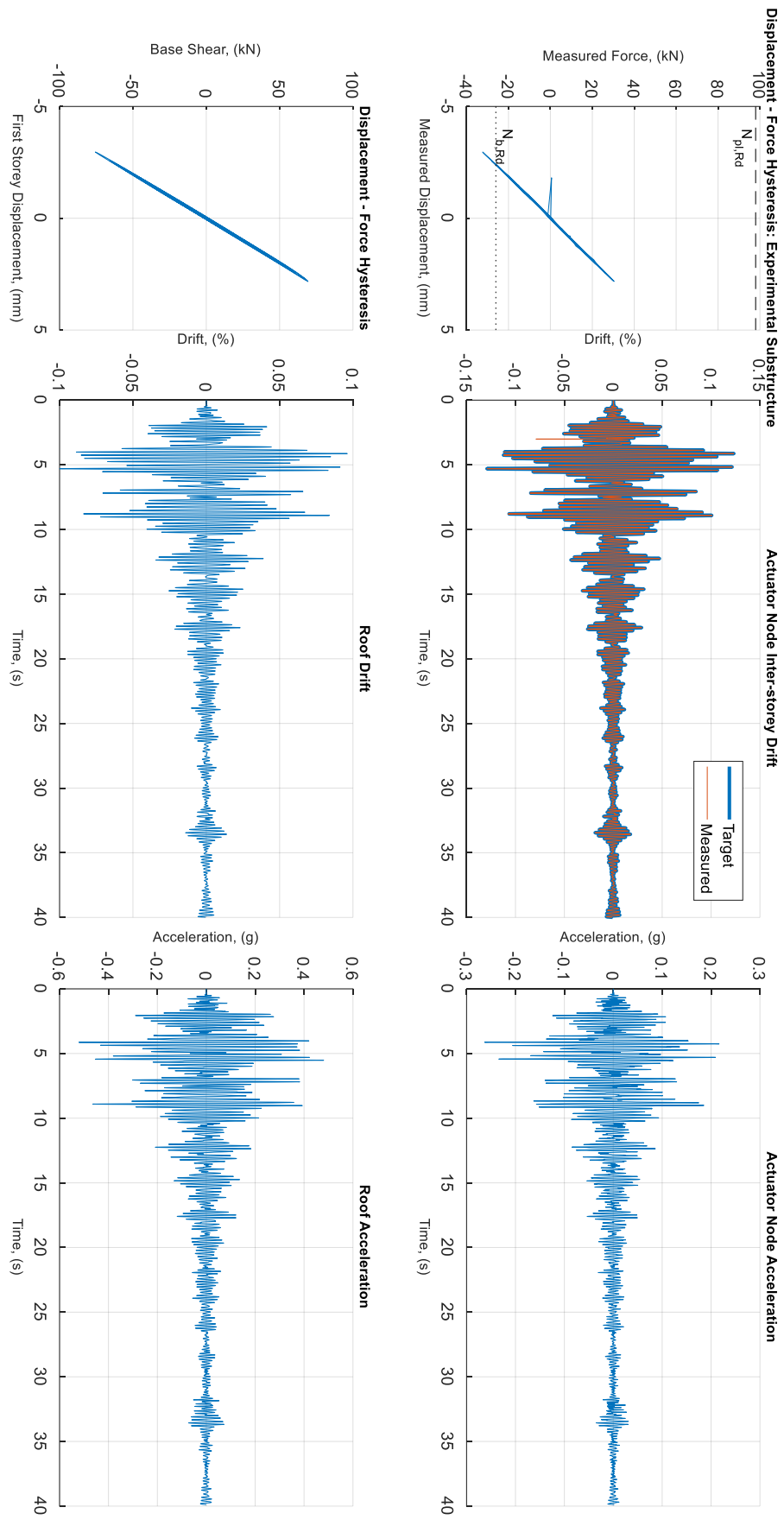


Figure 9-11 Experimental response of the NSS-GM4-50 simulation

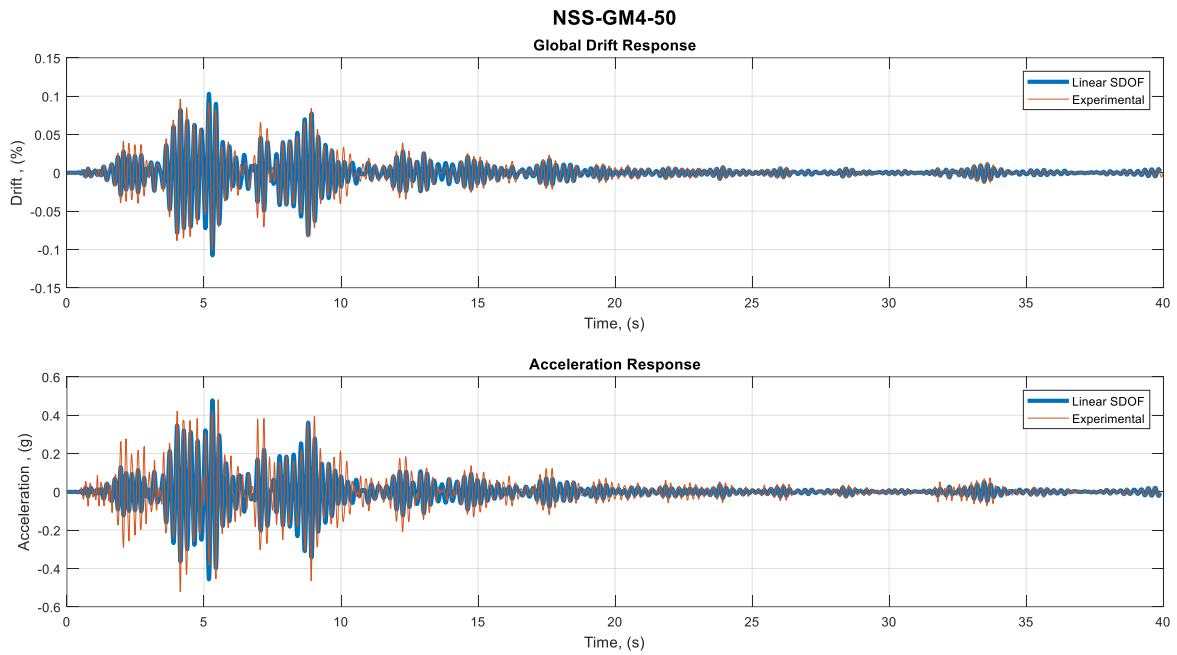


Figure 9-12 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM4-50 test run

9.3.1.1.5 NSS-GM5-50

The results of the NSS-GM5-50 simulation, shown in Figure 9-13, exhibit similar trends to the other simulations at the 50% in 50 year intensity level. Behaviour of the experimental substructure is broadly linear, with spurious displacement measures noted at two points. Some nonlinear response is noted in the numerical substructure. The peak drift and acceleration response at both the actuator node and roof level are greater for this simulation than any of the other 50% in 50 year simulations, with the peak roof acceleration response of $0.74g$ in particular being noticeably larger than the next highest value of $0.60g$.

Like the other simulations, the experimental roof drift is slightly greater than that of the equivalent SDOF structure. The peak roof acceleration is also greater than the value for the equivalent linear SDOF structure, as with all bar one of the other simulations. However, the difference between the recorded acceleration and the SDOF value, $0.26g$, in this simulation is more than twice the value noted in any of the others, where the maximum difference is $0.12g$.

NSS-GM5-50

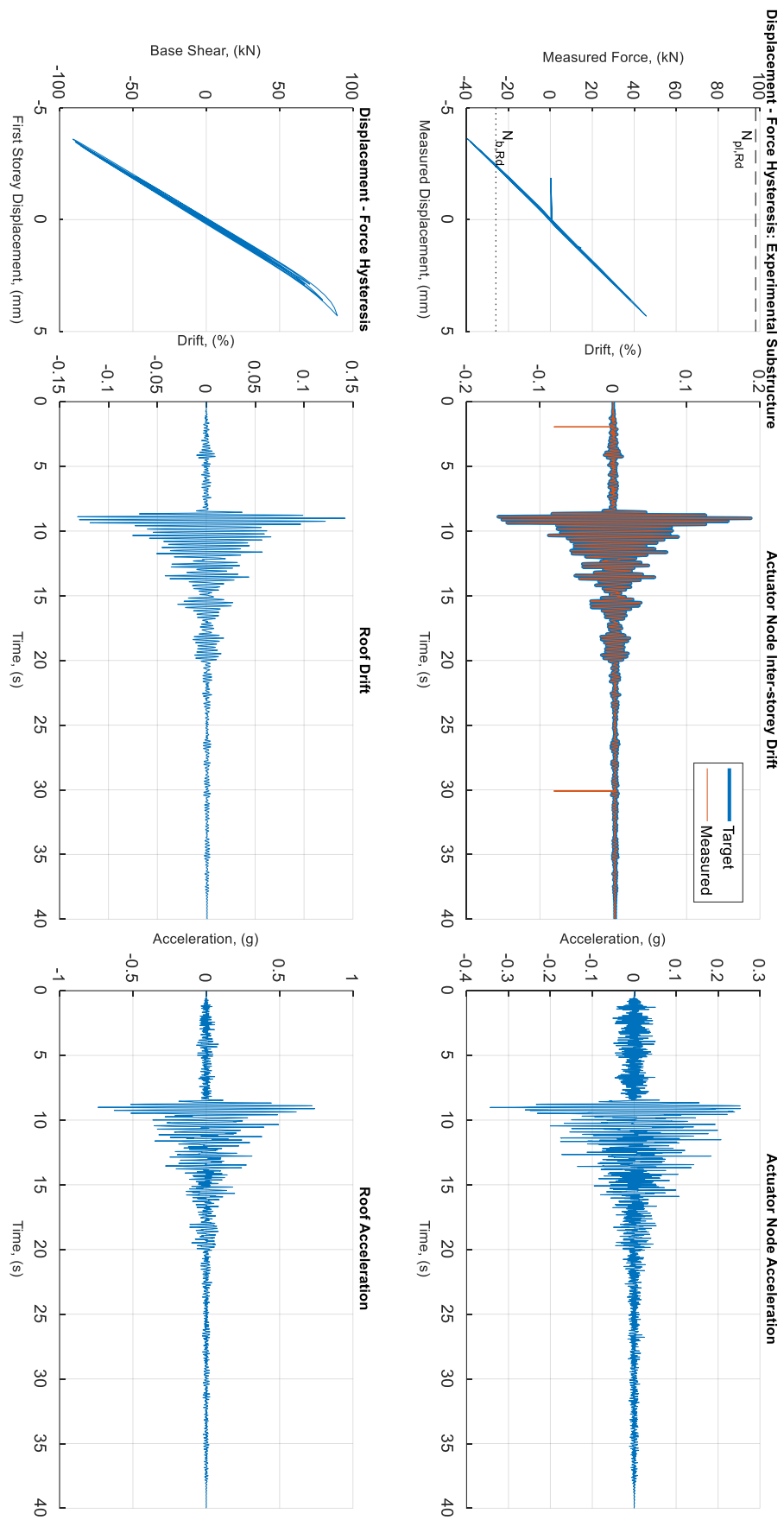


Figure 9-13 Experimental response for the NSS-GM5-50 simulation

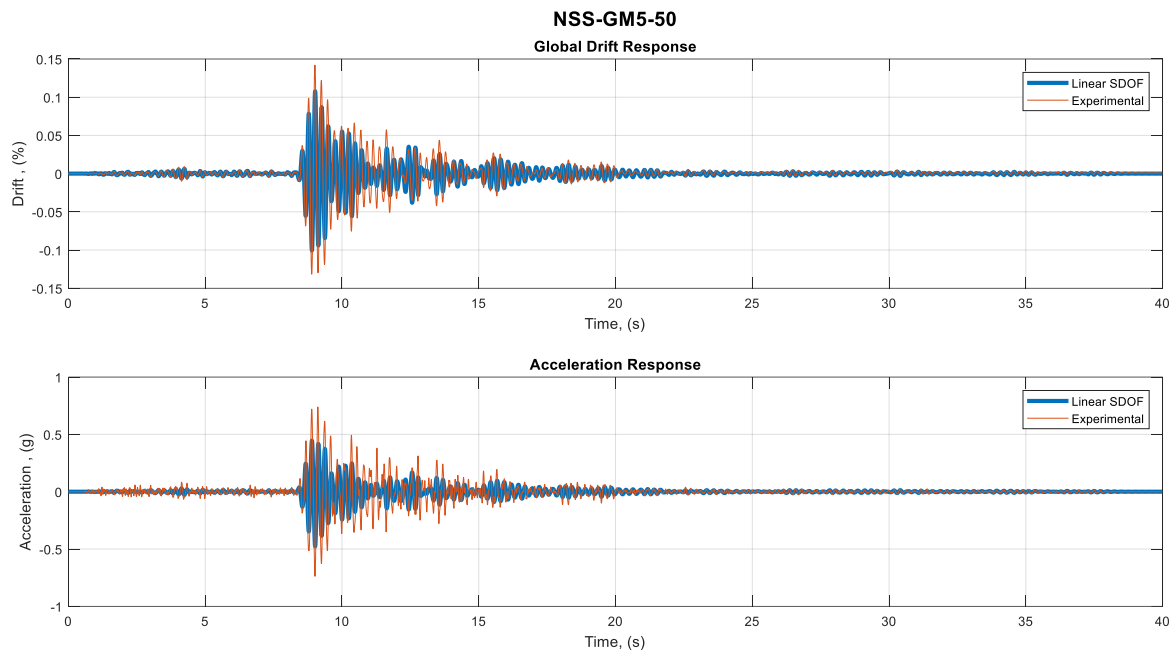


Figure 9-14 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM5-50 test run

9.3.1.2 10% in 50 Year Simulations

9.3.1.2.1 SS-GM1-10

Figure 9-15 shows the experimental response observed in the SS-GM1-10 hybrid simulation. Despite the peak compression force being almost twice the approximate $N_{b,Rd}$ value, no brace buckling was visible during the test. This observation is supported by the measured force-displacement plot for the experimental substructure, which appears to be almost perfectly linear. The peak inter-storey drift was 0.4%, meaning brace buckling would generally be expected. However, this occurred only in the positive direction with the experimental brace in tension; the peak inter-storey drift in the negative direction was approximately 0.2%. Hence, it is reasonable that no buckling was noted in the experimental substructure. However, the experimental hysteresis plot shows clear evidence of energy dissipation for positive displacement, highlighting that buckling occurred in the numerically modelled brace, which is as expected given the peak inter-storey drift. In hindsight, preliminary numerical analysis could have been used to highlight the potential for this unbalanced drift demand prior to the test and the ground motion could have been applied in the opposite direction in order to induce buckling in the experimentally tested brace. There was no evidence of tensile yielding in either brace.

From a performance assessment viewpoint, minor, non-structural drift dependant damage would be very probable given such drift demands. Peak floor acceleration demand was experienced at the roof level and approached $1.2g$, meaning significant damage to some components is possible, while the peak floor acceleration at the ground storey, approximately $0.6g$, would be expected to lead to moderate damage.

Figure 9-16 compares the experimental response at the roof level with a linear SDOF system. As with the linear 50% in 50 year responses, the two response histories are reasonably similar. This appears to be particularly true for the drift response in this case. Acceleration response calculated through the experiment is initially greater than for the SDOF system, however the SDOF response is greater later in the test. This can be attributed to yielding in the numerical substructure resulting in reduced acceleration demand compared to a linear system, where no yielding occurs.

Table 9-3 compares the peak roof response values with those for the corresponding linear SDOF system. Peak drift response is generally greater for the hybrid simulation, while no pattern is clear in peak acceleration response. Compared to an SDOF system, higher mode contributions can increase peak acceleration response of a nonlinear MDOF system, whereas yielding tends to reduce it, meaning it is reasonable that varying trends are observed here. The overall level of agreement between the hybrid simulation and the SDOF peak acceleration is equal to or slightly better than that observed in the 50% in 50 year response above (Table 9-2).

Table 9-3 Comparison of peak roof drift and acceleration with SDOF system response for the 10% in 50 year hybrid simulations

Test ID	Peak Roof Drift (%)		Peak Roof Acceleration (g)	
	SDOF	Hybrid Sim	SDOF	Hybrid Sim
SS-GM1-10	0.27	0.27	1.19	1.18
SS-GM2-10	0.27	0.34	1.19	1.09
SS-GM3-10	0.27	0.33	1.19	0.98
NSS-GM4-10	0.27	0.33	1.19	1.16
NSS-GM5-10	0.27	0.28	1.19	1.42

SS-GM1-10

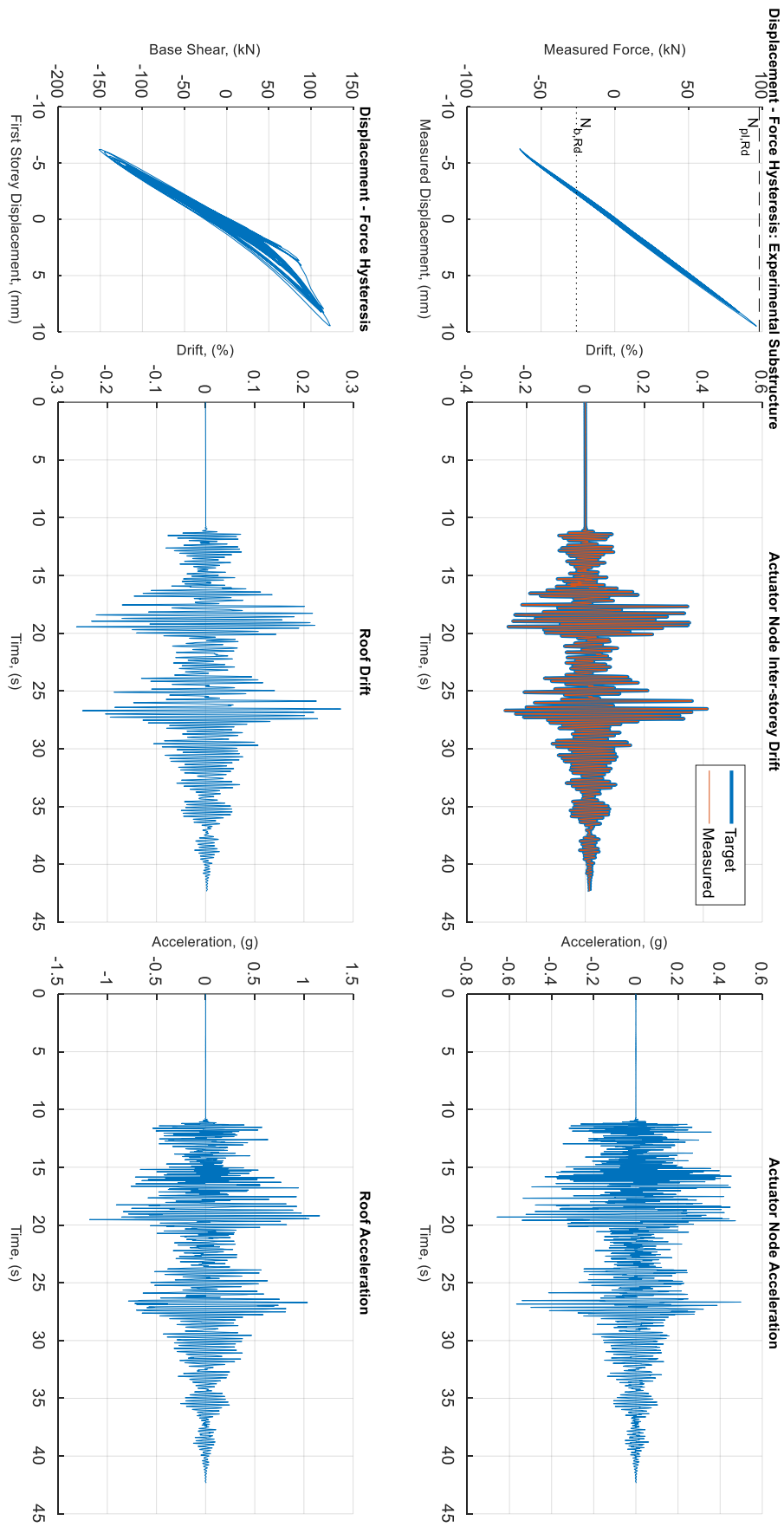


Figure 9-15 Experimental response for the SS-GM1-10 simulation

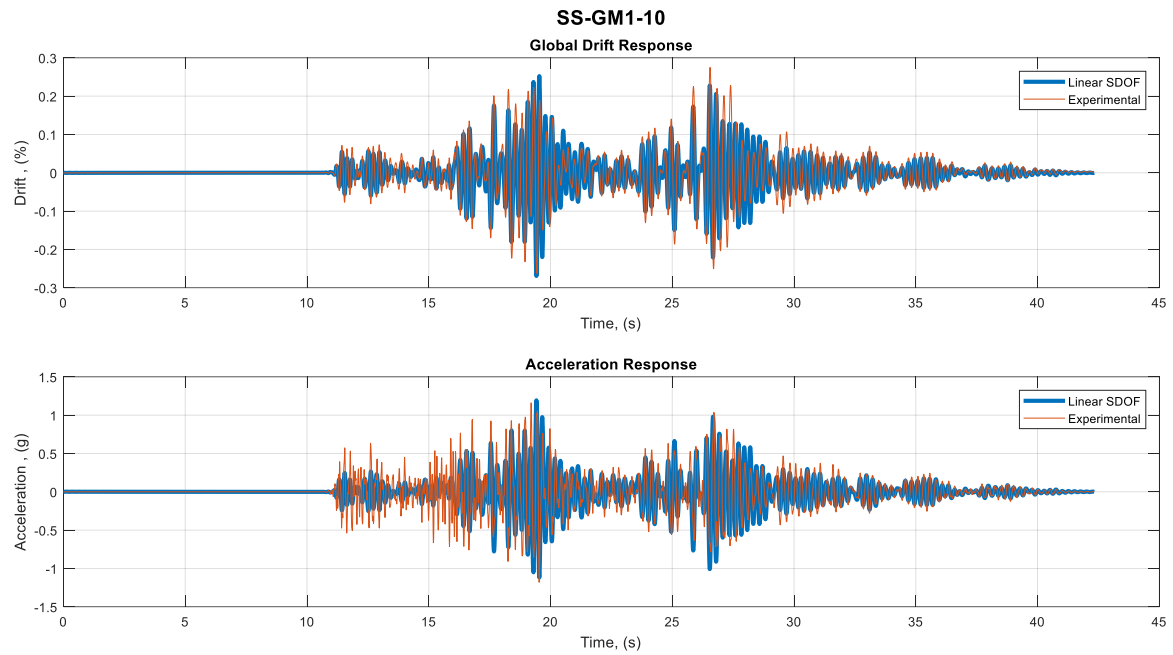


Figure 9-16 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM1-10 test run

9.3.1.2.2 SS-GM2-10

Figure 9-17 shows the experimental results for the SS-GM2-10 simulation. As with the SS-GM1-10 simulation, there is greater drift demand in the positive direction with the experimental brace in tension. However, here, the peak force experienced by the experimental substructure in the positive direction exceeds the horizontal component of the design plastic resistance, $N_{pl,Rd}$, and the force-displacement plot for the experimental substructure suggests tensile yielding occurs. Again, buckling is not expected and does not occur due to the lower drift demand in the negative direction of 0.2%. The hysteresis plot including the numerical substructure shows clear evidence of buckling in the numerical substructure, as would be expected with a peak inter-storey drift of 0.5%. Peak acceleration demands are similar to SS-GM1-10 at approximately 0.6g and 1.0g at the first floor and roof level respectively.

The response is compared to a linear SDOF system in Figure 9-18. As before, these are quite similar, with peak displacement response being greater for the experiment and peak acceleration greater for the SDOF system, something which again can be associated with inelastic response.

SS-GM2-10

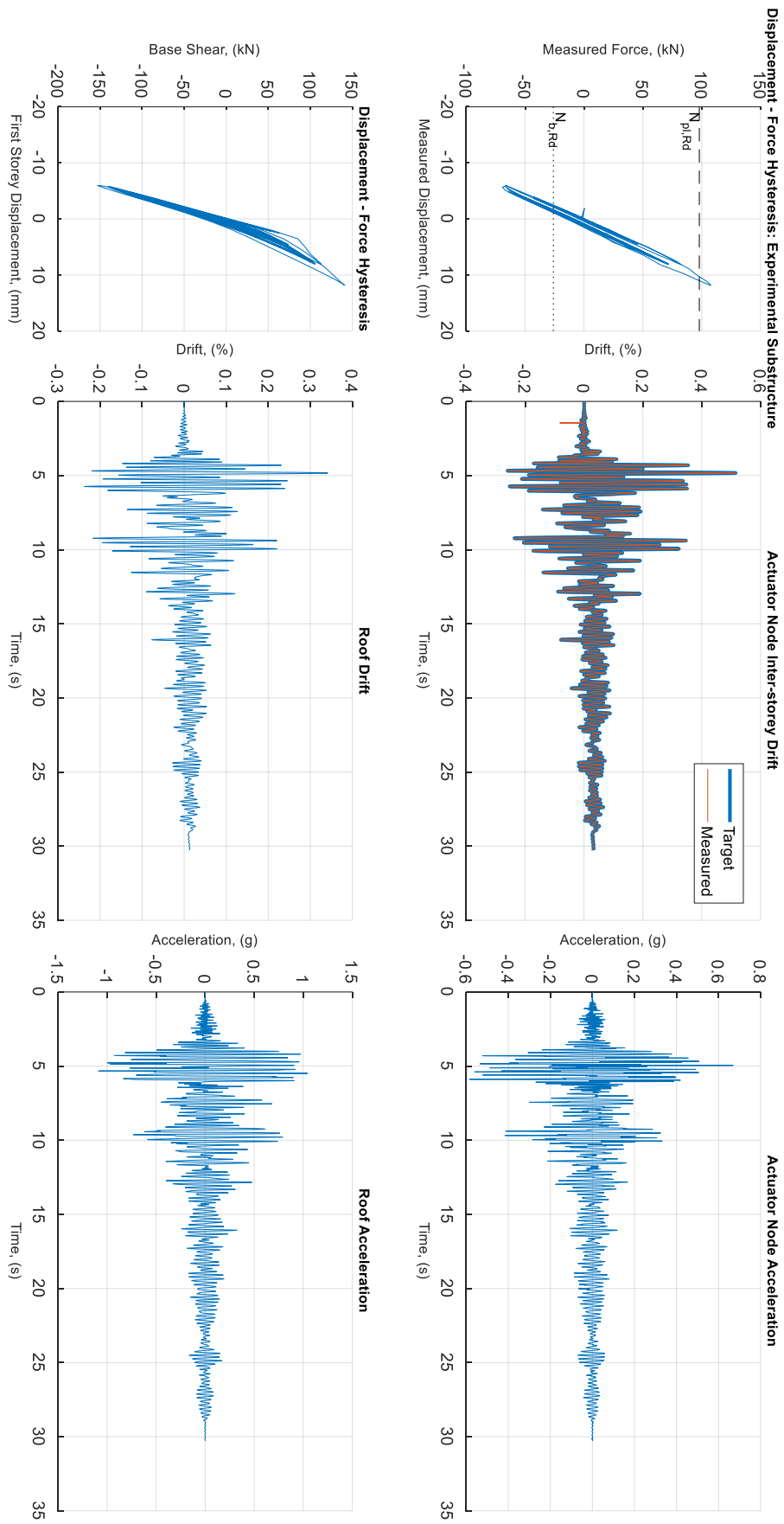


Figure 9-17 Experimental response for the SS-GM2-10 simulation

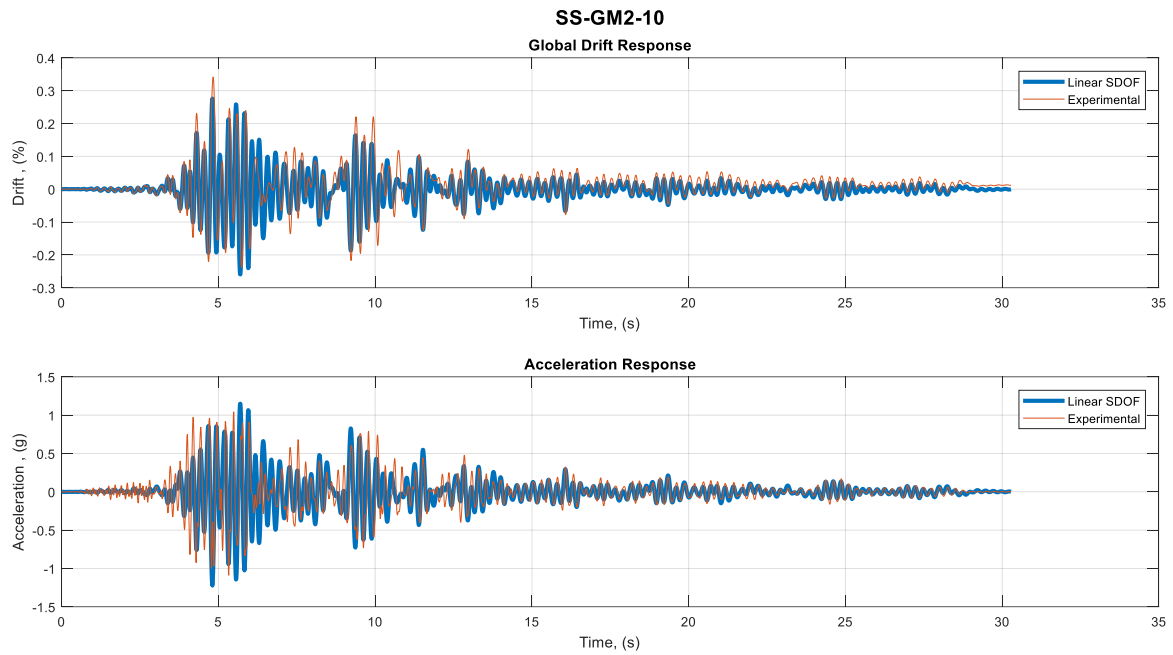


Figure 9-18 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM2-10 test run

9.3.1.2.3 SS-GM3-10

Figure 9-19 shows the results of the SS-GM3-10 simulation. Drift demand is similar in both the positive and negative directions, with peak demands of between 0.3% and 0.4%. Brace global buckling was visible during the simulation and there is some evidence of brace buckling in the hysteresis plot for the experimental substructure, however this is not particularly pronounced.

As mentioned previously, there appears to be greater yielding and energy dissipation in the numerically modelled brace than in the experimentally tested specimen. The drift demands are reasonably even in both directions; however, the hysteresis loop is asymmetrical. More energy dissipation and reduced force demand is seen in the positive displacement direction, i.e. when the experimental brace is in tension and the numerical brace is in compression, indicating a greater degree of buckling in the numerical substructure.

Trends in peak acceleration response and comparison with the corresponding SDOF response, as shown in Figure 9-20, are similar to those previously discussed.

SS-GM3-10

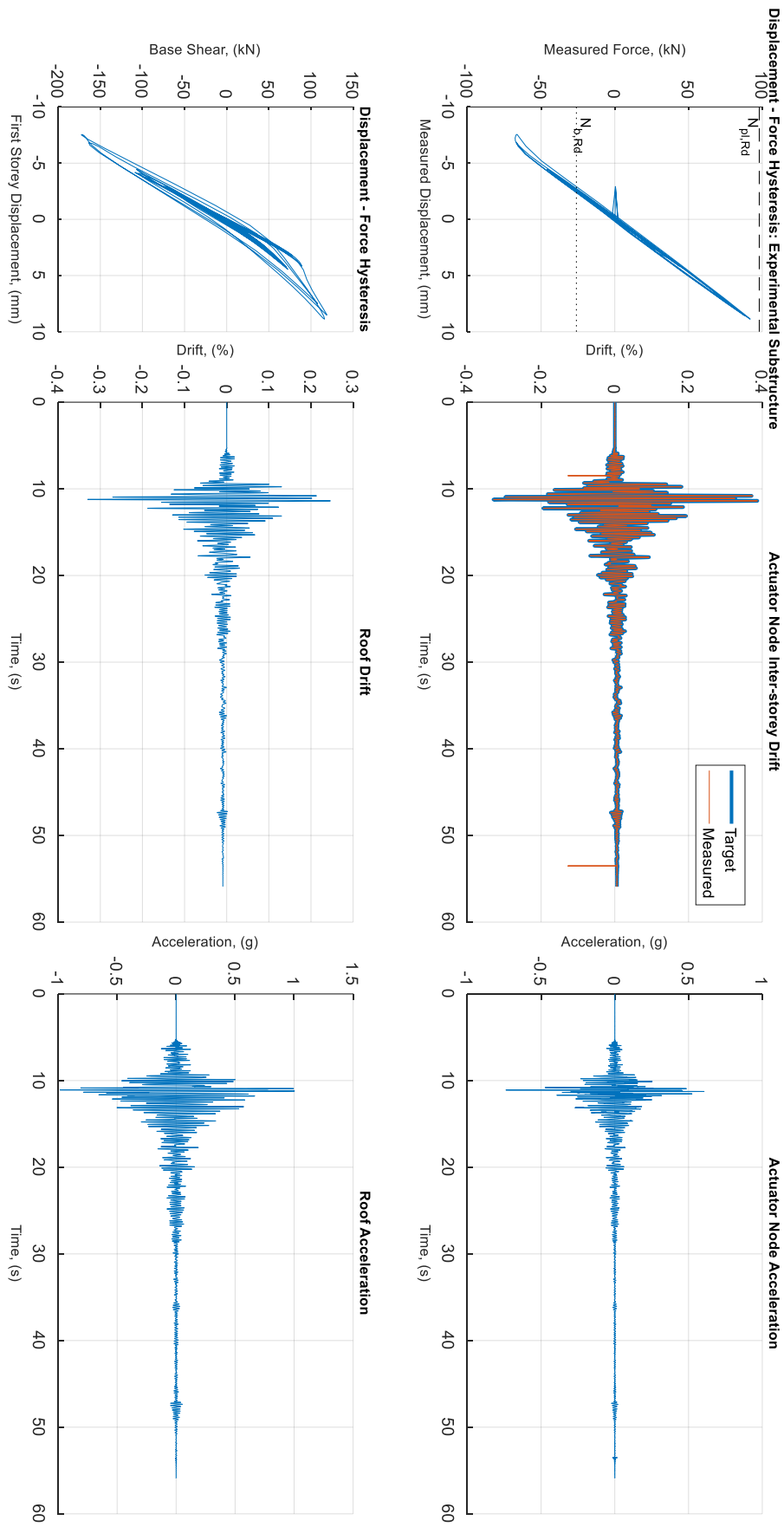


Figure 9-19 Experimental response for the SS-GM3-10 simulation

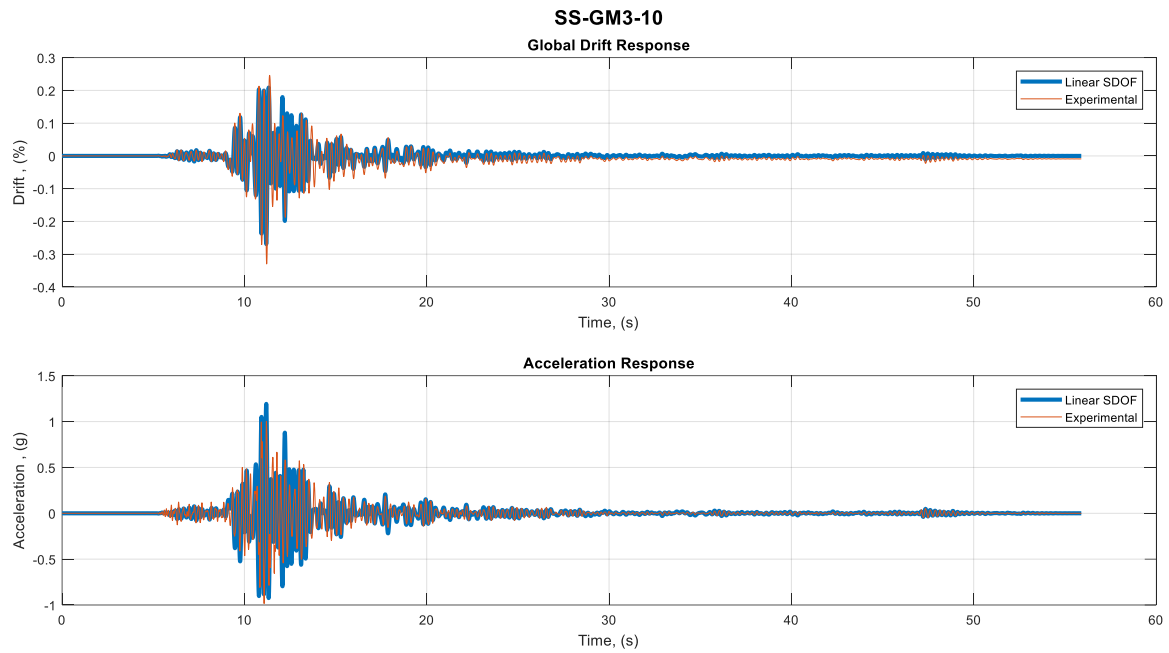


Figure 9-20 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the SS-GM3-10 test run

9.3.1.2.4 NSS-GM4-10

Figure 9-21 shows the results of the NSS-GM4-10 simulation. Peak drift demand is approximately 0.4% in both directions. Brace buckling was noted during the simulation, as would be expected given the drift demand, and is apparent in the hysteresis plot for the experimental substructure.

As in SS-GM3-10, there hysteresis loop is asymmetrical suggesting greater nonlinear behaviour in the numerical substructure.

Peak floor acceleration demands are similar to the other simulations at this intensity level, with values of approximately 0.6g and 1.2g obtained at the first floor level and roof level respectively.

NSS-GM4-10

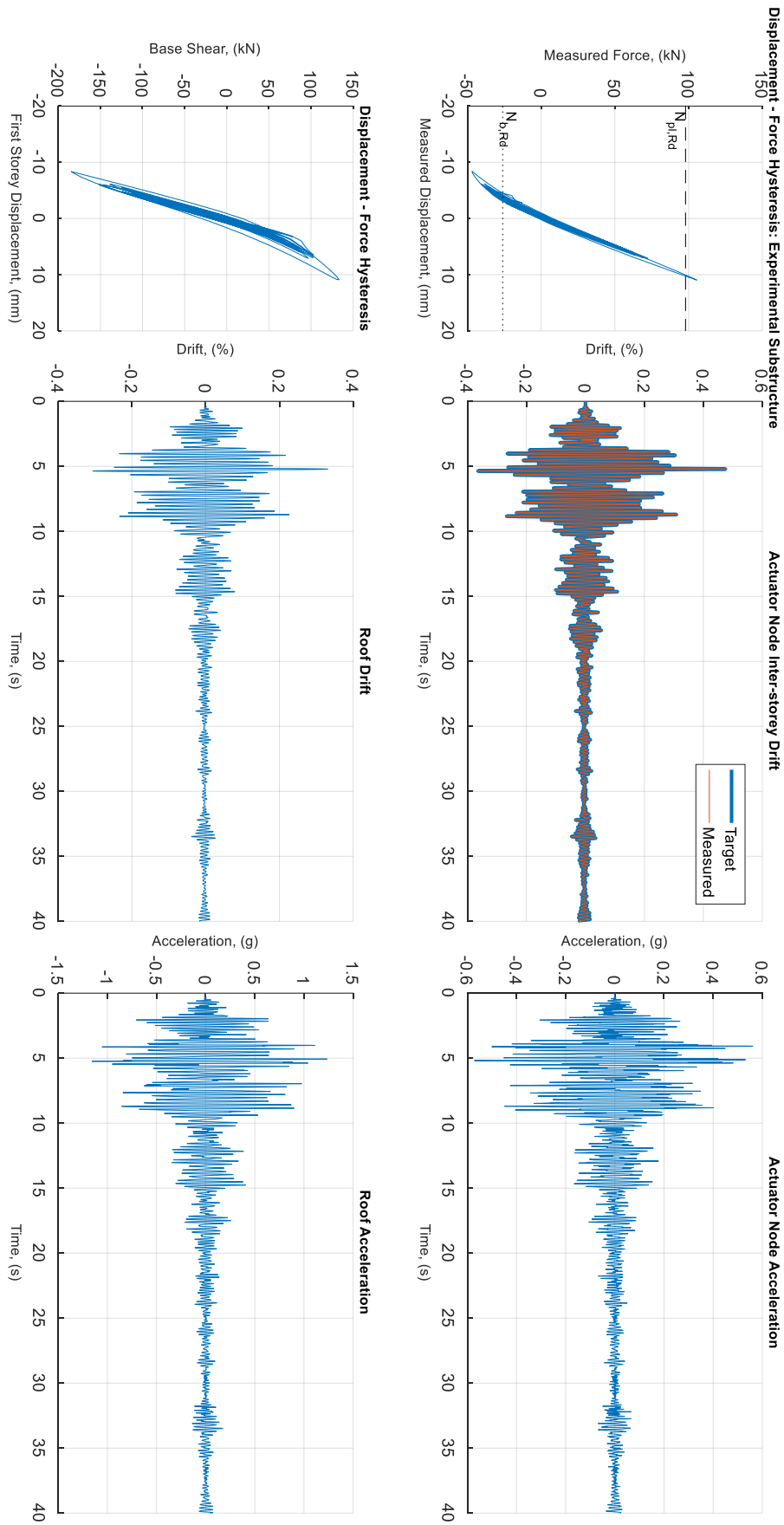


Figure 9-21 Experimental response for the NSS-GM4-10 simulation

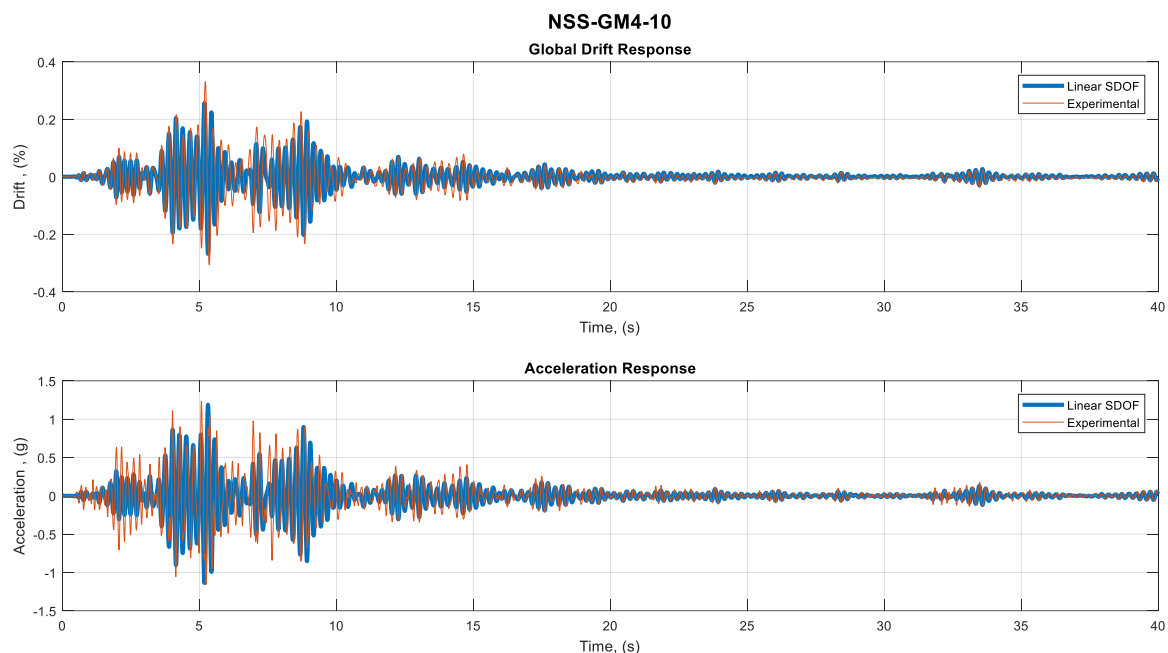


Figure 9-22 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM4-10 test run

9.3.1.2.5 NSS-GM5-10

The key trends in NSS-GM5-10, presented in Figure 9-23, are reasonably similar to those discussed previously. The peak drift demand for the experimental substructure is approximately 0.3%. Some nonlinear behaviour is apparent in the hysteresis plot, however this is possibly partly attributable to the load history of the brace specimen at this point in the test programme.

As before, an asymmetrical hysteresis loop is obtained when the numerical substructure is included despite relatively similar demands in both directions; again this is attributable to the greater degree of nonlinear behaviour exhibited by the numerically modelled brace.

Peak acceleration demands are slightly higher than in the other 10% in 50 year simulations, with a peak value of almost $1.5g$ at the roof and $0.8g$ at the first floor level. This is not reflected in the response of the linear SDOF system, which is generally less than the hybrid simulation response, unlike the other 10% in 50 year tests. The applied ground motion used in this test was a scaled version of the record used in NSS-GM5-50 in which a similarly large peak roof acceleration was noted.

NSS-GM5-10

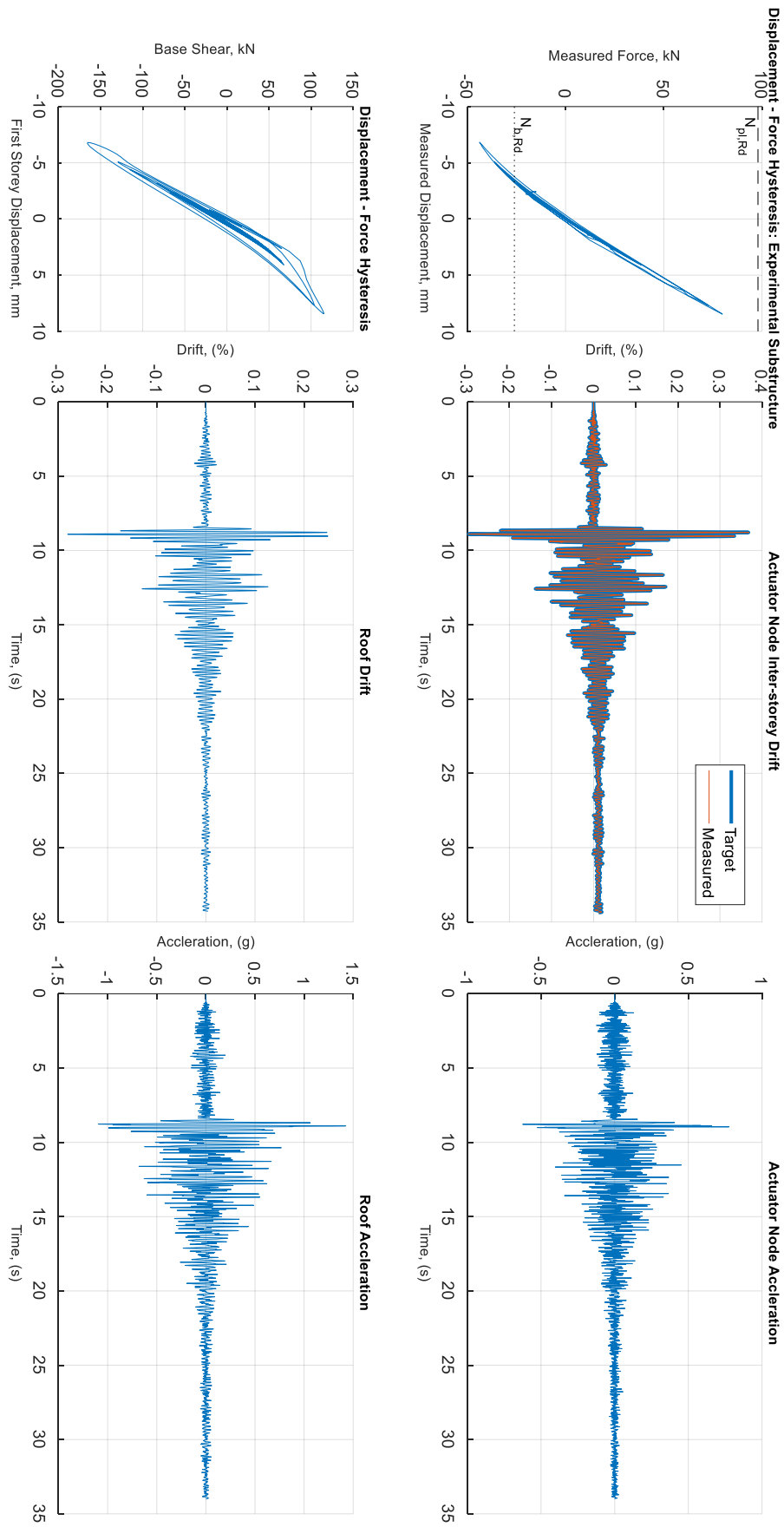


Figure 9-23 Experimental response for the NSS-GM5-10 simulation

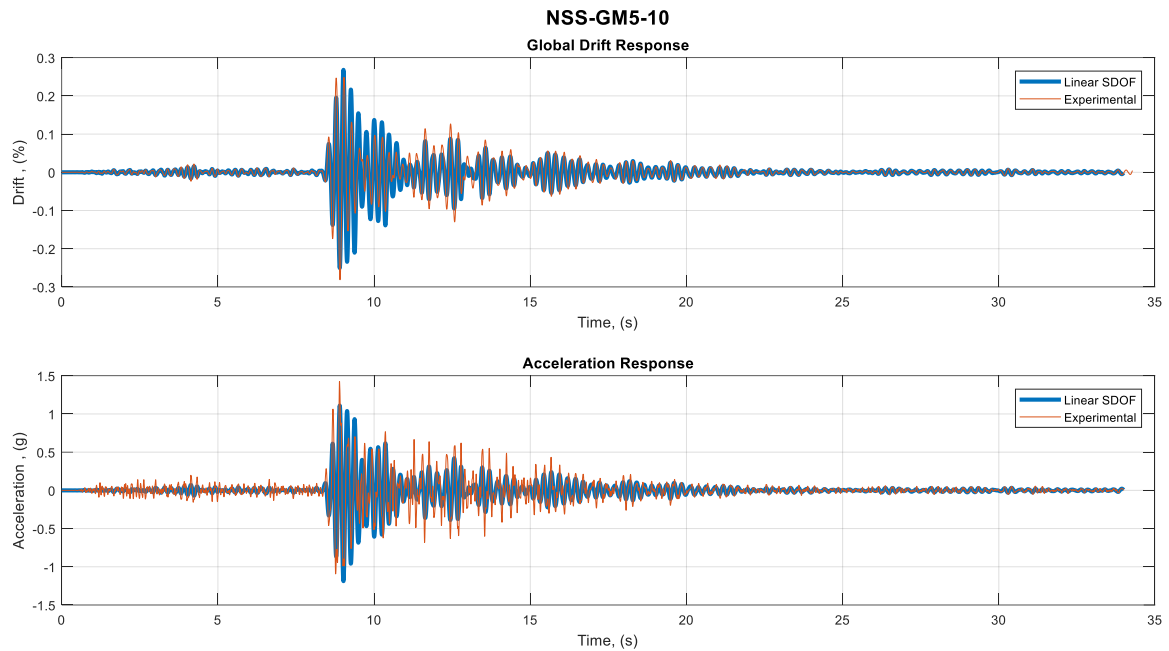


Figure 9-24 Comparison of global drift and roof acceleration from the hybrid simulation with a linear SDOF system for the NSS-GM5-10 test run

Table 9-4 summarises the key experimentally recorded data for the experimental substructure in each of the hybrid simulations performed. Ductility demand was estimated for both displacement in the positive direction (i.e. with the experimental brace in tension) and in the negative direction (with the experimental brace in compression) in accordance with the procedure outlined in ECCS 42 (ECCS, 1986), where the yield displacement is estimated as the intersection of the line defining the initial stiffness passing through the origin and a line with 1/10 the slope of the initial stiffness passing through the point of maximum force on the force-displacement plot.

The initial stiffness of the experimental substructure was measured as slightly less than 11kN/mm. The value of initial stiffness obtained through a pushover analysis of a numerical model of the experimental substructure was approximately 15 kN/mm (see Figure 10-3 (e)), which, clearly, is greater than that measured in the experiment. The reasons for this are discussed further in Chapter 10. It can also be noted from this table that for the 10% in 50 year records the initial stiffness of the experimental substructure decreases as the tests proceed (the tests were run in order from SS-GM1-10 to NSS-GM5-10), suggesting that a certain amount of softening behaviour occurs with the load history affecting the characteristics of the experimental substructure.

Table 9-4 Summary of response of experimental substructure

Test ID	Peak Displacement (mm)	Peak Drift (%)	Peak Force (kN)	Initial Stiffness (kN/mm)	Positive Ductility Demand	Negative Ductility Demand
SS-GM1-50	4.06	0.18	43.44	10.79	1.01	0.99
SS-GM2-50	3.53	0.15	36.06	10.78	1.00	0.99
SS-GM3-50	3.99	0.17	42.90	10.70	1.00	0.99
NSS-GM4-50	2.96	0.12	30.39	10.73	1.00	0.98
NSS-GM5-50	4.30	0.19	45.76	10.72	1.01	0.98
SS-GM1-10	9.47	0.41	96.05	10.40	1.03	1.02
SS-GM2-10	11.83	0.52	107.71	10.71	1.20	0.92
SS-GM3-10	8.88	0.39	91.85	10.43	1.00	1.17
NSS-GM4-10	10.90	0.48	105.52	9.60	1.00	1.87
NSS-GM5-10	8.44	0.37	79.82	9.46	1.00	1.55

2% in 50 Year Runs

As previously stated, it was initially intended to perform further hybrid simulations for ground motions with a 2% in 50 year probability of exceedance. However, prior to doing this, a component in the test system was damaged meaning it was not possible to complete the tests before the end of this research.

The nature of the damage to the test system is highlighted in Figure 9-25. The problem area is in the communication between the servo-controller and the actuator; hence it was not possible to impose any displacement on the experimental substructure. After inspection from the manufacturers, MTS, the card that controls this communication was found to be damaged. The card used in the test system at TCD is no longer produced, hence it was not possible to simply replace the damaged part and resume testing. At the time of writing MTS are attempting to source an existing card from a previously developed system. Failing this, the entire servo-controller will have to be replaced.

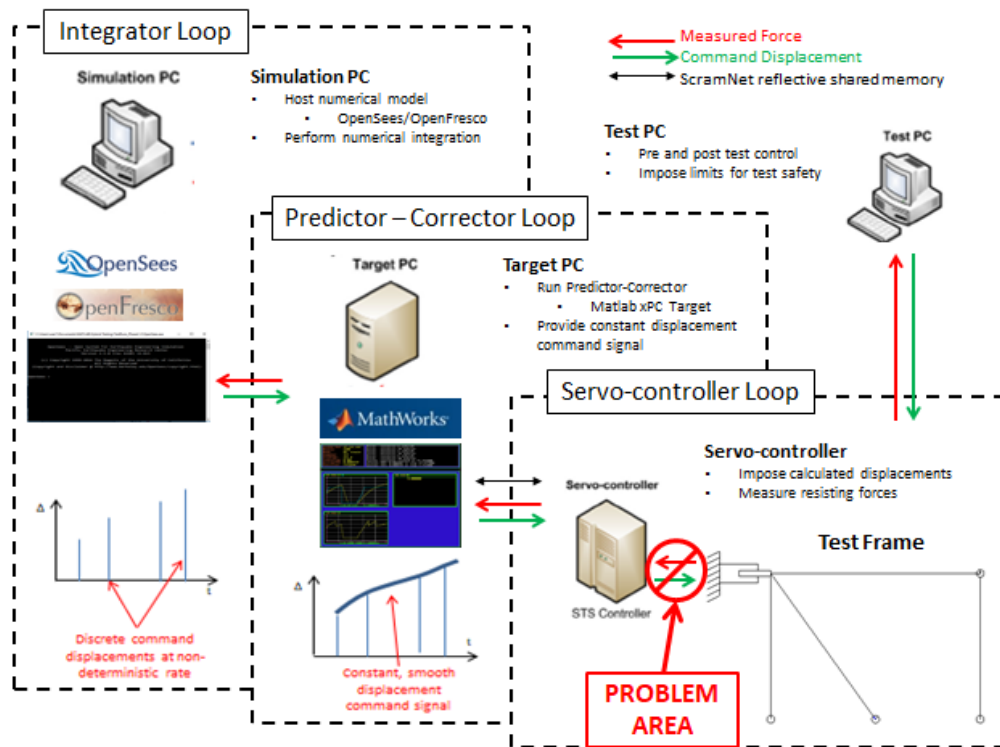


Figure 9-25 Illustration of the damaged component in the test system

9.3.2 Comments & Trends

A number of general trends are evident from examination of the experimental data. Firstly, it appears that using brace fragility functions and inter-storey drift demand is a more reliable method of predicting brace buckling than buckling loads predicted by Eurocode 3. This is firstly a result of the assumed end conditions as well as being related to the assumed out-of-straightness in EC3. The EC8 buckling load was exceeded in all simulations, however buckling only occurred when peak drift in the negative direction exceeded 0.3%, which is in line with the values predicted by the fragility functions proposed in FEMA P-58 (FEMA, 2012a) or by Lignos and Karamanci (2013).

A second trend repeated throughout the simulations is that the bracing members in the numerical substructure appear to display greater energy dissipation than the experimentally tested brace sample. This suggests that improved results may be obtained with an improved model of the bracing element. It is possible that this can be achieved through model updating routines to optimize parameters in the numerical substructure based on the response of the experimental substructure.

9.4 Comparison with Numerical Models

A fully numerical model of the test structure was developed in OpenSees. This was identical to the model used for the numerical substructure previously described, but with the experimental substructure replaced by further numerical elements.

The following figures present the results of the hybrid simulations and compare them with results from this fully numerical model. For each test a hysteresis plot of base shear against first storey displacement is shown to indicate whether the experimental substructure is behaving linearly or not. The peak inter-storey drift profile over the height of the structure, as well as the displacement and acceleration time histories at the actuator node and the roof are also presented.

SS-GM1-50

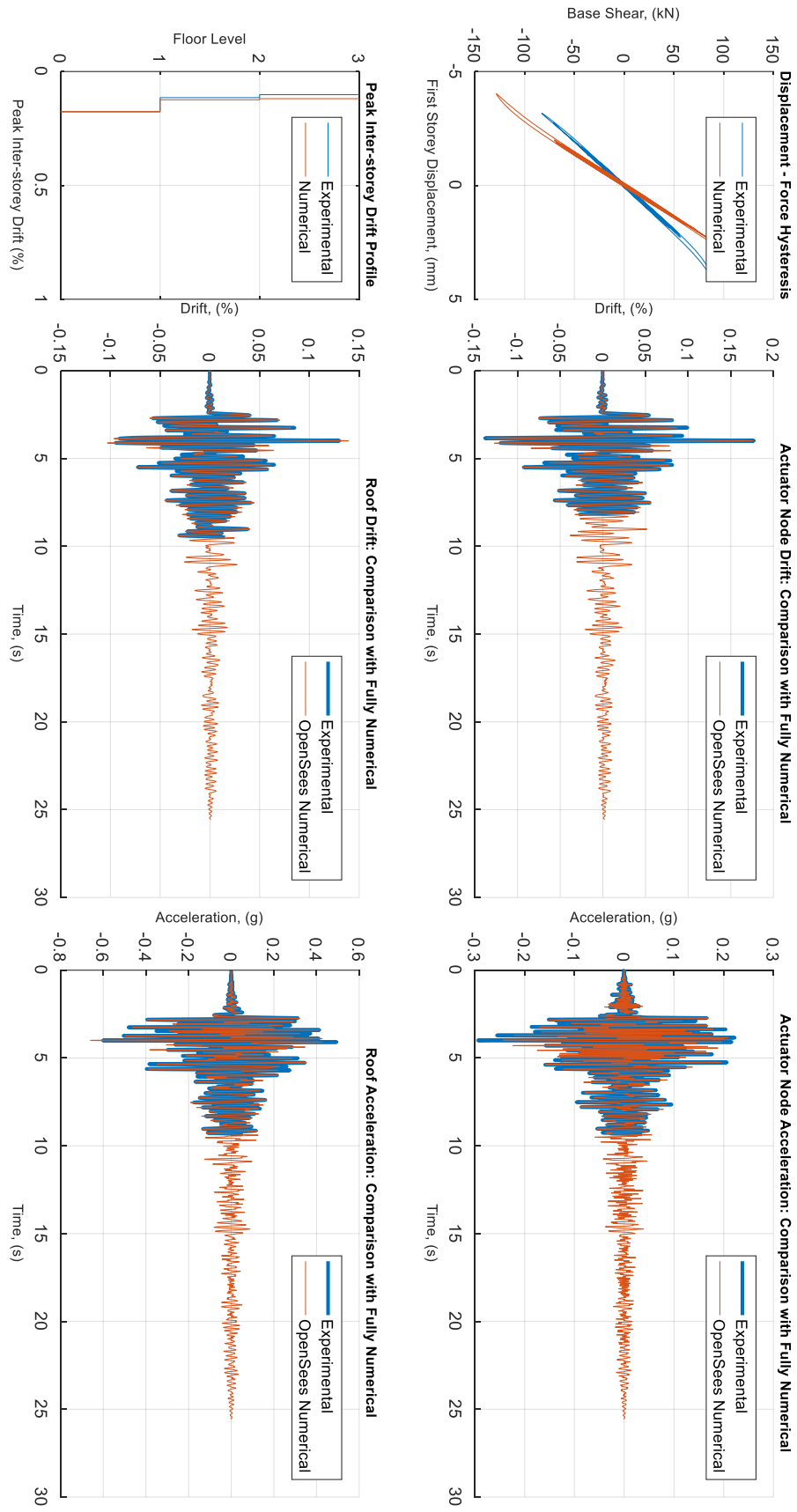


Figure 9-26 SS-GM1-50: Comparison between numerical model and hybrid test results

SS-GM2-50

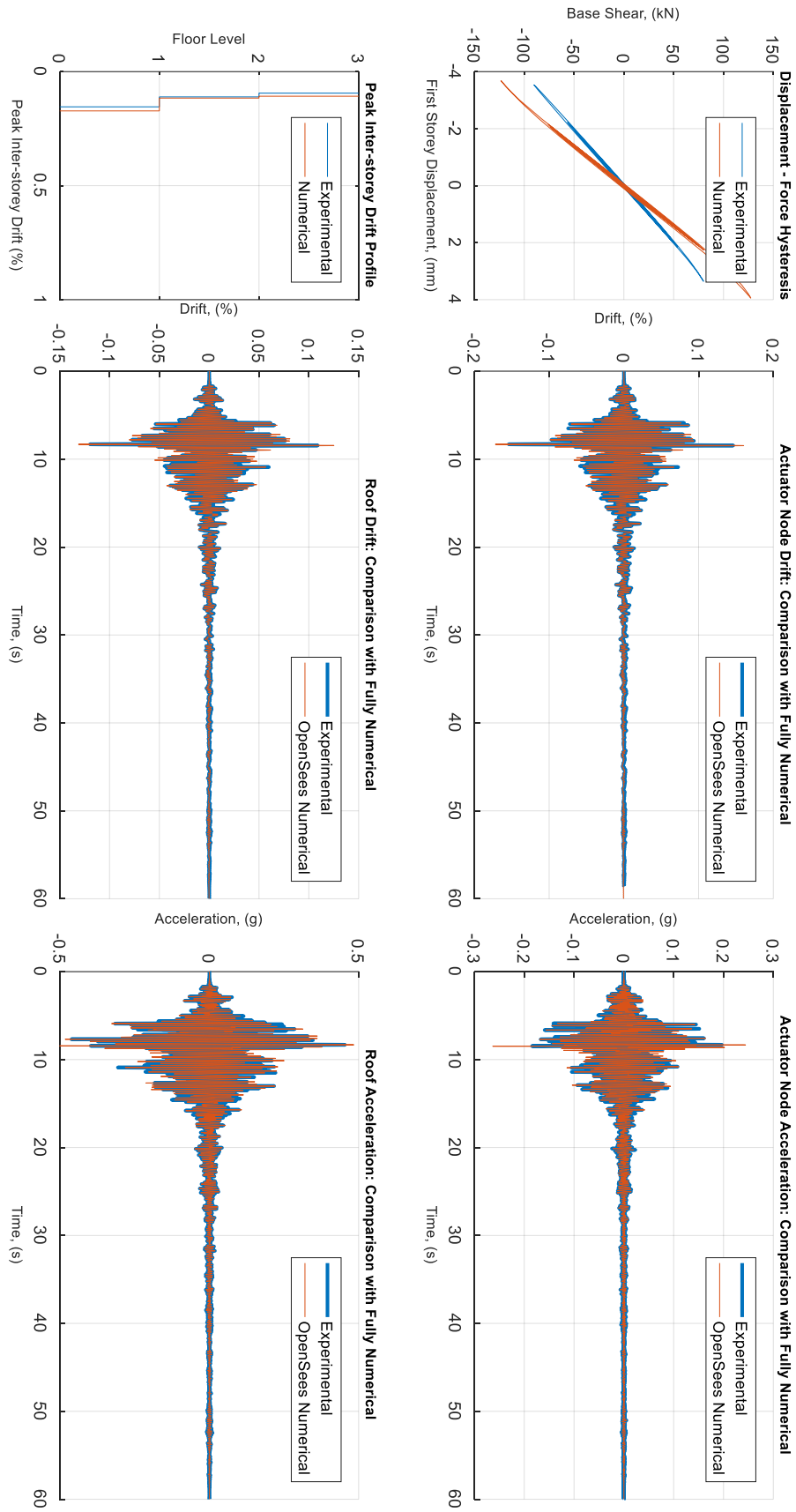


Figure 9-27 SS-GM2-50: Comparison between numerical model and hybrid test results

SS-GM3-50

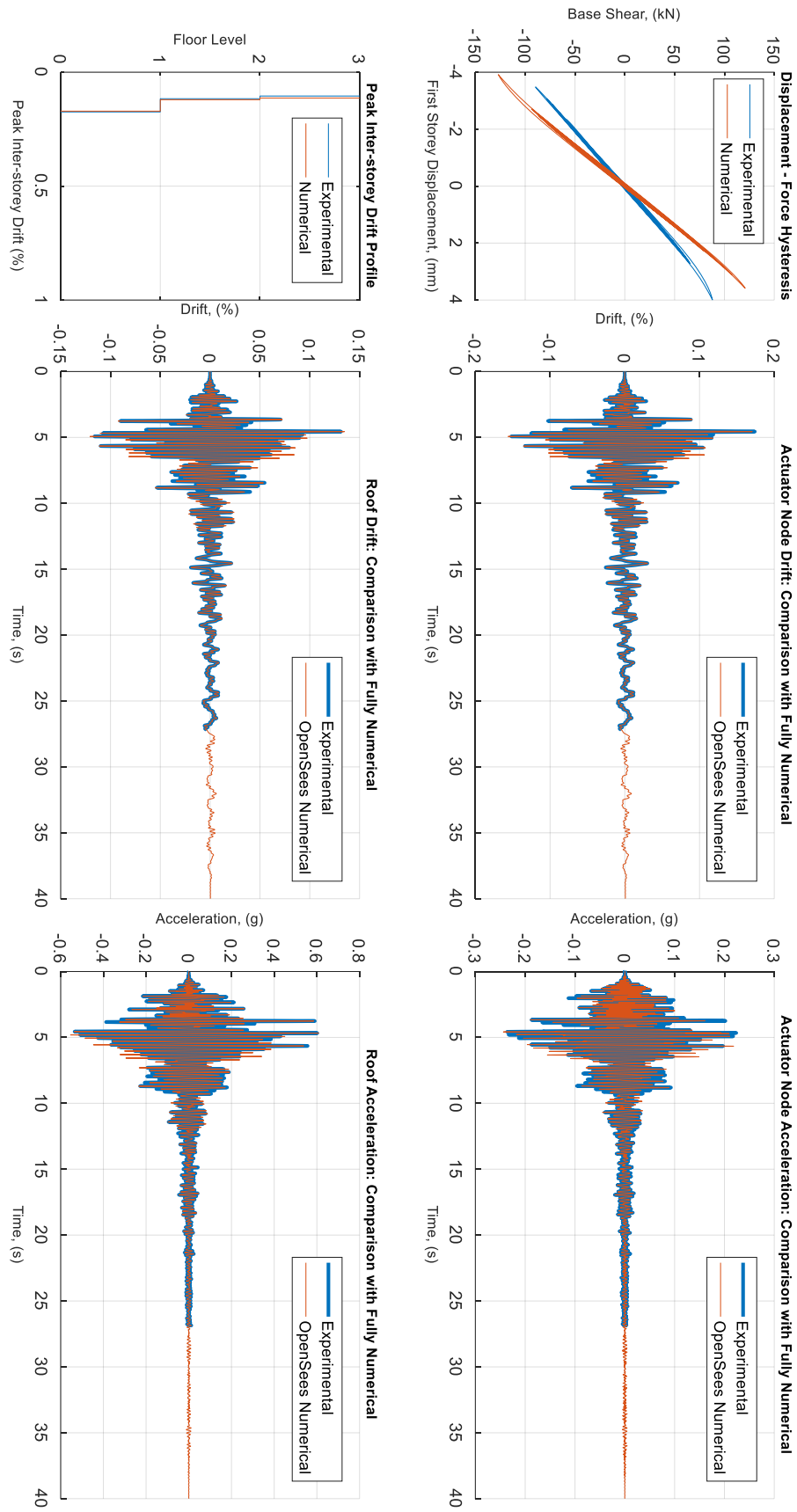


Figure 9-28 SS-GM3-50: Comparison between numerical model and hybrid test results

NSS-GM4-50

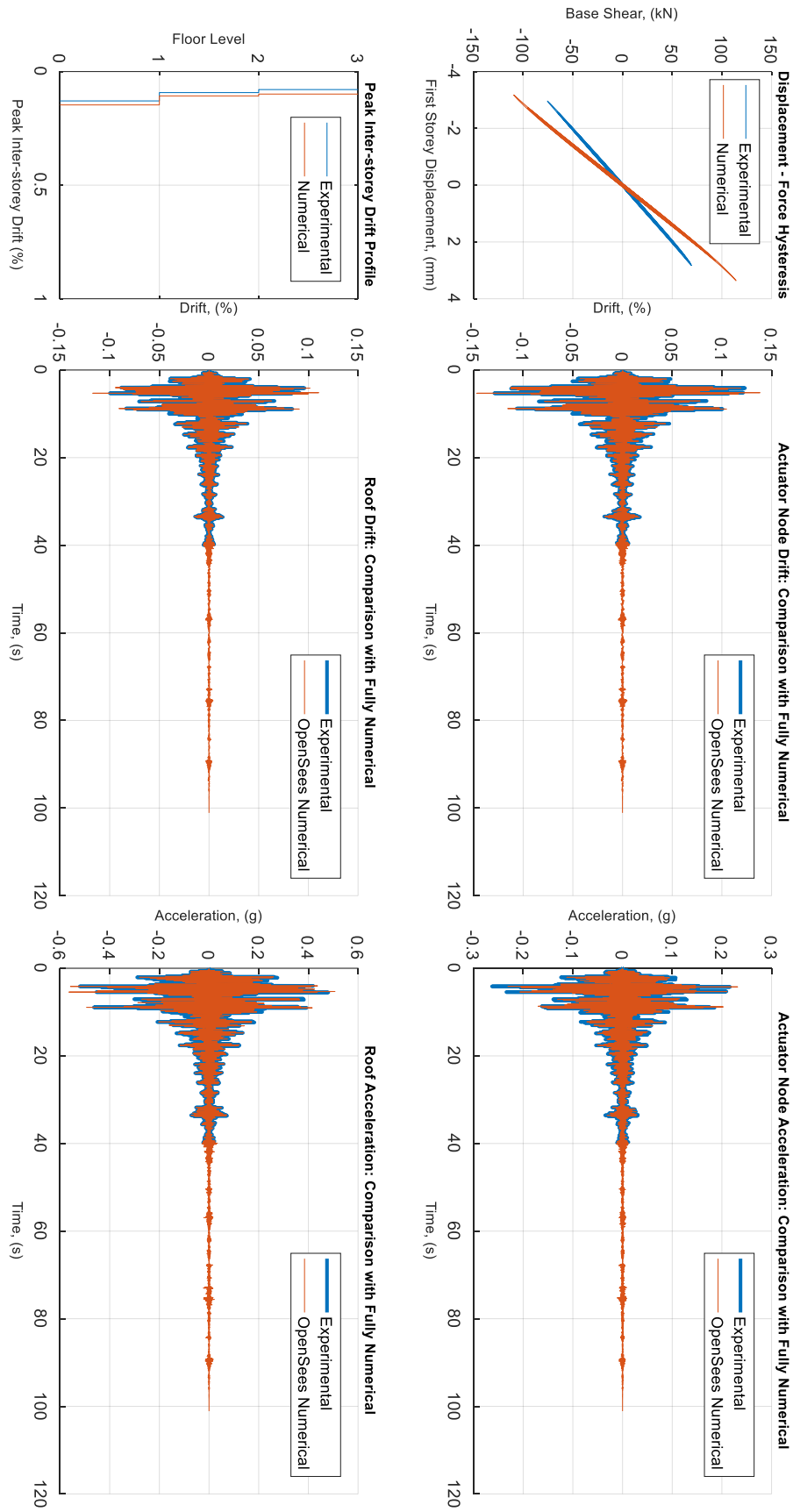


Figure 9-29 NSS-GM4-50: Comparison between numerical model and hybrid test results

NSS-GM5-50

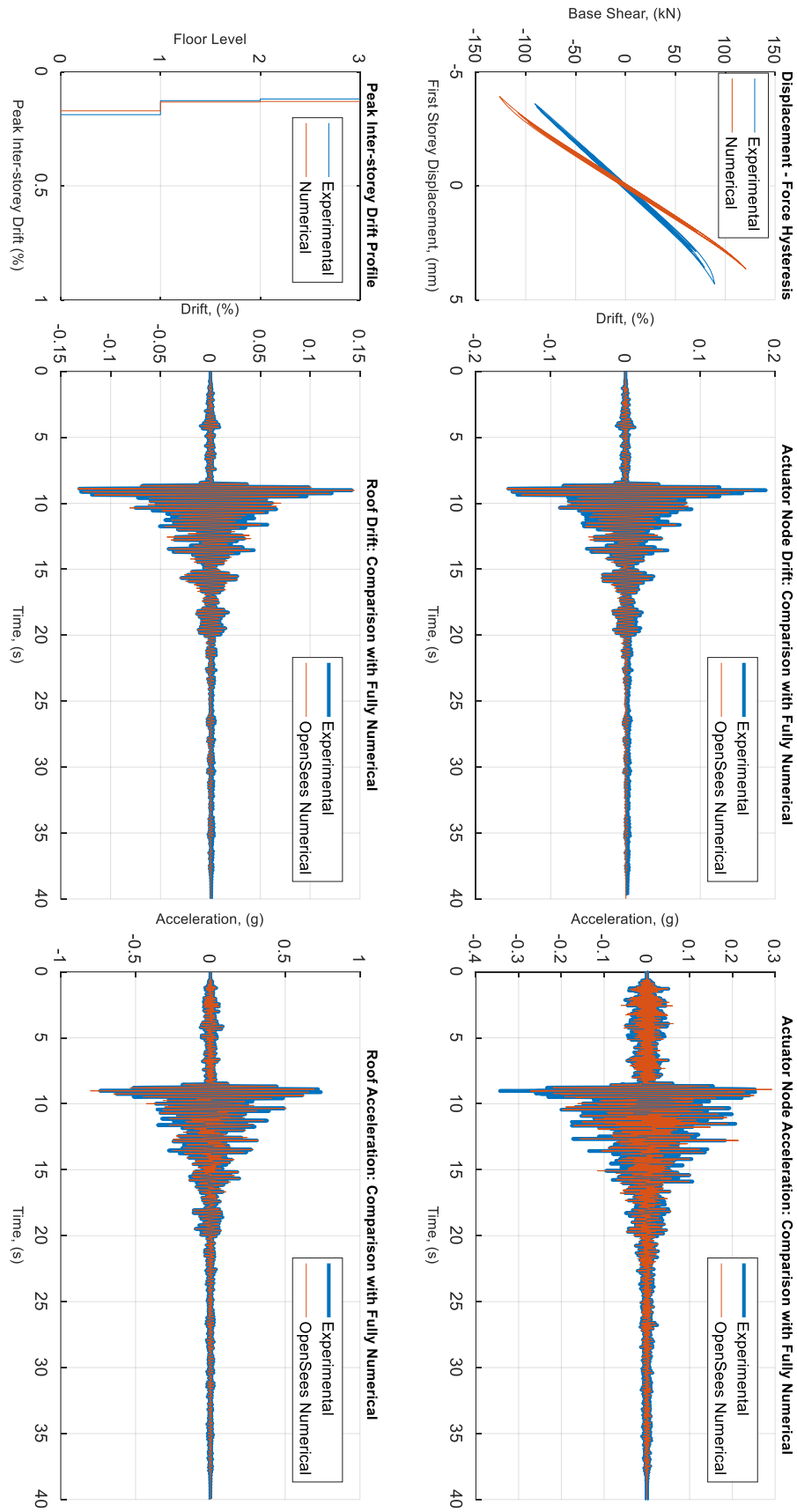


Figure 9-30 NSS-GM5-50: Comparison between numerical model and hybrid test results

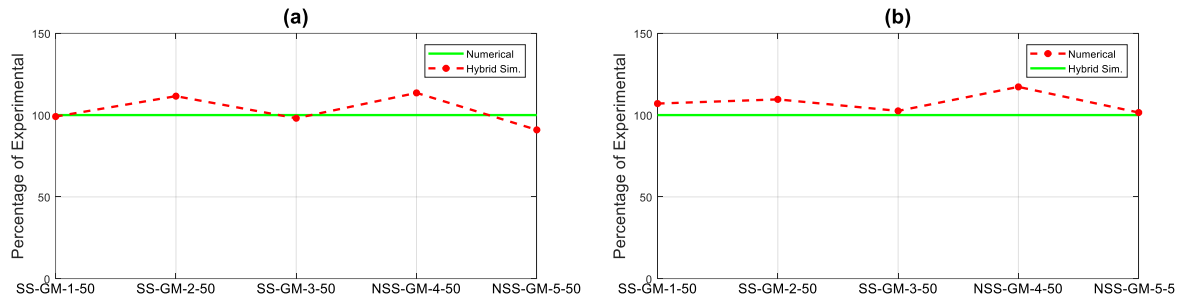


Figure 9-31 Comparison between hybrid simulation and numerical results for peak displacement for 50% in 50 year intensity level ground motions for (a) Level 1, i.e. the location of the actuator, and (b) roof level

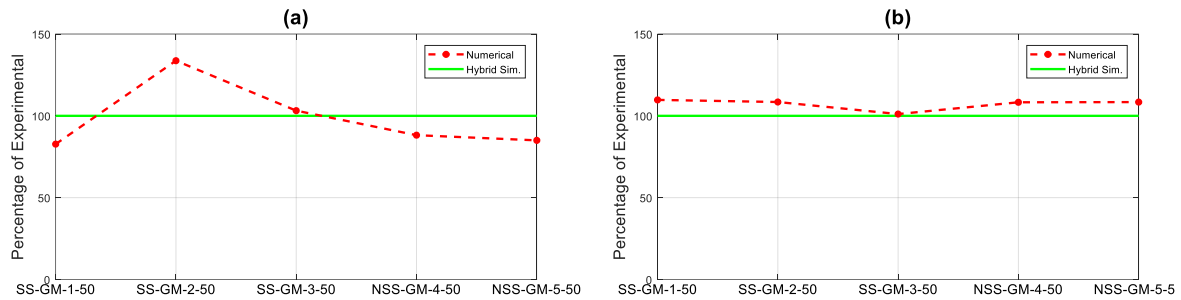


Figure 9-32 Comparison between hybrid simulation and numerical results for peak acceleration for 50% in 50 year intensity level ground motions for (a) Level 1, i.e. the location of the actuator, and (b) roof level

Table 9-5 Comparison of *EDPs* from experimental and numerical analyses

Ground Motion ID	Δ_{Level_1} (mm)		Δ_{roof} (mm)		a_{Level_1} (g)		a_{roof} (g)	
	Experiment	Numerical	Experiment	Numerical	Experiment	Numerical	Experiment	Numerical
SS-GM1-50	4.1	4.0	9.0	9.6	0.29	0.24	0.60	0.66
SS-GM2-50	3.5	3.9	8.2	9.0	0.20	0.26	0.46	0.50
SS-GM3-50	4.0	3.9	9.0	9.3	0.24	0.24	0.60	0.61
NSS-GM4-50	3.0	3.4	6.9	8.1	0.26	0.23	0.52	0.56
NSS-GM5-50	4.3	3.9	9.8	9.9	0.34	0.29	0.74	0.80

Table 9-6 *EDPs* from numerical analyses expressed as a percentage of experimental value

Ground Motion ID	Δ_{Level_1}	Δ_{roof}	a_{Level_1}	a_{roof}
SS-GM1-50	99.1	107.0	82.72	109.74
SS-GM2-50	111.6	109.6	133.70	108.45
SS-GM3-50	98.1	102.6	103.17	101.09
NSS-GM4-50	113.5	117.2	88.18	108.28
NSS-GM5-50	91.0	101.6	85.01	108.40

Figures 9-26 to 9-30 present the results for the hybrid simulations performed at the 50% in 50 year probability of exceedance ground motion intensity. It can be seen from the hysteresis loops

that behaviour is generally elastic for these tests. The drift and acceleration time-histories from the hybrid simulations are broadly similar to those obtained from the purely numerical model.

This is further illustrated in Figures 9-31 and 9-32, where the peak values of drift and acceleration at the roof levels and actuator node from numerical modelling are compared with those from the hybrid simulations. It can be seen that these peak values, which are the values employed in performance assessment, are generally very similar in the purely numerical and hybrid simulations. The *EDPs* recorded in the experimental and numerical analyses are given in Table 9-5, while Table 9-6 presents the numerical *EDPs* as a percentage of the experimental values.

There are however noticeable differences in the force-displacement hysteresis loops obtained numerically and experimentally. Whilst both demonstrate linear behaviour, the numerical model can be seen to be consistently stiffer than the experimental test. Potential reasons for this include errors in the modelling of boundary conditions, connections and material properties. These are investigated further in Chapter 10.

SS-GM1-10

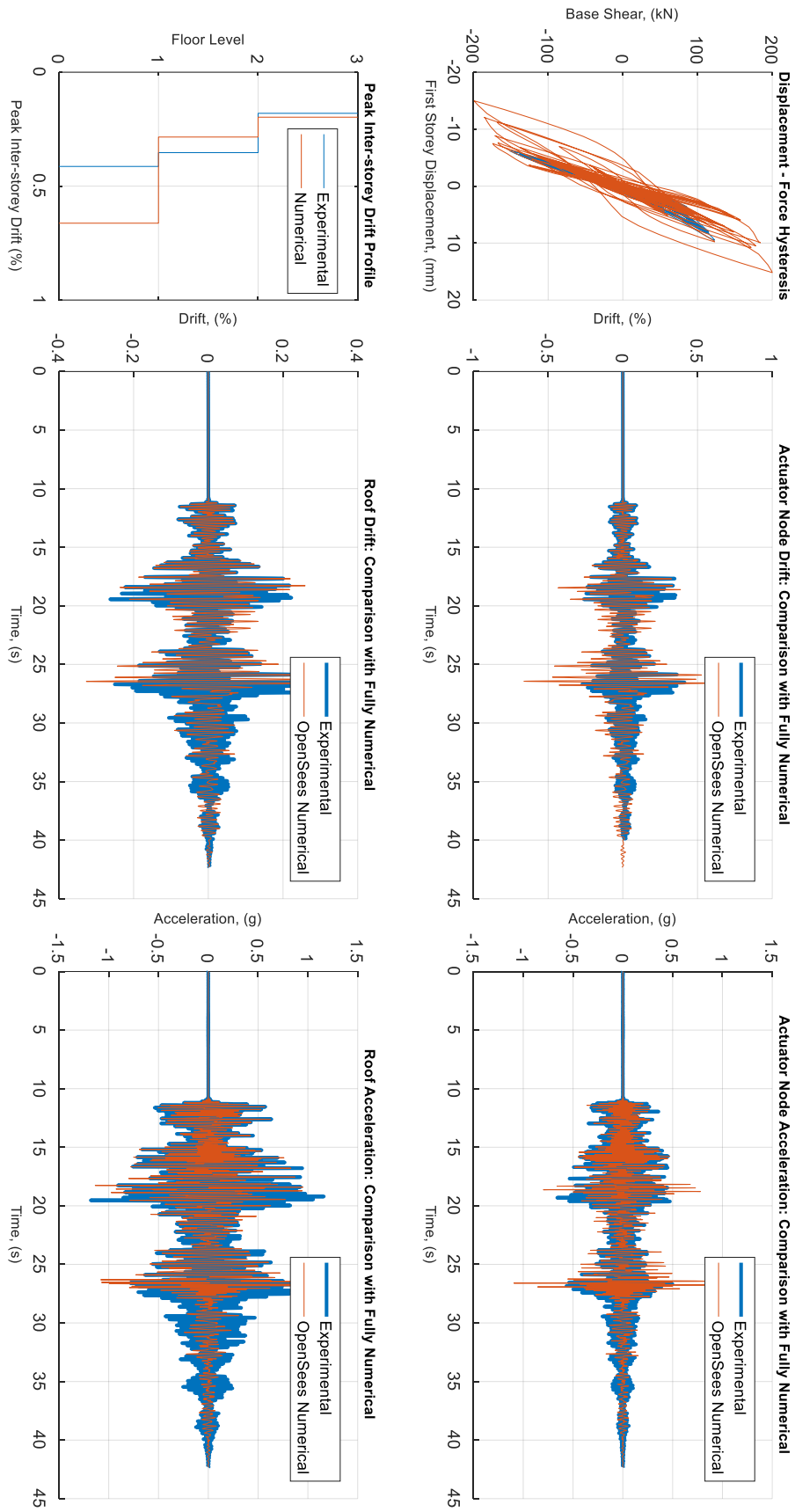


Figure 9-33 SS-GM1-10: Comparison between numerical model and hybrid test results

SS-GM2-10

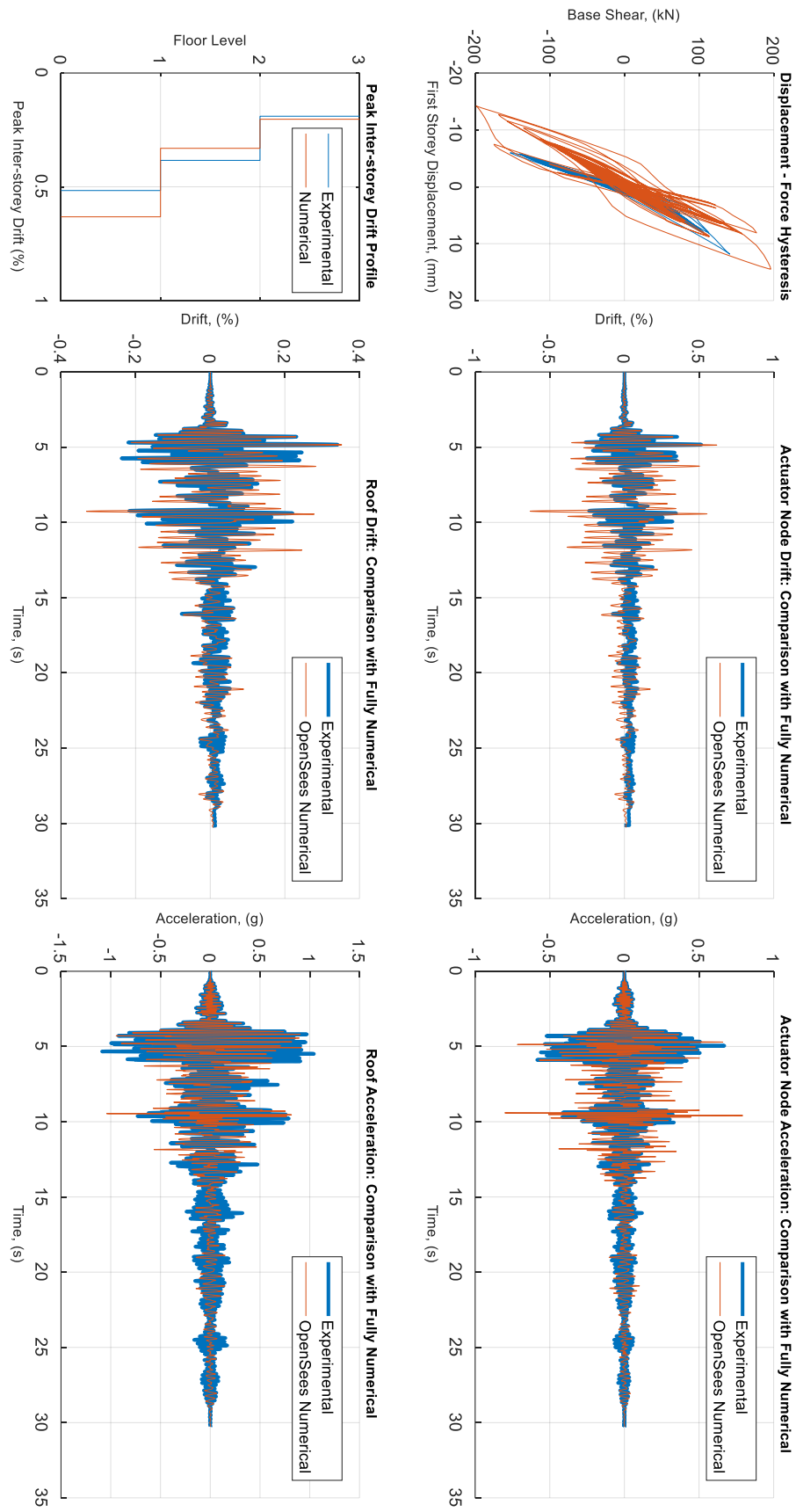


Figure 9-34 SS-GM2-10: Comparison between numerical model and hybrid test results

SS-GM3-10

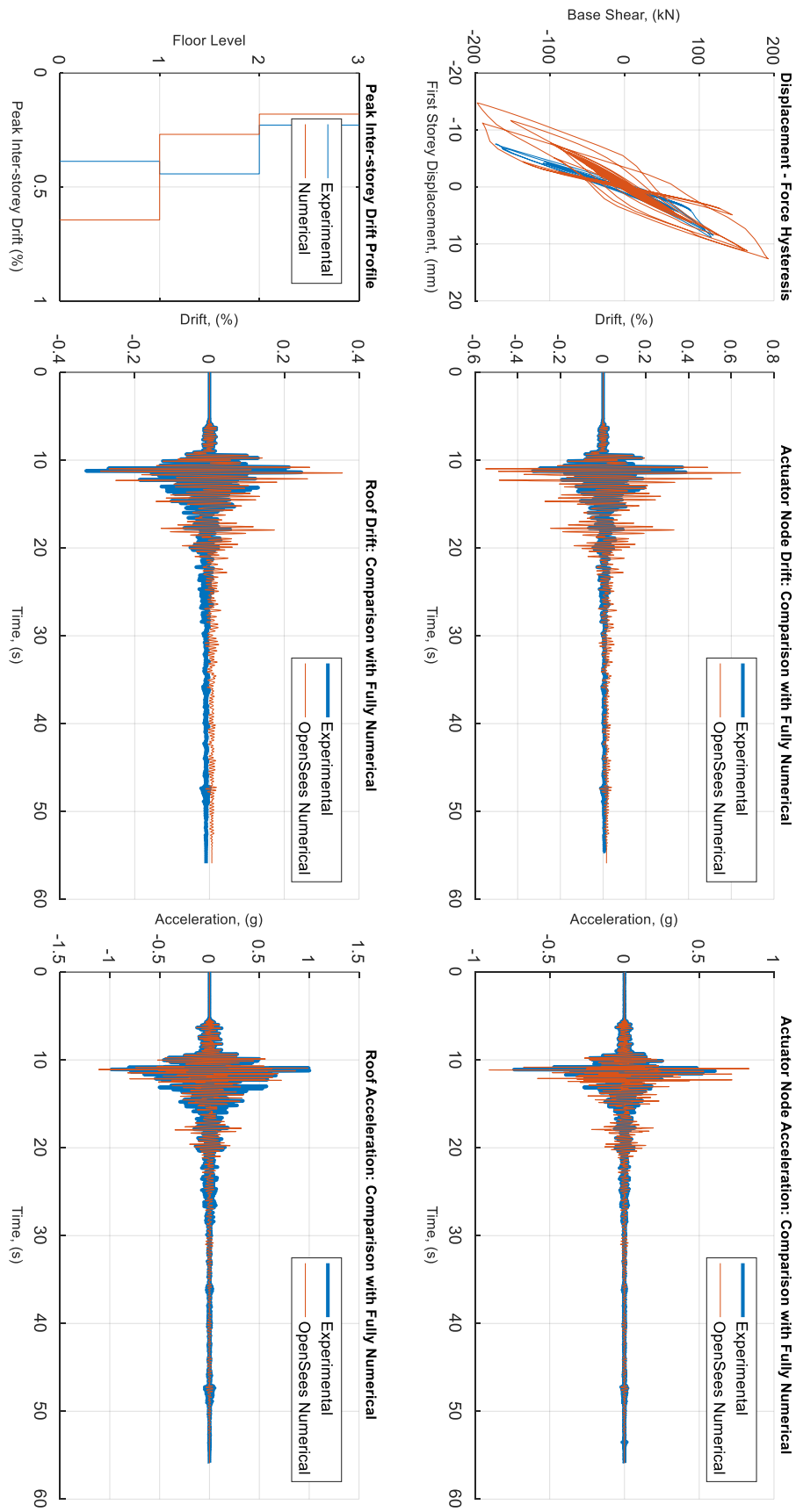


Figure 9-35 SS-GM3-10: Comparison between numerical model and hybrid test results

NSS-GM4-10

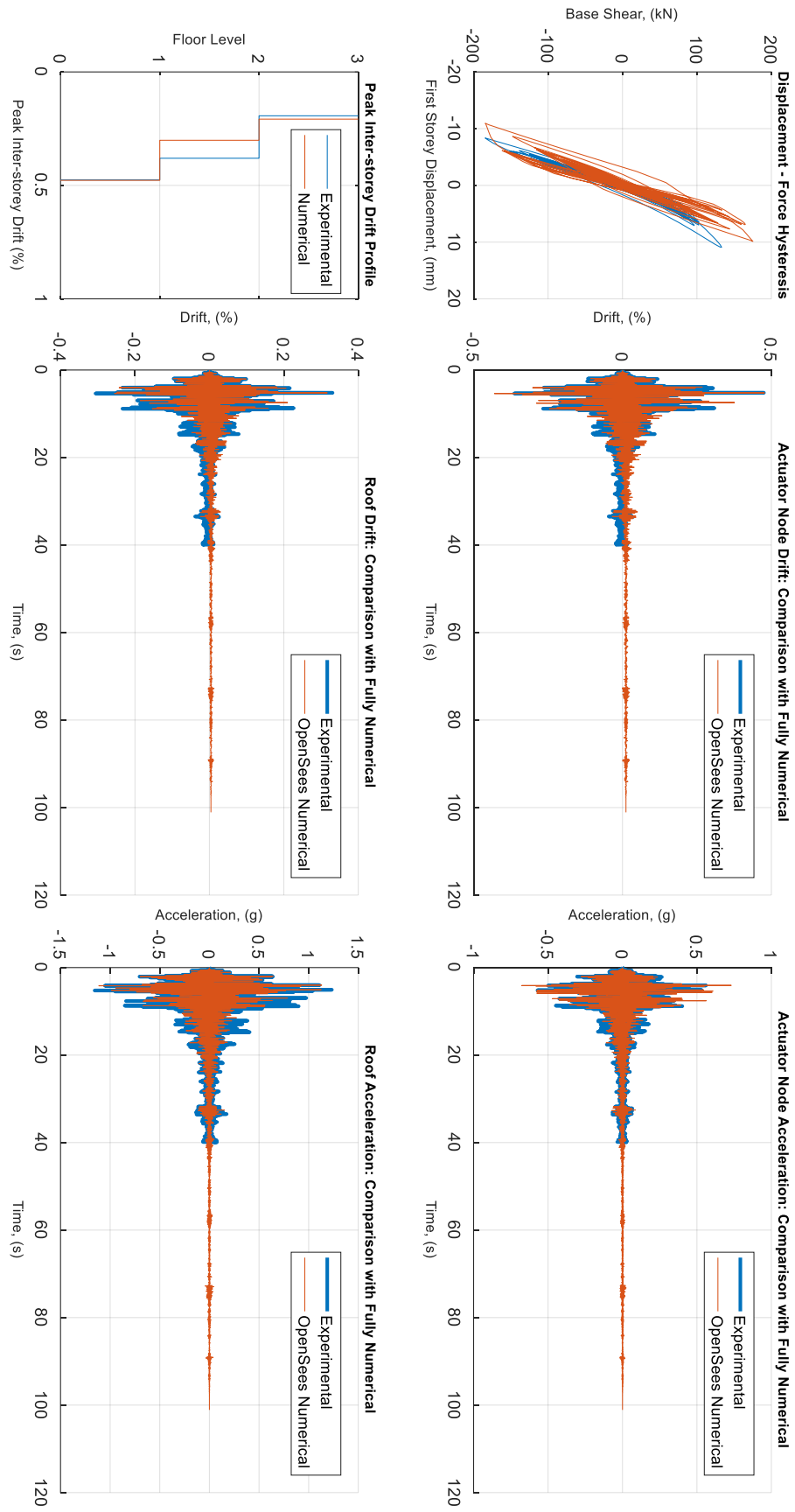


Figure 9-36 NSS-GM4-10: Comparison between numerical model and hybrid test results

NSS-GM5-10

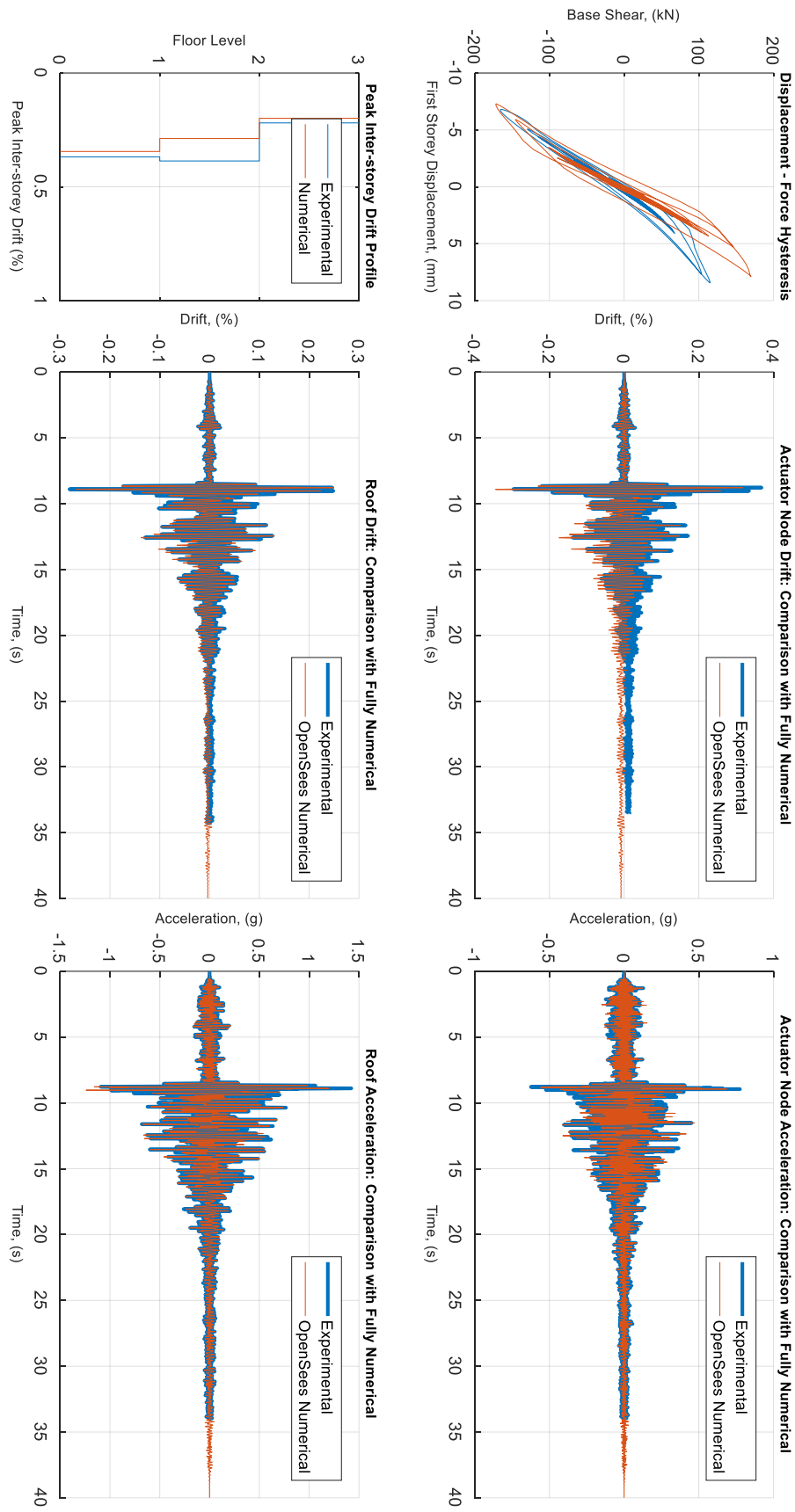


Figure 9-37 NSS-GM5-10: Comparison between numerical model and hybrid test results

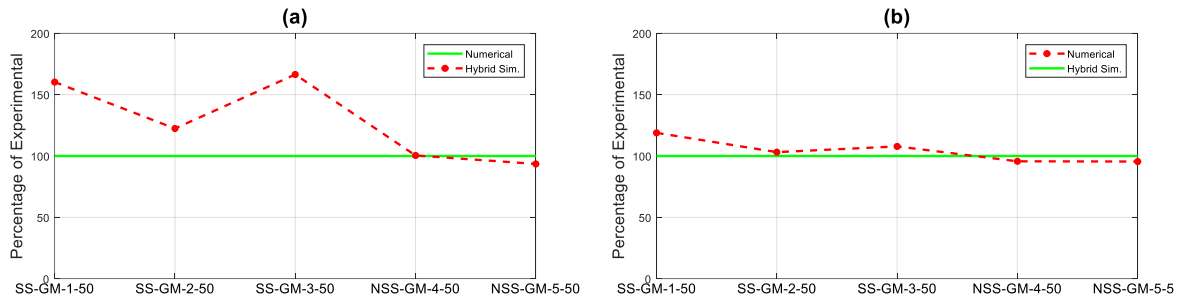


Figure 9-38 Comparison between hybrid simulation and numerical results for peak displacements for 10% in 50 year intensity level ground motions for (a) actuator node and (b) roof level

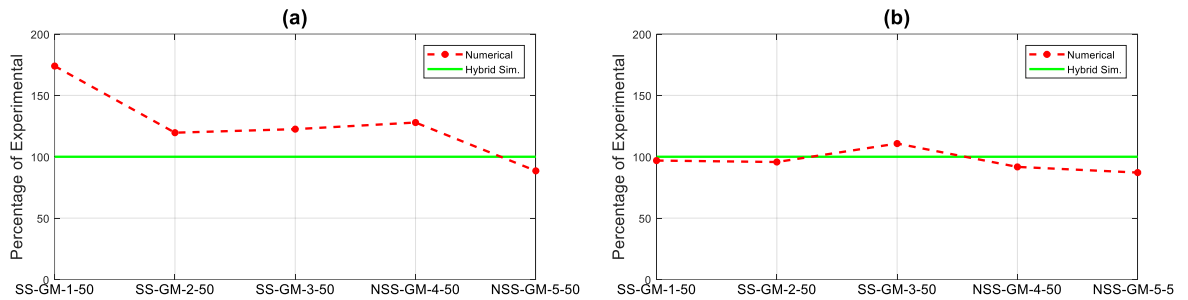


Figure 9-39 Comparison between hybrid simulation and numerical results for peak acceleration for 10% in 50 year intensity level ground motions for (a) actuator node and (b) roof level

Table 9-7 Comparison of EDPs from experimental and numerical analyses

Ground Motion ID	Δ_{actuator} (mm)		Δ_{roof} (mm)		a_{actuator} (g)		a_{roof} (g)	
	Experiment	Numerical	Experiment	Numerical	Experiment	Numerical	Experiment	Numerical
SS-GM1-10	9.471	15.176	18.921	22.490	0.659	1.146	1.180	1.143
SS-GM2-10	11.825	14.473	23.499	24.236	0.670	0.801	1.088	1.041
SS-GM3-10	8.875	14.767	22.701	24.471	0.738	0.904	1.003	1.111
NSS-GM4-10	10.897	10.943	22.776	21.789	0.571	0.731	1.234	1.132
NSS-GM5-10	8.442	7.895	19.362	18.486	0.776	0.687	1.424	1.240

Table 9-8 EDPs from numerical analyses expressed as a percentage of experimental value

Ground Motion ID	Δ_{actuator}	Δ_{roof}	a_{actuator}	a_{roof}
SS-GM1-10	160.23	118.86	173.89	96.89
SS-GM2-10	122.39	103.14	119.60	95.71
SS-GM3-10	166.39	107.80	122.49	110.74
NSS-GM4-10	100.43	95.67	127.88	91.73
NSS-GM5-10	93.53	95.47	88.51	87.10

Figures 9-33 to 9-37 present the results of the hybrid simulations for the 10% in 50 year probability of exceedance ground motion intensity, while Figures 9-38 and 9-39 compare the

peak acceleration and drift responses of the 5 ground motions from the hybrid simulations and numerical models. It can be seen from the hysteresis loops that behaviour is generally inelastic for these tests. Comparing the hybrid simulation and numerical time histories, both the drift and acceleration response at roof level is generally very similar. However, there are differences between the responses at the actuator node; generally the numerical models predict a greater level of displacement and acceleration than observed in the hybrid simulations. Further examinations of the time-histories at this level show a close match while the response is relatively small, i.e. while the frame is still behaving elastically. Divergence between the numerical and hybrid simulation time histories appears to occur at greater response levels, i.e. when significant buckling or yielding occurs. This is particularly noticeable in the SS-GM1-10, SS-GM2-10 and SS-GM3-10 tests. For these, as can be seen from the inter-storey drift profiles, the drifts from the numerical model tend to be concentrated at the lowest story level, whereas a more even distribution of drift over the frame height is found in the hybrid simulations. This is also reflected in the force-displacement hysteresis loops, where significantly more yielding at the first storey is visible for these three ground motions. This suggests possible errors in the numerical models, particularly in the parameters that control post-elastic behaviour. The selection of values for these parameters in the numerical model is discussed more thoroughly in Chapter 10, where a model updating algorithm is developed to optimize parameter selection based on experimental feedback. The difference between the hybrid and numerical simulations may also be partly attributable to the fact that the initial stiffness matrix, as opposed to the tangent stiffness matrix, is used throughout the hybrid simulation calculation for the experimental substructure.

The trends outlined above are reinforced in Figures 3-38 and 3-39, where the peak inter-storey drift and peak floor accelerations obtained in the hybrid and entirely numerical simulations are compared. The *EDPs* from the numerical analyses are greater at the test storey level for the majority of cases. However, at roof level the response parameters are similar, again showing the more even distribution of response parameters over the frame height in the hybrid simulations.

9.5 Conclusion

The Chapter presents the results of a hybrid simulation testing programme. Hybrid simulations were performed for the 50% in 50 year and 10% in 50 year intensity levels for a case study site. Due to the failure of a component in the test system, experiments at the 2% in 50 year intensity level were not performed, as has originally been intended.

The tests completed were executed successfully. Examination of the experimental response showed that compression brace forces exceeded the buckling load determined using Eurocode 3. Furthermore, bracing members in the numerical substructure appeared to display greater energy dissipation than the experimentally tested brace specimens.

In general, the hybrid simulations and analytical models compare very well for the displacement and acceleration response at the 50% in 50 year intensity level, while the structure is behaving elastically, albeit with a small, noticeable difference in the force-displacement hysteresis. At the 10% in 50 year intensity level *EDPs* at the test storey level recorded in the numerical analyses are generally greater than those observed in the hybrid simulations. Furthermore, the extent of the nonlinear behaviour is greater in the numerical analyses.

The reasons for these discrepancies will be investigated systematically in Chapter 10, where a model updating algorithm is implemented in an attempt to optimize parameters in the numerical models based on hybrid simulation results.

10 Hybrid Simulation with Offline Model Updating

10.1 Introduction

This Chapter details the development of a Model Updating algorithm. Originally, this algorithm was developed with the goal of performing hybrid simulation with online model updating. As discussed in Chapter 5, online model updating involves optimizing model parameters in the numerical substructure based on the measured feedback from the experimental substructure. Therefore, compared to conventional hybrid simulations, tests including model updating allow more accurate models of the numerical substructure to be developed and thus improve the accuracy of the test method overall. Furthermore, the values obtained from model updating for various model parameters can be employed to improve the accuracy of entirely numerical analyses.

However, as discussed in the previous chapter, in the course of the research, a part in the test system at TCD failed and the intended online tests were not completed. Instead, offline model updating was performed using the results of hybrid simulations carried out without online updating. Clearly, this doesn't allow the first goal of model updating, namely hybrid simulations of improved accuracy, to be achieved. However, offline model updating does allow the optimized modelling parameters to be examined. These can in turn be used to inform decisions about what values of parameters to use in numerical analyses, which in turn should generate more accurate values for *EDPs* of interest.

The Chapter begins by introducing the proposed model updating algorithm. As discussed in Chapter 5, this algorithm is designed to be implemented entirely in tcl, meaning it can be incorporated in OpenSees-OpenFresco hybrid simulations without the need to communicate with further external software. Even though, as discussed, online hybrid simulation was not performed, the algorithm was developed for this purpose and is presented in that context. The proposed algorithm is then validated using a series of numerical analyses for various scenarios of increasing complexity. Once the algorithm is fully developed, offline model updating is performed for the hybrid simulations detailed in Chapter 9. The Chapter concludes by making recommendations for modelling of CBFs in OpenSees based on the results of this model updating.

10.2 Proposed Algorithm for Hybrid Simulation with Online Model

Updating in OpenSees-OpenFresco

The proposed algorithm for hybrid simulation with online model updating is based on the algorithm used by Kwon and Kammula (2013), discussed in Chapter 5. However, the algorithm is adapted slightly so it can be used in the OpenSees-OpenFresco environment.

As mentioned previously, the algorithm is based on calculating a weighted average of a number of numerical models with different parameters, known as numerical counterparts. These weights are calculated based on the measured response of the experimental substructure and are used to update the numerical substructure.

The basic setup of experimental substructure, numerical substructure and numerical counterparts employed is illustrated in Figure 10-1. However, as this Chapter involves using a series of numerical models to validate the model updating approach, it was decided to change the conventionally employed terms ‘numerical substructure’ and ‘experimental substructure’ to ‘updated substructure’ and ‘control substructure’ respectively. This is to avoid a situation where reference is made to a numerical and experimental substructure within a numerical model.

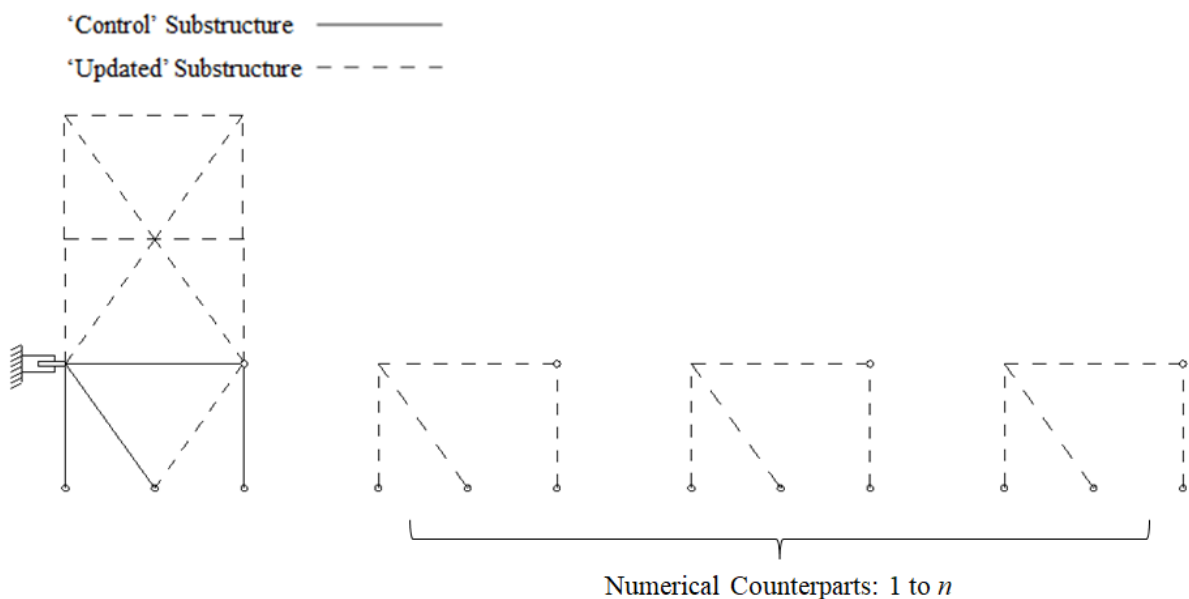


Figure 10-1 Basic setup for proposed model updating algorithm showing the ‘control substructure’ (previously the ‘experimental substructure’), the ‘updated substructure’ (previously the ‘numerical substructure’) and the numerical counterparts

The basic steps involved in the proposed model updating process employed can be summarised as follows:

1. Calculate displacement of the control substructure, u_t , i.e. the result of a time history analysis step
2. Impose displacement u_t on the control substructure and measure restoring force, R_{ctrl}
3. Impose displacement u_t on the numerical counterpart substructures and measure restoring forces, $R_{NC,i}$
4. Calculate weighting factors for each numerical counterpart, $w_{NC,i}$
 - a. At each timestep m , Minimise the difference in restoring forces through constrained optimization of the equation:

$$F(w) = \sqrt{\sum_{j=1}^m \left[\left(R_{ctrl}(t_j) - w_1 R_{NC,1}(t_j) - w_2 R_{NC,2}(t_j) - \dots - w_n R_{NC,n}(t_j) \right)^2 \right]} \quad 10.1$$

Subject to the constraints that:

$$w_1 + w_2 + \dots + w_n = 1 \quad 10.2$$

$$0 \leq w_i \leq 1 \quad 10.3$$

5. Identify which modelling parameters should be updated
6. Calculate the value of parameters to be updated, P_{update} , using the expression:

$$P_{updated} = w_1 P_1 + w_2 P_2 + \dots + w_n P_n \quad 10.4$$

7. Update the values of the parameters to the calculated values
8. Advance to the next timestep

Figure 10-2 illustrates these steps graphically.

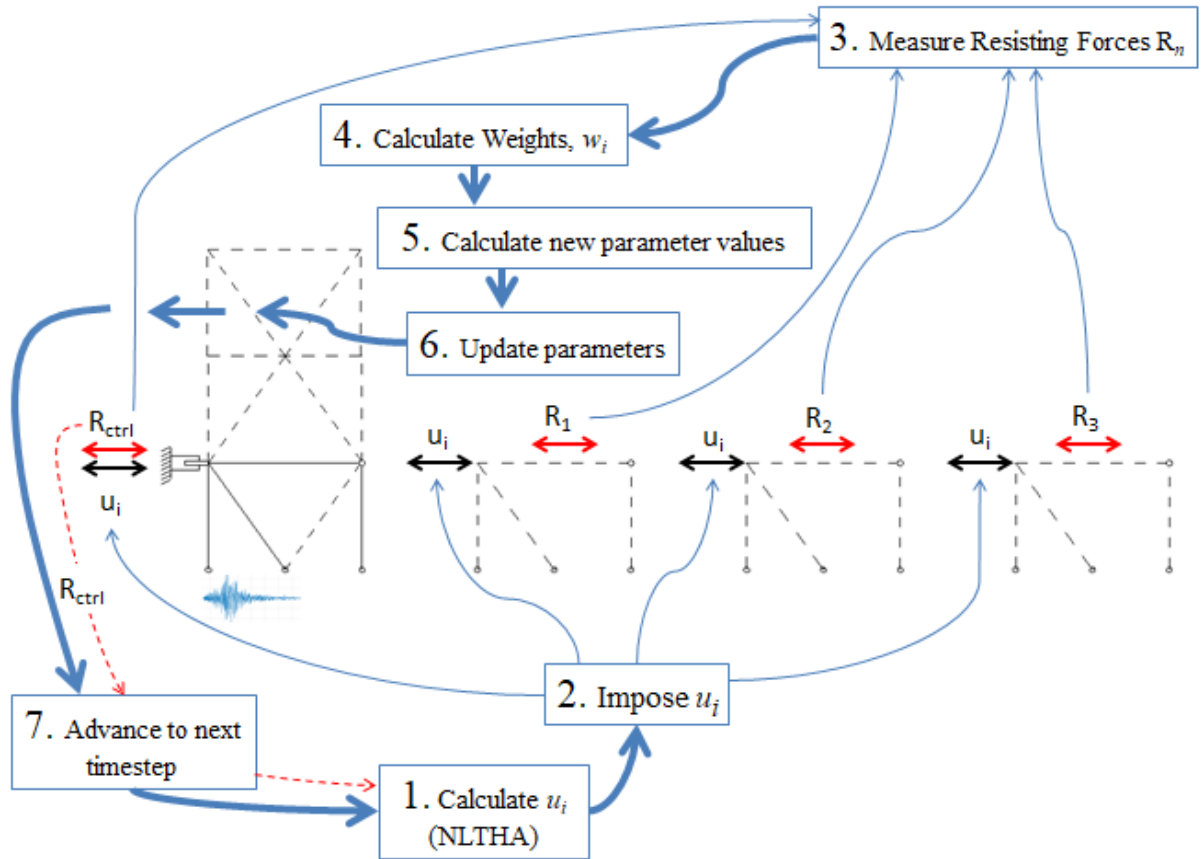


Figure 10-2 Illustration of the proposed model updating algorithm

The first step of the algorithm is the same as in a regular hybrid simulation. The structure is modelled in OpenSees-OpenFresco, numerical integration is performed and the displacement is calculated. The target displacement is imposed on the experimental substructure through OpenFresco, again as in a regular hybrid test. However, this displacement is also imposed on the numerical counterparts, which are included in the same OpenSees model, as shown in Figure 10-3(a) and discussed in Section 10.2.1.2. The restoring forces from the experimental substructure and the numerical counterparts are measured and used in Equation 10.1 to calculate the weighting factors for each counterpart so that the difference between the measured and weighted sum of the restoring forces since the start of the test is minimised. Thus the highest weight is given to the counterpart whose response is most similar to the experimental substructure. The parameters of the numerical substructure are then updated according to these weights, something which is dealt with in greater detail in Section 10.3.2. In this study, these steps are also carried out in OpenSees, which allows the model updating to be performed using the experimental setup already in place in TCD. Numerical integration for the next step in the analysis is then carried out using the model consisting of the control (experimental) substructure and the updated (numerical) substructure. As in regular hybrid simulation, the

restoring force measured from the experimental substructure at the previous time-step is used in this calculation.

The algorithm is the same as that employed by Kwon and Kammula (2013) up until step 5. In that study, the weighting factors were used to weight a set of alternative numerical models, and the restoring force obtained from a weighted sum of these was used to advance the simulation. Here, the weighting factors are used to update parameters of the updated (numerical) substructure. In other words, Kwon and Kammula (2013) updated the force vector whereas here the stiffness matrix is updated. There are two main advantages to doing this. Firstly, it allows the method to be implemented in OpenSees, as element properties, and therefore the stiffness matrix, can easily be changed during an analysis. Secondly, it allows the updated values of weighted parameters to be inspected individually, meaning there is a greater scope to learn about which values should be deployed in future purely numerical models. This is in contrast to the Kwon and Kammula (2013) setup, where only the suitability of different combinations of parameters is identified.

The disadvantage to updating parameters of the numerical substructure is that an extra step identifying which parameters should be updated needs to be included. As will be discussed in Section 10.3.2, this is necessary to avoid spurious updating of parameters affecting post-elastic response whilst the structure is behaving elastically.

10.2.1 Implementation of Model Updating in OpenSees & OpenFresco

The entire model updating process has been implemented within the OpenSees – OpenFresco framework, meaning it can be deployed within the existing three-loop architecture for hybrid simulation at TCD, described in Chapter 8. This also means that the material libraries, computational power and nonlinear analysis capabilities of OpenSees are available, thus avoiding the need to develop a finite element model in other software, such as Matlab, as was required in previous hybrid simulations with model updating performed elsewhere, for example in Hashemi et al. (2014) or Wu et al. (2016).

There are also other potential future benefits to developing a model updating application using OpenSees-OpenFresco at TCD. Research engineers familiar with the OpenSees framework for numerical analysis will be able to easily move to an online model updating environment, allowing effort to be focused on structural research rather than technical challenges in development of the test method, while finite element models developed for numerical analysis

in OpenSees can be easily adapted for hybrid simulation with OpenSees-OpenFresco, and vice versa.

The following subsections detail how the algorithm was implemented in OpenSees for each of the steps outlined previously.

10.2.1.1 Step 1 & 2 – Regular Hybrid Simulation

Steps 1 and 2 are the same as in a regular hybrid simulation without updating. Step 1 involves carrying out a NLTHA timestep; this is done in OpenSees. Step 2 involves imposing the calculated displacement on the updated substructure and measuring the restoring force. This is done through OpenFresco, using the framework discussed in Chapter 8

10.2.1.2 Step 3 – Numerical Counterparts

Step 3 involves imposing the target displacement in the numerical counterparts and measuring the restoring forces. This is done within the OpenSees model, which contains both the test structure (consisting of the control and updated substructures) and the numerical counterparts, as shown in Figure 10-3. Regular nonlinear time history analysis is carried out on the test structure, with the ground motion record applied to the base of the test structure using the multi-support excitation pattern. At each timestep the calculated displacement of the experimental substructure is imposed on each of the numerical counterparts. This is a displacement controlled static analysis implemented using the single point constraint command.

The process of running a dynamic time-history analysis and a displacement controlled static analysis of separate structures concurrently in the one OpenSees model was verified in preliminary numerical analyses, as illustrated in Figure 10-3. It can be seen that identical results are obtained using a ‘multiframe’ mode, where a dynamic and a pushover analysis are run concurrently on separate frames, and using ‘regular’ individual frame analyses in separate OpenSees models.

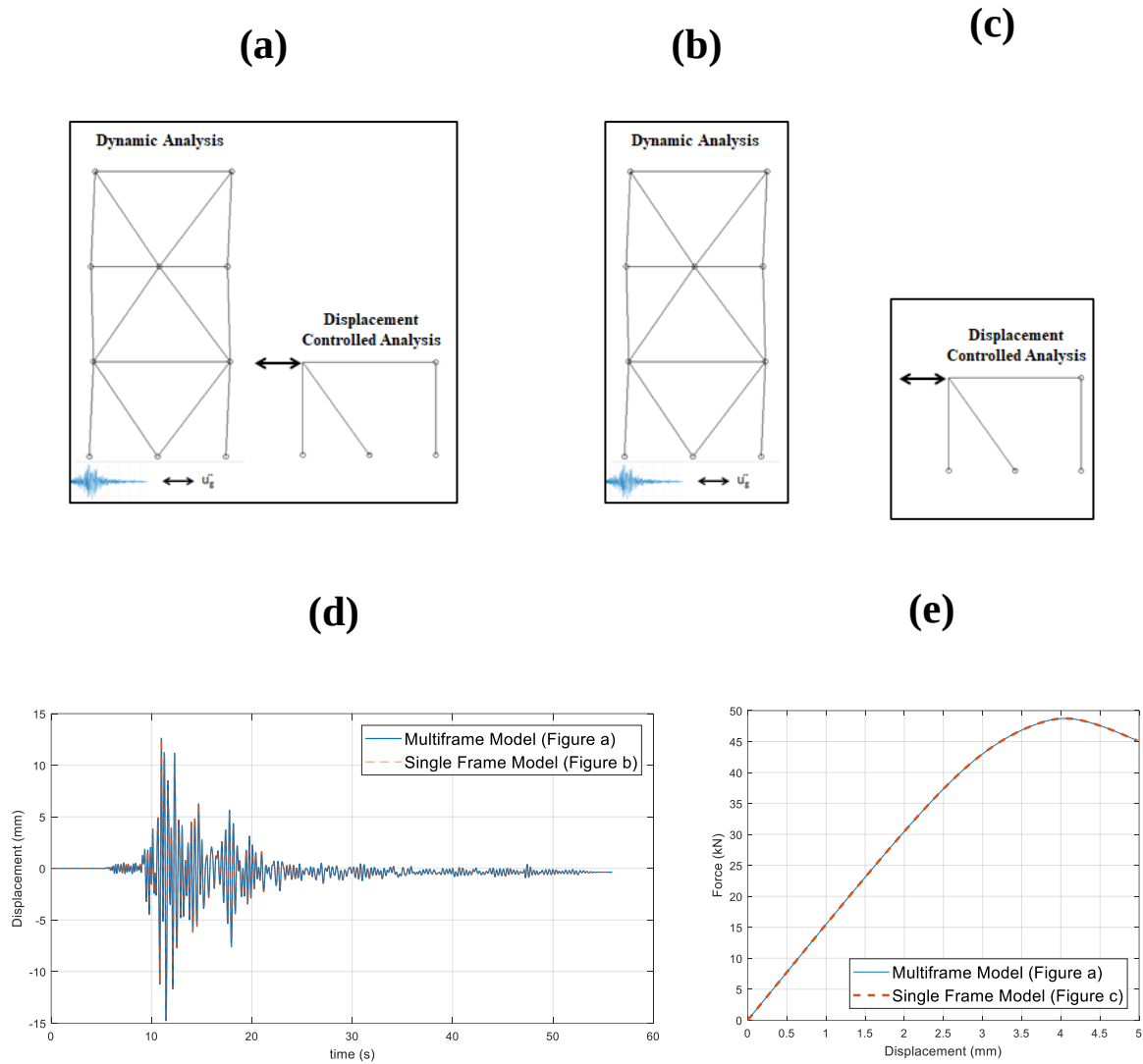


Figure 10-3 Verification of the concept of running dynamic and displacement controlled analysis concurrently in a single OpenSees model: (a) OpenSees ‘multiframe’ model for concurrent dynamic and displacement controlled analysis; (b) Regular OpenSees model for dynamic analysis; (c) Regular OpenSees model for displacement controlled analysis; (d) Comparison of dynamic analysis from the ‘multiframe’ and regular models; displacement time history is identical; (e) Comparison of displacement controlled analysis from the ‘multiframe’ and regular models; pushover curves are identical

10.2.1.3 Step 4 – Calculating Weighting Factors

Step 4 involves calculating the weighting factors for each numerical counterpart. As discussed, the weighting factors in Equation 10.1 are calculated to minimise the difference between the measured resisting force from the experimental substructure and the numerical counterparts. In order to do this, a constrained minimisation routine was implemented in tcl. Due to the limited functionality of tcl (e.g. the inability to quickly perform matrix manipulation), the minimisation routine developed was very simplistic. The objective function was evaluated using all values in the feasible region, i.e. using all combinations between 0 and 1 in steps of 0.01 for each weighting factor, and the minimum value obtained. Thus, the optimum weighting factors can be

found. For a limited number of numerical counterparts, this minimisation method was found to work reasonably efficiently. As the routine was implemented in tcl, it can be included within an OpenSees script, thereby avoiding the need to use other software to perform the minimization. The algorithm was written specifically for the objective function in Equation 10.1 and constraints associated with this particular problem; hence it is not scalable or transferable to other problems.

More efficient optimization methods have been implemented elsewhere, for example in Matlab. However, the routine used here avoids the need for communication between OpenSees and other software like Matlab. Due to delays associated with this communication step, the basic minimisation algorithm developed here was found to work quicker for a limited number of numerical counterparts than more efficient algorithms in other software. In the future it is possible that a more elegant minimization routine could be implemented in the OpenSees source code.

10.2.1.4 Step 5 & 6 – Parameter Identification and Parameter Value Calculation

Step 5 and 6, identifying the parameters to be updated and calculating the values of the updated parameters, are both implemented in tcl. The method of identifying parameters to update is based on whether the structure is behaving linearly or nonlinearly, and is discussed further in Section 10.3.2. The values of the updated parameters are calculated according to Equation 10.2. Both of these steps are implemented in tcl, and are executed before the end of each analysis step.

10.2.1.5 Step 7 - Parameter Updating

The weighting factors obtained from the minimisation routine are then used to obtain the values of the parameters to be applied to the numerical substructure in accordance with Equation 10.1. Examples of parameters that can be update include the material properties of steel such as the elastic modulus, yield strength and post yield stiffness or the stiffness of the connections. The value for the initial camber of the bracing element suggested by the algorithm can be monitored, but online updating is not possible as, by its nature, an initial camber has to be set at the start of the test.

Scott and Haukaas (2008) developed a method for implementing parameter updating in OpenSees, the *parameter* and *updateParameter* objects allow member properties, applied loadings, and nodal coordinates to be identified and repeatedly updated during analysis. These commands were originally developed to carry out sensitivity analysis in OpenSees, however they

can also be used to implement a model updating algorithm within the OpenSees-OpenFresco framework, as has been done by Padilla (2015). These updating commands have not yet been implemented in the OpenSees source code for the Steel02, or Giuffre-Menegotto-Pinto, material model that is used elsewhere in this work. Hence, the bilinear Steel01 material model is used here for the updated substructure.

The tcl code that implements the developed analysis algorithm for model updating is given in Appendix G.

10.3 Numerical Verifications

The following section presents a series of numerical verifications of the updating algorithm intended to show that the algorithm works correctly.

10.3.1 Updating in Linear Analysis

To verify the basic principle of the algorithm discussed above, updating is applied for a single parameter in a linear analysis. The value that the algorithm returns for the updated parameter is examined firstly, before the impact of updating on response is investigated.

10.3.1.1 Updated Parameter Value Verification

Figure 10-4 shows the setup for the first numerical verification. The updating algorithm was validated by demonstrating that the approach can be used to reach a known target value of a parameter, the modulus of elasticity in this case. Frame A in Figure 10-4 is the ‘control’ substructure, which is treated as being the ‘true’ or correct model for the purposes of numerical verification. As can be seen from the Figure 10-4, the material used in the control substructure has a modulus of elasticity of 210 kN/mm². Frames B and C are the numerical counterparts and have moduli of elasticity of 170 kN/mm² and 220 kN/mm² respectively. The three frames are exactly the same aside from this. The updating algorithm is applied for the modulus of elasticity with the ideal outcome being that it returns a value of 210 kN/mm²; i.e. it recommends updating the value of E to the ‘true’ value used in the control substructure. In order to obtain this value of E, the weighting factors for frames B and C should equal 0.2 and 0.8 respectively, as, using Equation 10.4:

$$w_B \times E_{Frame\ B} + w_C \times E_{Frame\ C} = E_{Updated} \quad 10.5$$

$$0.8 \times 220 + 0.2 \times 170 = 210$$

It should be noted that no updating is actually performed here as no numerical substructure is modelled; the purpose is simply to examine the suggested value of elastic modulus returned by the algorithm.

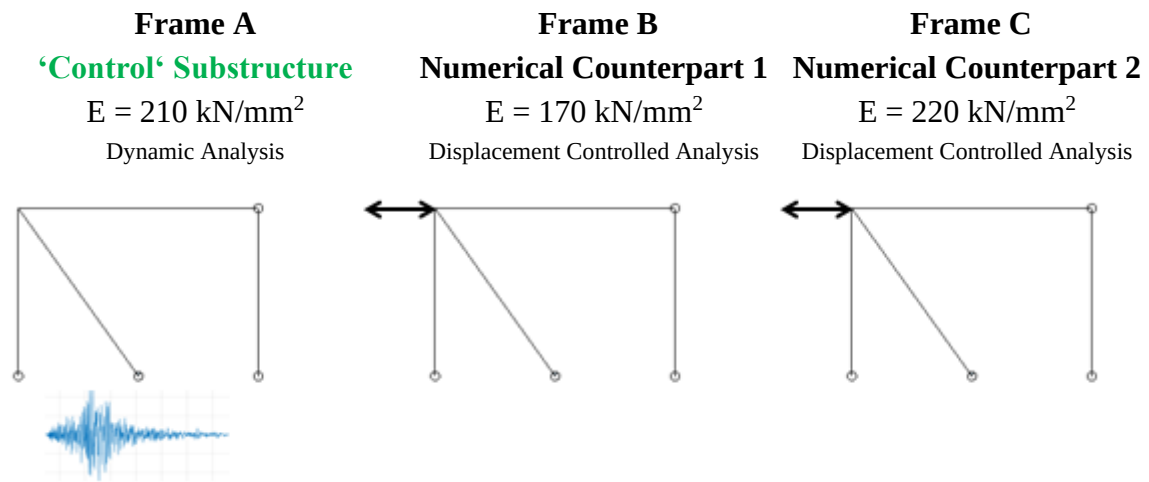


Figure 10-4 Frame properties and OpenSees Model for validation of parameter updating algorithm

Time history analysis was performed for Frame A, the control substructure. The calculated displacements were then imposed on the Frames B and C, the numerical counterparts, using the single point constraint method as illustrated in Figure 10-4.

Figure 10-5 shows the value to which the modulus of elasticity would be updated if a numerical substructure was included in the model. Analysis was performed using a low intensity ground motion, meaning the frames behaved elastically, as is illustrated by the force-displacement hysteresis plot. It can be seen that the value converges rapidly to 210 kN/mm², the value used in the control substructure. This shows the updating algorithm works correctly and proves the basic assumptions made in its development are correct.

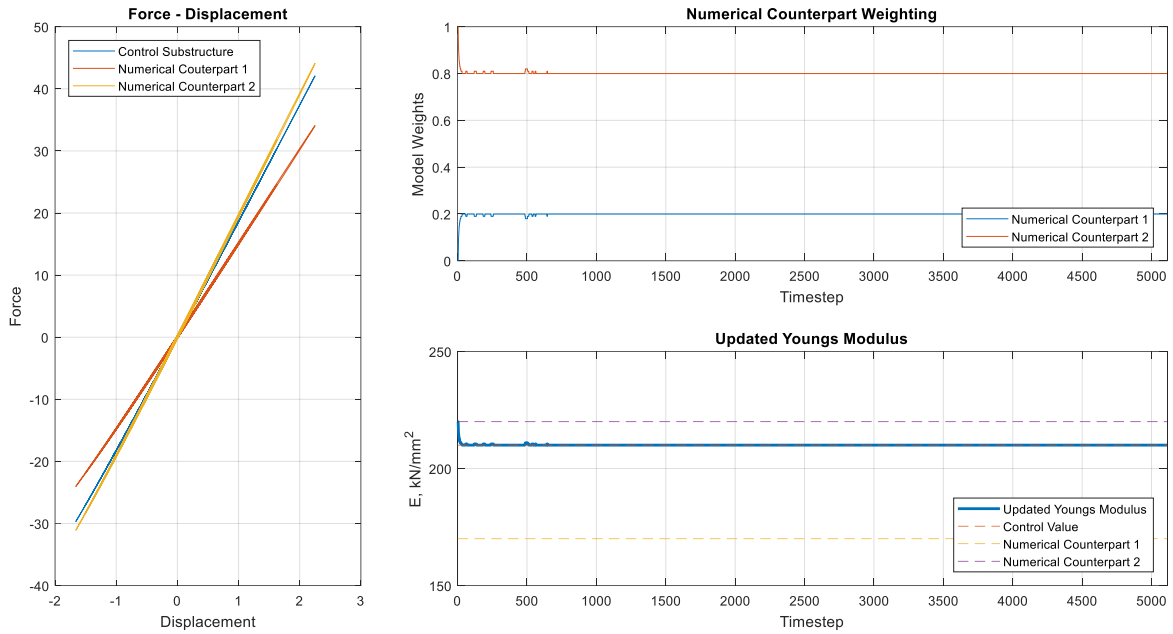


Figure 10-5 Validation of parameter updating; value of the updated elastic modulus matches the true value from the control substructure

10.3.1.2 Updating Verification

The impact of applying model updating to a numerical model is now examined. This is a development of the previous section where the value of the updated elastic modulus was shown to reach its target value in a linear analysis.

As indicated in Figure 10-6, a fourth frame, Frame D, is included in the analysis and represents the ‘updated’ substructure, which in a hybrid simulation would be the numerical substructure. The value of the modulus of elasticity of the material used in this model is initially set to 180 kN/mm². This replicates how in a regular analysis modelling parameters are assigned values that are expected to be different from the true value. In an experimental simulation the true value of a parameter in the control substructure is unknown, however here the ‘true’ value of elastic modulus is set as 210 kN/mm².

Dynamic analysis is carried out on the control substructure, as before, and on the updated substructure. Again as before, the displacements calculated for the control substructure are imposed on the numerical counterparts, with the resisting forces from these and the control substructure used to calculate the weighting factors to calculate the updated value of elastic modulus.

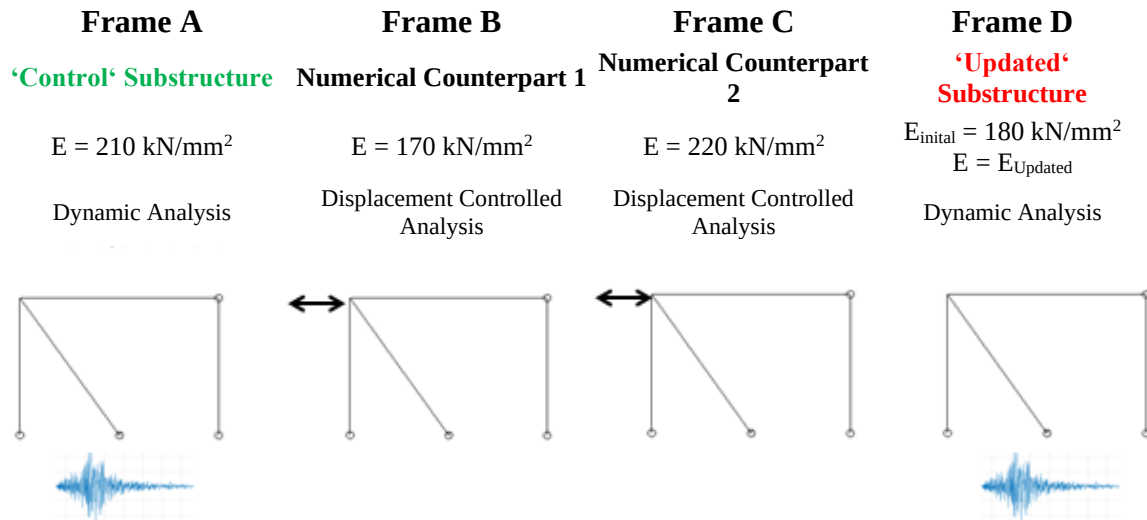


Figure 10-6 Frame properties and illustration of the OpenSees model to examine the impact of model updating

Figure 10-7 shows the displacement time history and force-displacement hysteresis of the ‘control’ substructure and the ‘updated’ substructure when the updating algorithm is not applied, i.e. the elastic modulus of steel in the updated substructure remains constant at 180 kN/mm^2 throughout the analysis. The difference between the two responses is evident.

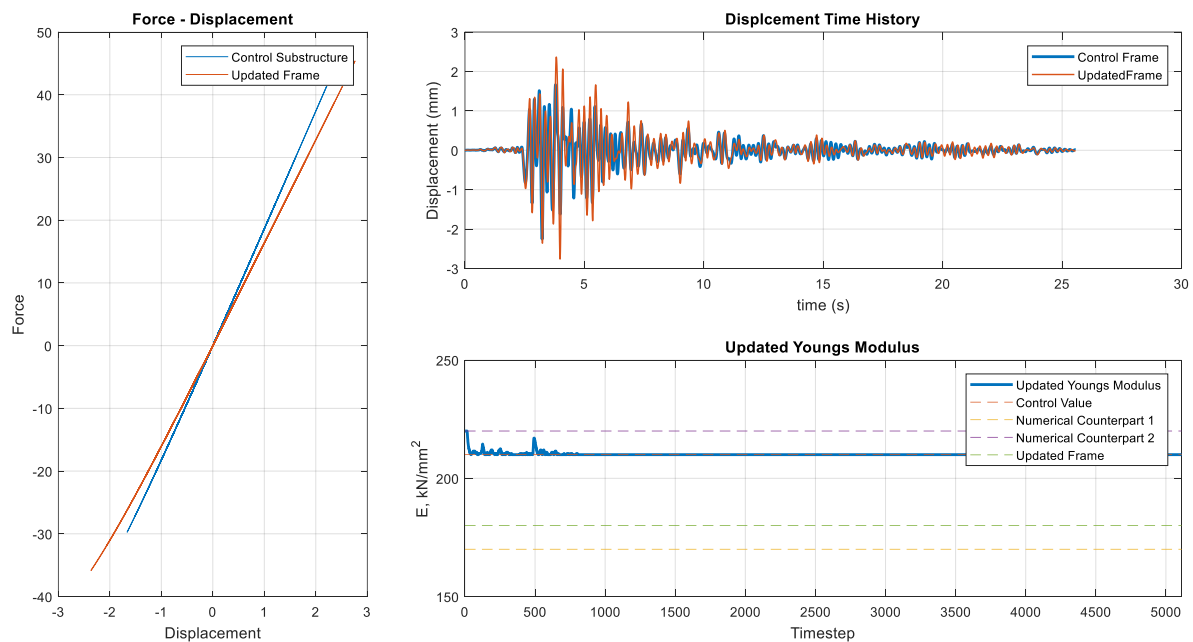


Figure 10-7 Comparison of the ‘true’ response from the control substructure and the modelled response from the updated substructure with no model updating

Figure 10-8 shows the same data for an analysis where updating is applied; i.e. the value of the modulus of elasticity used in the ‘updated’ substructure is updated at every timestep. It can be seen that the results are practically indistinguishable from the ‘true’ values obtained from the

control model. This indicates that the updating algorithm works well and the basic concepts underpinning it are valid. It also demonstrates the potential value of model updating for improving the accuracy of hybrid simulation.

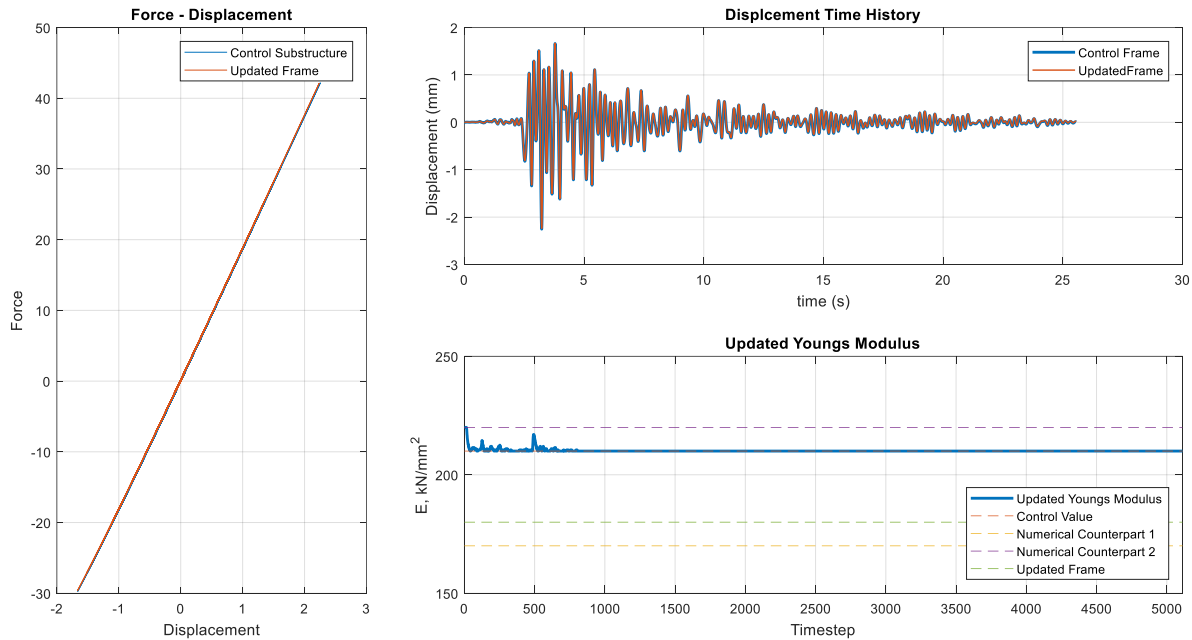


Figure 10-8 Comparison of the ‘true’ response from the control substructure and the modelled response from the updated substructure with model updating applied

10.3.2 Multiple Parameter Updating in Non Linear Analysis

The previous section shows that the algorithm developed can be used to correctly update one unknown parameter in a linear analysis and that models using this updated parameter can match the ‘true’ response. However in a real hybrid simulation the modeller is required to assign values to multiple parameters whose true values are unknown. Furthermore analysis at high intensity levels is not always linear, meaning different parameters affect different stages of the analysis and the optimum value of a parameter may change throughout an analysis. The following section discusses how these two challenges are addressed.

10.3.3 Updating Multiple Parameters

Challenges relating to the uniqueness of the solution are a major obstacle in the implementation of the proposed model updating algorithm. Until now, the validations have been performed in numerical situations where only one parameter differs between the control substructure and the numerical counterparts. However, in a real hybrid simulation there are multiple modelling uncertainties that can lead to differences between a numerical counterpart and the control (experimental) substructure meaning that multiple parameters need to be updated

concurrently. This can lead to issues with the uniqueness of the solution obtained from the model updating algorithm.

In order to update multiple parameters separate sets of numerical counter parts are used for each parameter. In each set of counterparts the only variation is in the parameter that the counterpart is used to update. Therefore, differences between the restoring force measured from the two counterparts are attributable solely to the parameter in question. This concept is illustrated in Figure 10-9, where the first pair of counterparts is used to update the elastic modulus and the second pair is used for the yield strength.

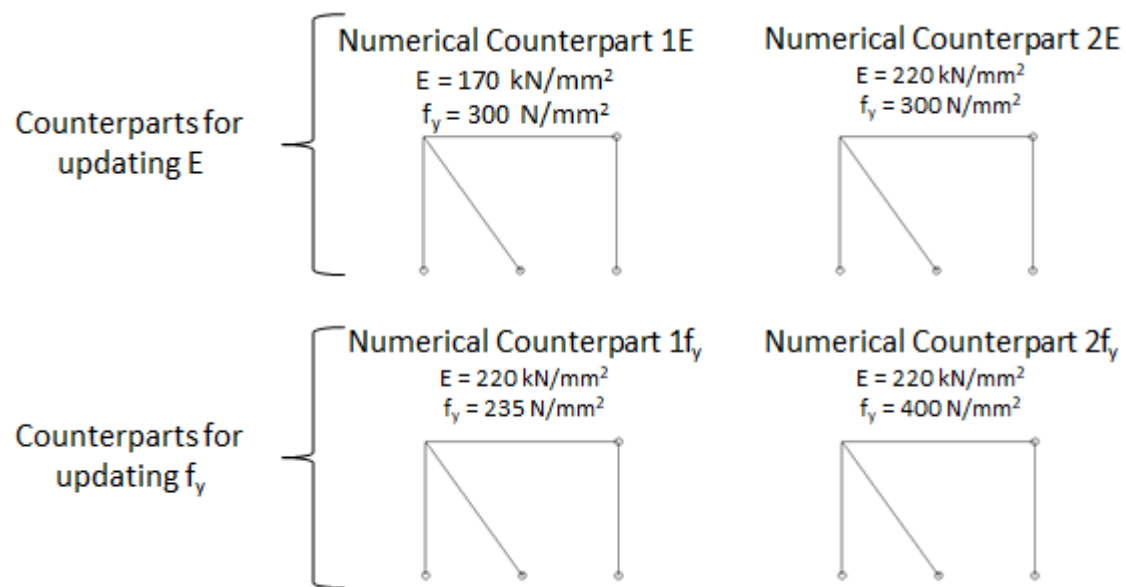


Figure 10-9 Numerical counterparts for updating multiple parameters; the only variation between counterparts in each set is for the parameter being updated

The results obtained from this method can become dependent on the values used for the non-updated parameters in the numerical counterparts, for example the value of elastic modulus used in counterparts employed to update the yield strength. However, through preliminary analysis and prior knowledge it should be possible to select reasonable values for these that don't greatly impact on the results.

10.3.4 Linear & Nonlinear Parameters

When the model updating algorithm is applied in nonlinear analysis, parameters that only become relevant at yield or in the post elastic range, such as the yield strength, would be updated spuriously based on the response before yield occurs. This is illustrated in Figure 10-10, which shows the updated values for yield strength in a hypothetical structure. The 'true' value of yield strength is set as 355 N/mm^2 . However, it can be seen that early in the test, before the

structure yields, the yield strength spuriously fluctuates between 400 N/mm² and 235 N/mm², before settling at 235 N/mm². Clearly, in an online test, if this value was applied to the updated substructure, this would negatively affect the accuracy of the calculated response. This is particularly true if the yield strength is updated to a low value, as in Figure 10-10, which can result in premature yielding in the numerical substructure.

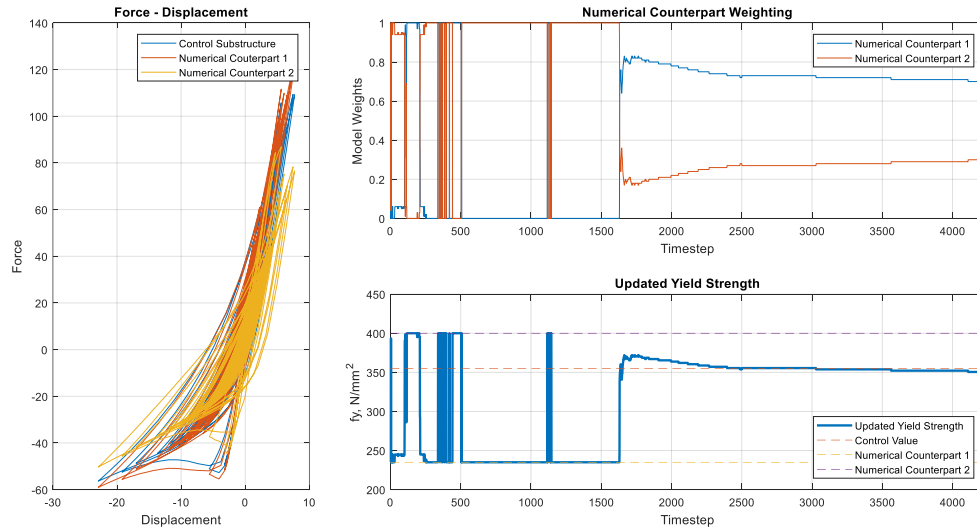


Figure 10-10 Example of spurious updating of the yield strength whilst the control substructure is behaving elastically

In order to deal with this a method to identify whether the control substructure is behaving linearly or nonlinearly was developed. Parameters are only updated during the relevant portion of the test. Hence, for example, yield strength is only updated once yielding has occurred.

The method for identifying whether the control substructure has yielded is illustrated in Figure 10-11. An approximation of the elastic stiffness of the control substructure is developed in the early stages of the test. For the particular control substructure examined here this is developed from the force-displacement hysteresis response while the displacement is less than 2mm, as it has been established that response is generally linear for the substructure under examination in this range. This range can be adapted for other substructures. The linear stiffness obtained is then extrapolated, as in Figure 10-11, to create an estimate of linear response over a wider displacement range.

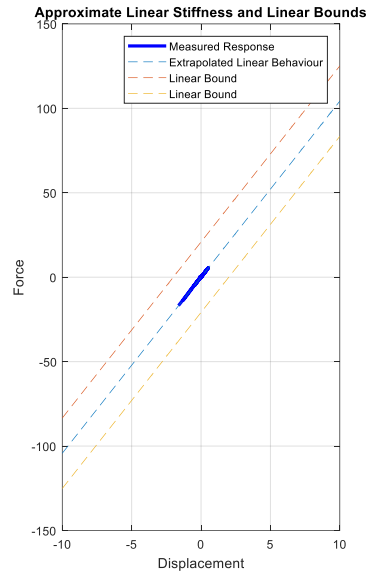


Figure 10-11 Illustration of the method for identifying whether the control substructure has yielded or not

An upper and lower bound is then created either side of the estimated linear stiffness estimate, as shown in Figure 10-11. These bounds are to allow for errors in the initial stiffness estimate and noise in the measurement system. The offset used to create these boundaries is arbitrary; here it was derived using a 2mm displacement at force of 0 kN. The slope of these boundary lines is the same as the estimated elastic stiffness. Thus, given that a value for the slope and a point on both lines are known, the equations describing these two lines can be easily calculated.

The control substructure is deemed to have yielded once the force-displacement hysteresis goes outside these bounds. This is identified by using the equations of the boundary lines to check whether the measured force is within the corresponding force limits. This concept is illustrated in Figures 11-12 and 11-13, which use the hysteretic responses obtained from hybrid simulations run as part of the experimental tests detailed in Chapter 9 to verify the method. In Figure 10-12 the hysteretic response stays within the yield limits, hence the structure is not deemed to have yielded at any point. Therefore the Boolean 'Has Yielded' variable remains at 0 throughout. In contrast, in Figure 10-13 the hysteresis plot crosses the yield limits after approximately 4 seconds. At this point the control substructure is deemed to have yielded, as indicated by the 'Has Yielded' variable shown in the plot.

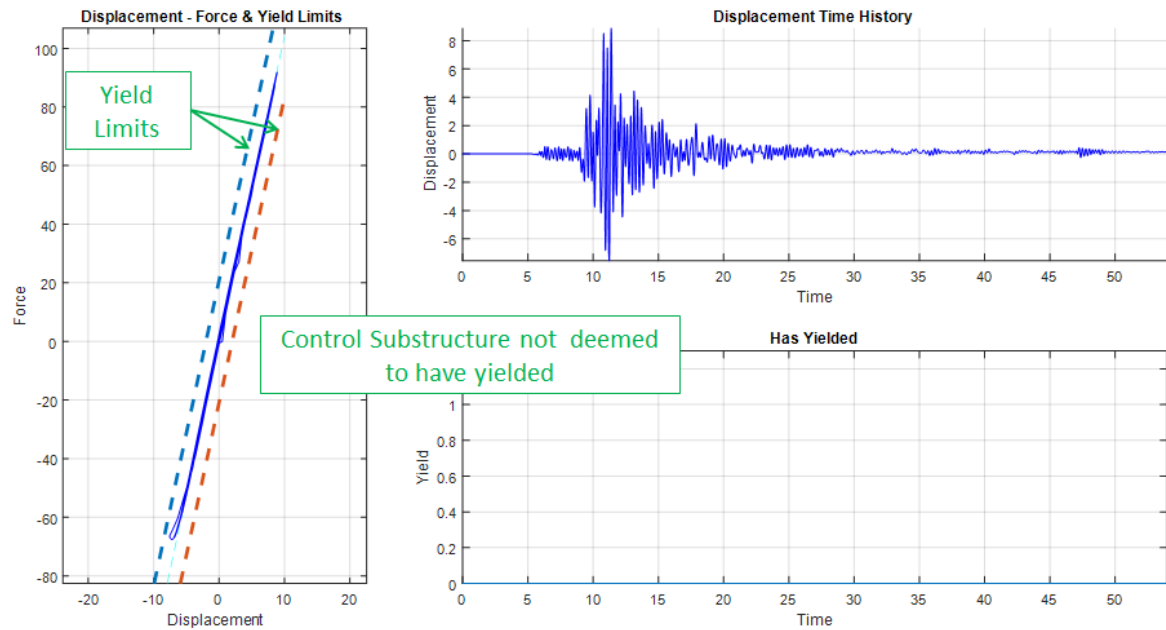


Figure 10-12 Hysteretic response of the control substructure stays inside the yield limits and hence is not deemed to have yielded

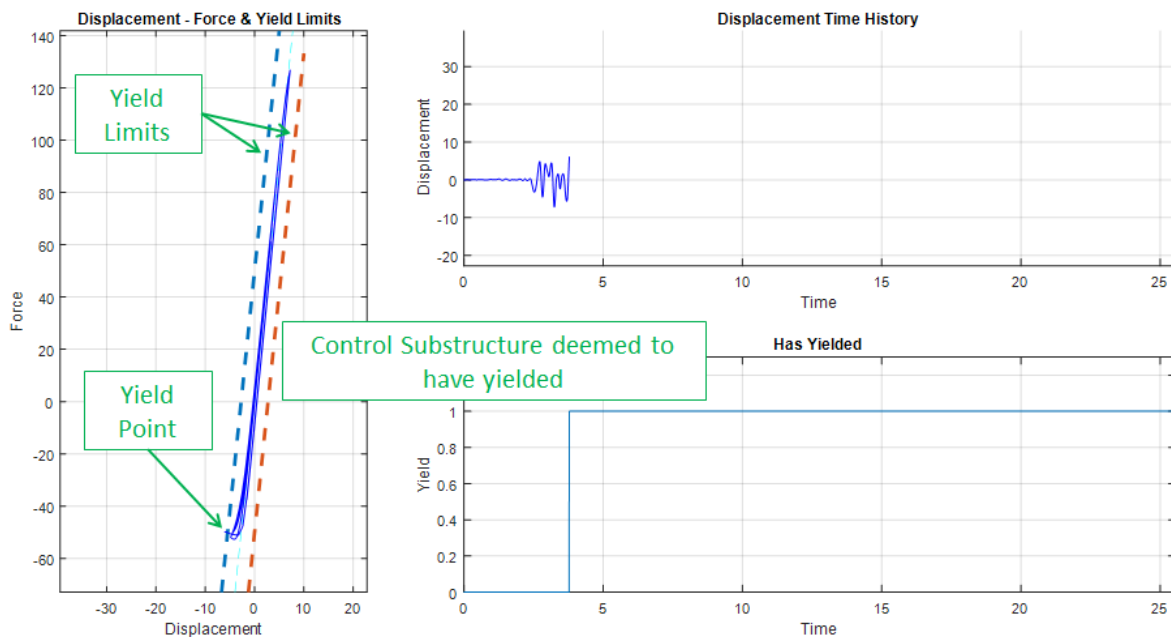


Figure 10-13 Hysteretic response of the control substructure stays inside the yield limits and hence is deemed to have yielded

For the purpose of model updating, as will be demonstrated in the next Section, it was found that better results were obtained if the linear parameters were only updated before the control substructure was deemed to have yielded and nonlinear parameters only updated after yield occurred. This method of identifying the yield point was implemented in tcl, which allows it to be included within an OpenSees analysis script.

10.3.4.1 Optimization Verification

Using these two adaptations, the numerical analysis employed in Section 10.3.1 is repeated. Again, this is used to verify that the algorithm can be used to reach the known target value of parameters, but this time the yield strength is also examined alongside the modulus of elasticity in a nonlinear analysis.

The setup employed for this verification is shown in Figure 10-14. Frame A is the control substructure and is assigned 'true' values of $E = 210 \text{ kN/mm}^2$ and $f_y = 355 \text{ N/mm}^2$. Frames B and C are the numerical counterparts used to update the elastic modulus, while D and E are the counterparts used for the yield strength.

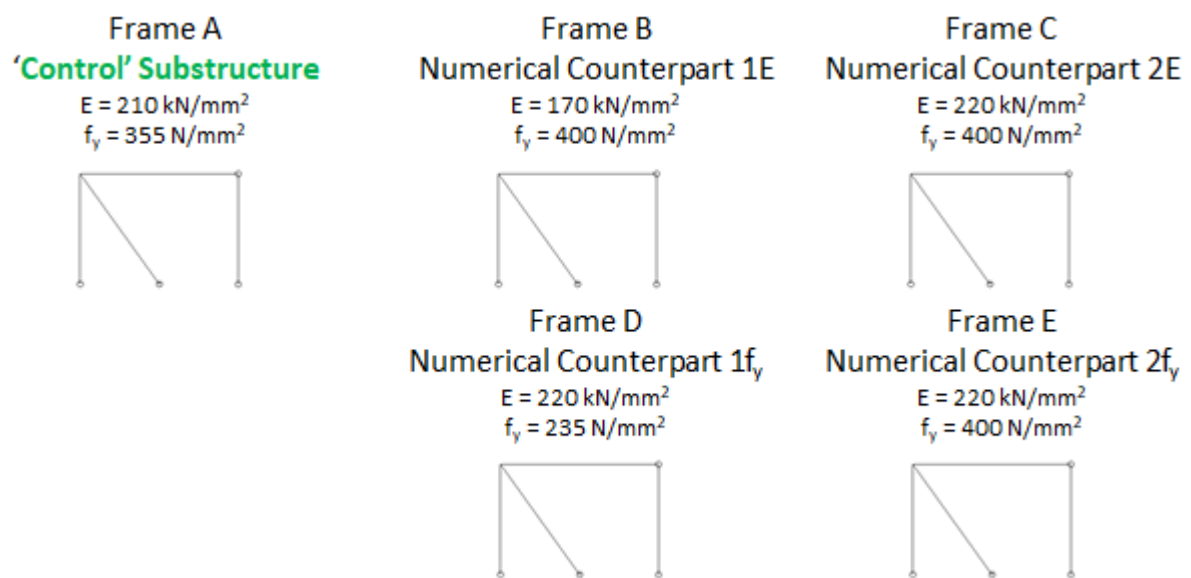


Figure 10-14 Setup for numerical verification of multi-parameter nonlinear updating

It should also be noted here that the counterparts have different values of the non-updated parameter to the control substructure; for example the frames used to update the elastic modulus have yield strengths of 400 N/mm^2 as opposed to the 'true' value of 355 N/mm^2 . This is to replicate the situation in a real hybrid simulation where the true values are unknown prior to the test.

Dynamic analysis is carried out for the control substructure, with the calculated control displacements imposed on the numerical counterparts and the measured resisting forces used to optimize relevant parameter weighting factors. Figures 11-15 and 11-16 show the results of this process. It can be seen that both the elastic modulus and yield strength are calculated to be very close to the 'true' values used in the control substructure. The Figures demonstrate how the elastic modulus is only updated in the linear range, while updating of the yield strength only

begins once yield is deemed to have occurred. The close match between the 'true' and updated parameter values shows that the updating strategy works well, and is clearly superior to the values obtained from updating throughout the analysis as shown in Figure 10-17.

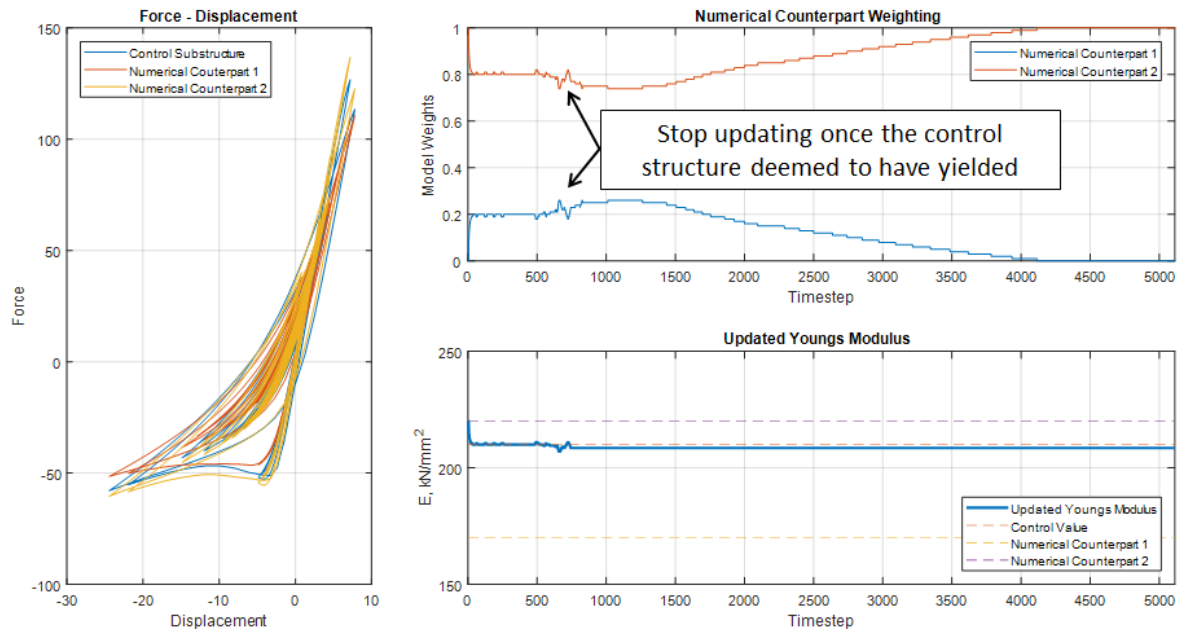


Figure 10-15 Updated value of Elastic Modulus

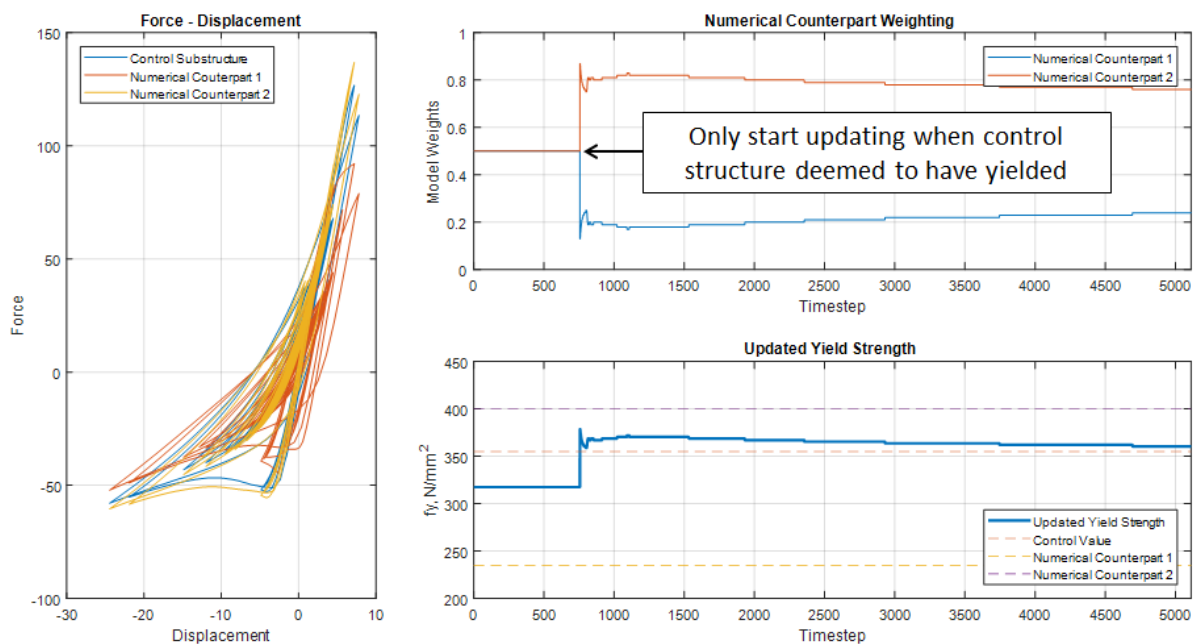


Figure 10-16 Updated value of Yield Strength

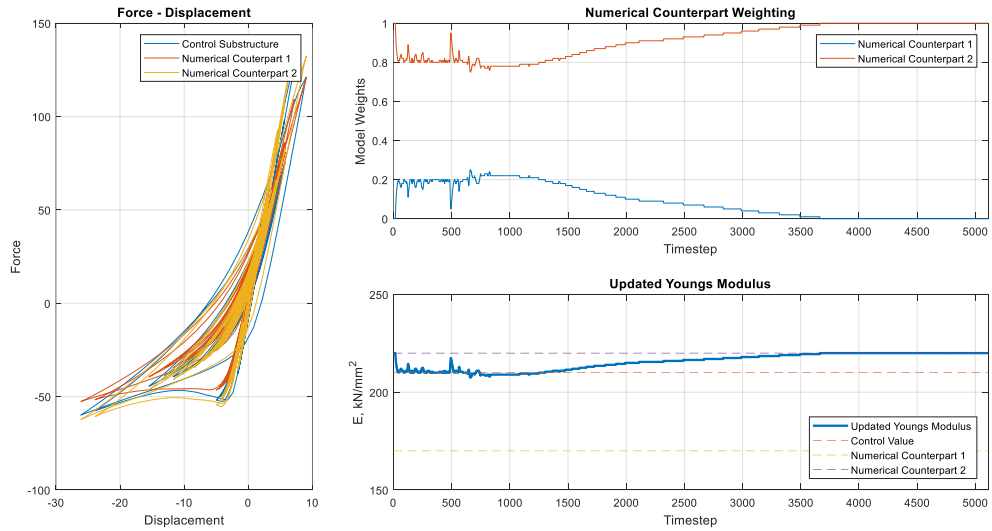


Figure 10-17 Impact of continuing updating for the entire duration of the test on elastic modulus, which moves away from the ‘true’ value assigned to the control substructure after yield

In Figure 10-16 it can be seen that the value of the updated yield strength initially fluctuates before stabilizing and slowly converging towards the ‘true’ value of the control substructure. This is because including a greater number of timesteps in the optimization of the weighting factors gives improved accuracy. Similar trends were observed by Kwon and Kammula (2013), as shown in Figure 10-18.

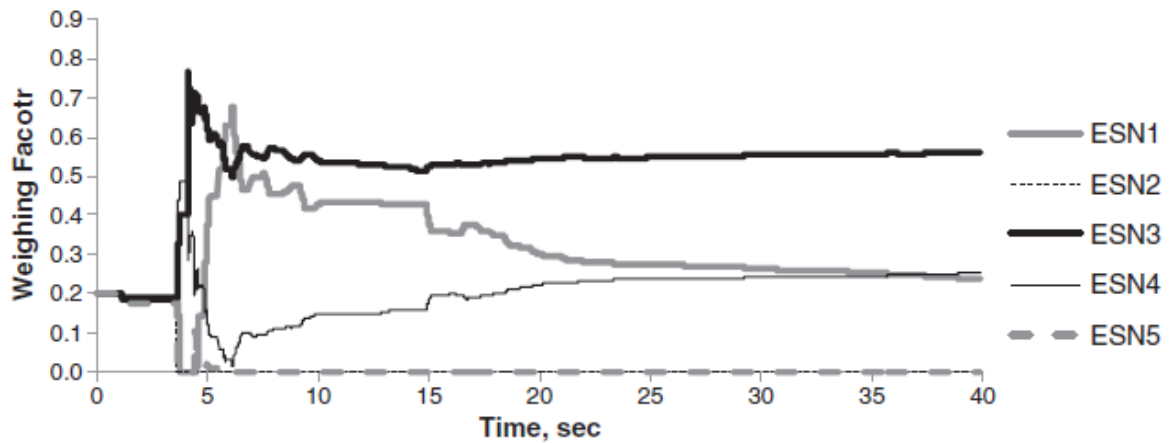


Figure 10-18 Variation in weighting of various numerical counterparts with time during a simulation with model updating by Kwon and Kammula (2013), showing how weighting factors fluctuate initially then stabilize once a sufficient number of timesteps are included in the optimization

10.3.4.2 Updating Verification

As before, the impact of updating parameters in an updated substructure is now examined. In Figure 10-19, it can be seen that an extra frame, Frame F, representing the updated substructure is added to the OpenSees model used to verify the updated parameter values in the previous section, shown in Figure 10-14. The value of the modulus of elasticity of the material used in the

updated substructure is initially set to 150 kN/mm^2 and the yield strength is set to 400 N/mm^2 . As can be seen from Figure 10-18, both of these values differ from the 'true' values assigned to the control substructure.

Again, dynamic analysis is performed for both the control and updated substructures, with the control displacements imposed on the numerical counterparts and the measured restoring forces used to optimize the parameter weighting factors. The basic premise of the verification is that if the updating algorithm is working correctly the response of the updated substructure should match that of the control substructure.

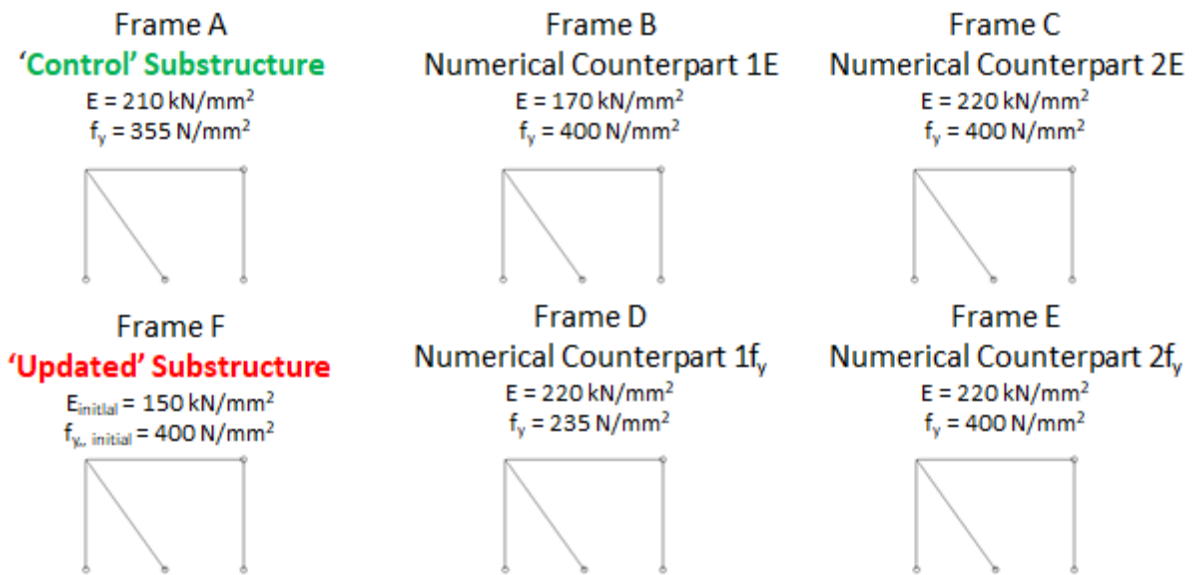


Figure 10-19 Setup for numerical verification of multiparameter updating for nonlinear analysis

Figure 10-20 shows the displacement time history and force-displacement hysteresis of the 'control' substructure and the 'updated' substructure when the updating algorithm is not applied, i.e. the elastic modulus and yield strength of steel in the updated substructure remain constant at 150 kN/mm^2 and 400 N/mm^2 respectively throughout the analysis. The difference between the two responses is evident.

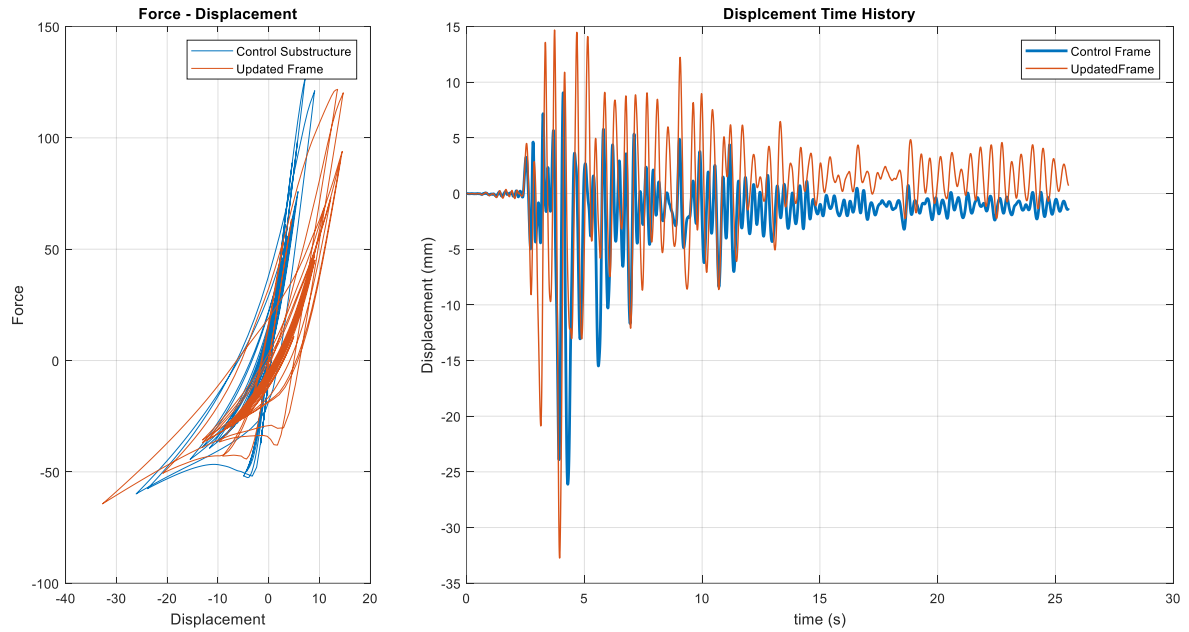


Figure 10-20 Difference in response between the control substructure, Frame A in Figure 10-19, and the updated substructure, Frame F, when no updating is applied

Figure 10-21 shows the values of the updated parameters when model updating is applied. It can be seen that the values are updated to approximately equal the 'true' values of elastic modulus and yield strength of the control substructure. Figure 10-22 shows the force-displacement hysteresis and displacement time history response of the control and updated substructures. It can be seen that the two substructures display very similar response. This demonstrates that the updating strategy adopted works well for nonlinear analysis where multiple parameters are updated.

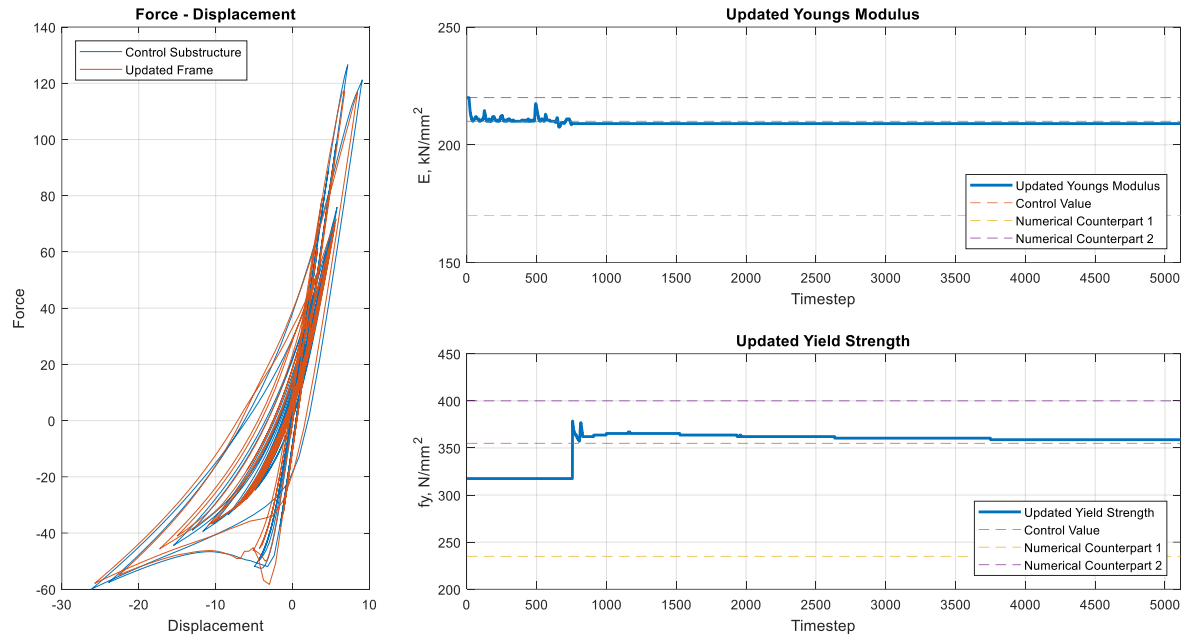


Figure 10-21 Updated values of elastic modulus and yield strength; which are close to the ‘true’ values

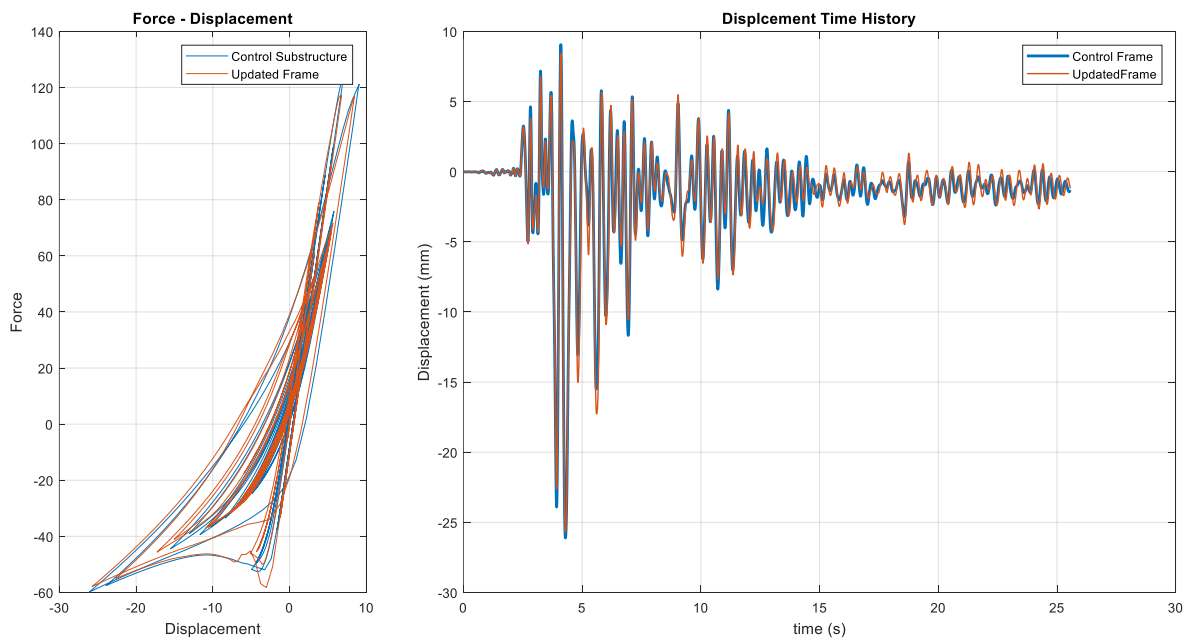


Figure 10-22 Updated hysteresis and time history response of the two substructures, which are very similar, demonstrating the validity of the updating algorithm

Figure 10-23 demonstrates the impact of updating on the displacement response of the updated substructure. The benefits of updating are clear from this Figure; the response is much closer to the ‘true’ response when model updating is applied.

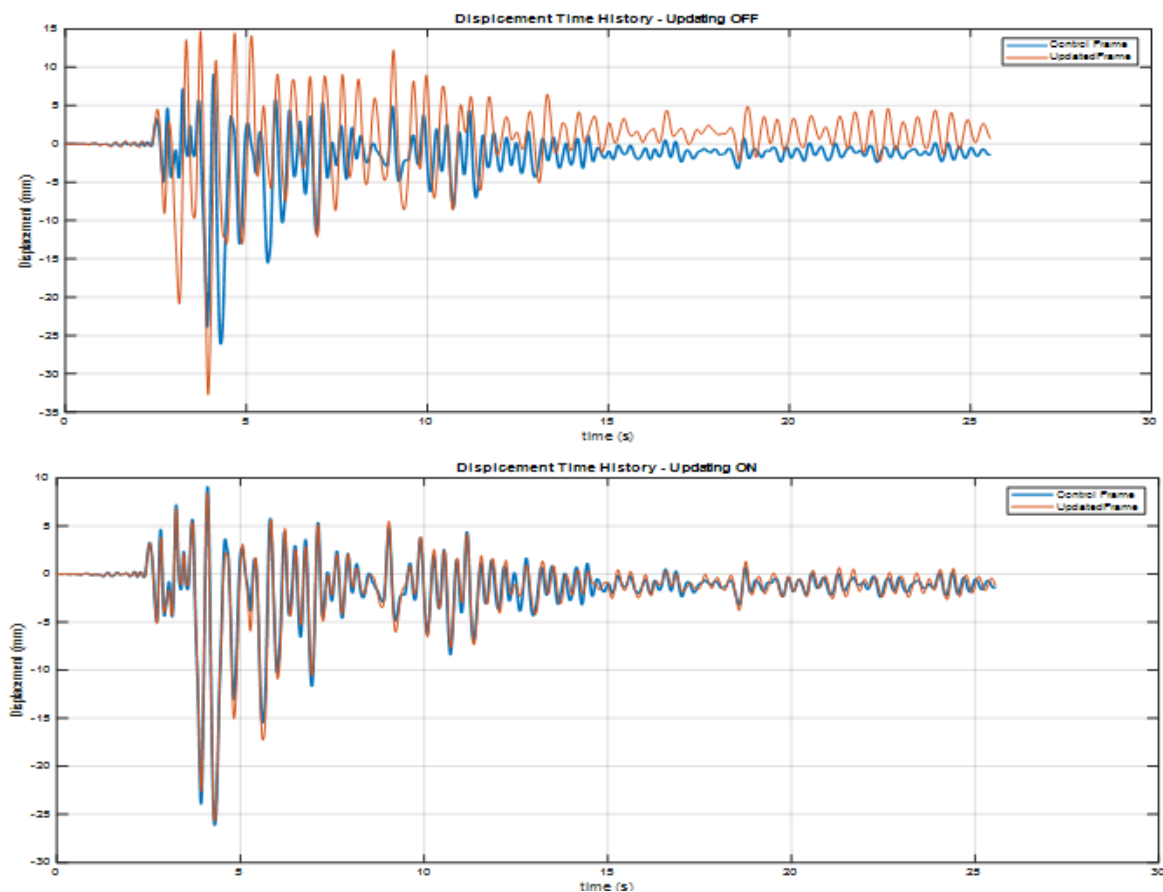


Figure 10-23 Impact of model updating; comparison of responses of updated substructure and control substructure with and without parameter updating

10.4 Offline Updating Using Experimental Data

So far the model updating algorithm has been validated and shown to work well using a numerically simulated control substructure. However, this is simplistic as the ‘true’ values are assigned prior to the simulation and the values of all parameters in the control substructure are known.

This section examines the experimental tests described in Chapter 9, and offline model updating is performed using this data. An OpenSees model containing only numerical counterparts was developed for this purpose. The calculated displacement at each timestep of the experimental test was applied to each of the counterparts and the restoring force was measured. Using the restoring force measured in the tests, and the restoring forces from the numerical counterparts, the weighting values of various parameters can be chosen to minimise Equation 10.1. The elastic modulus, yield strength and the initial brace camber are the parameters examined. The offline model updating process allows the values of parameters suggested by the model updating algorithm to be examined and compared to the values previously adopted in the hybrid simulation model.

10.4.1 Linear Hybrid Simulations

Figures 10-24 to 10-28 show the optimum values of elastic modulus suggested by the model updating algorithm for the 50% in 50 year hybrid simulations. The force-displacement hysteresis plots show that for each of these tests behaviour was largely linear, and all of the tests remained linear by the definition given in Section 10.3.4. Table 10-1 gives the values obtained from the updating algorithm for elastic modulus at the end of the simulation. In theory, the values at the end of the simulation are the most accurate values as the largest possible number of timesteps is considered in the calculation, as per Equation 10.1. It is observed that the updated elastic modulus fluctuates initially then stabilizes as the simulations progress. Generally, the initial fluctuations correspond to the initial phase of the ground motion where the amplitude of response is very small. After a number of cycles at greater response amplitudes, the updated values stabilize. This concept is illustrated in Figure 10-24 where the displacement response is superimposed on the graph showing the values of the numerical counterpart weights throughout the SS-GM1-50 simulation.

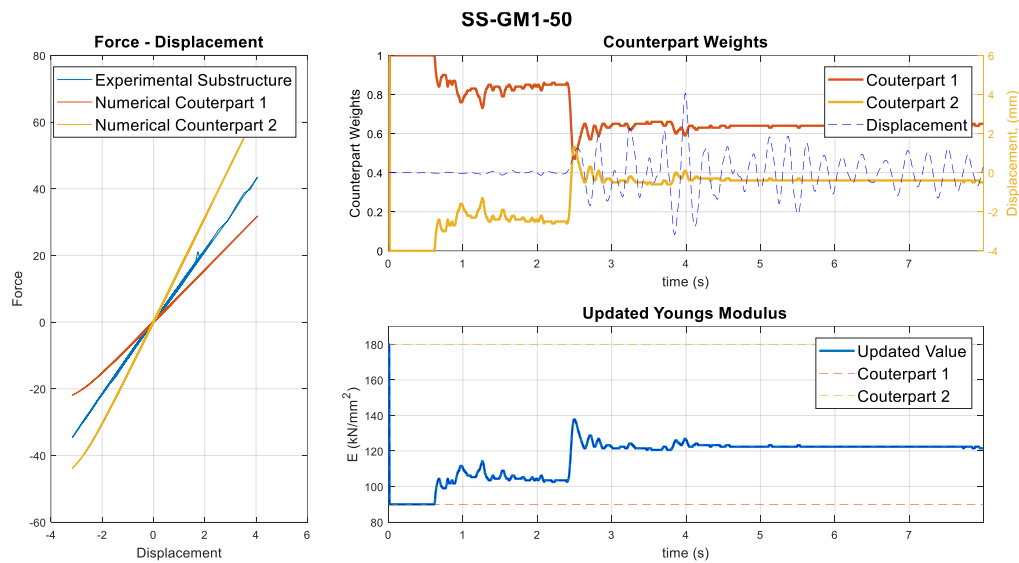


Figure 10-24 Test SS-GM1-50: Updated Elastic Modulus

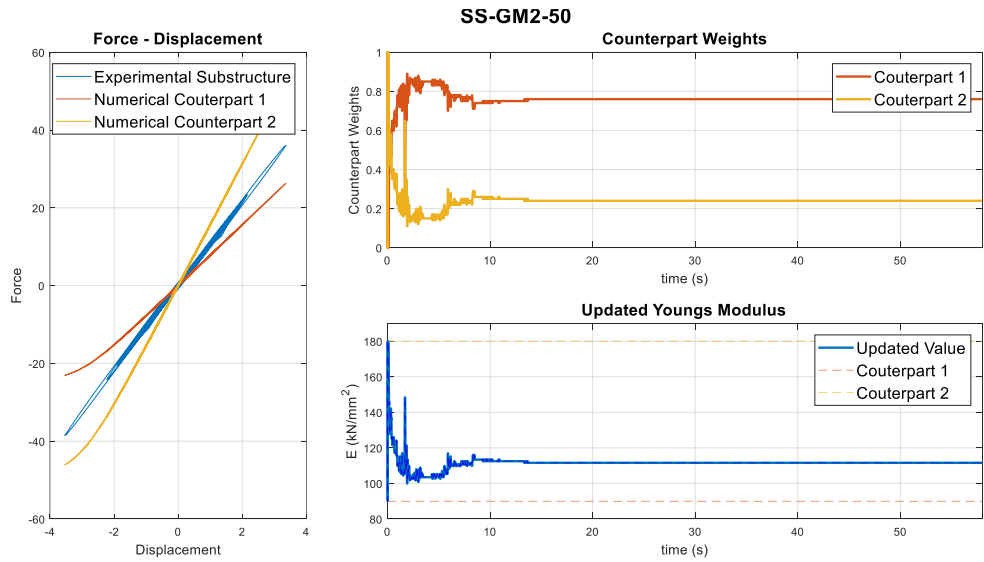


Figure 10-25 Test SS-GM2-50: Updated Elastic Modulus

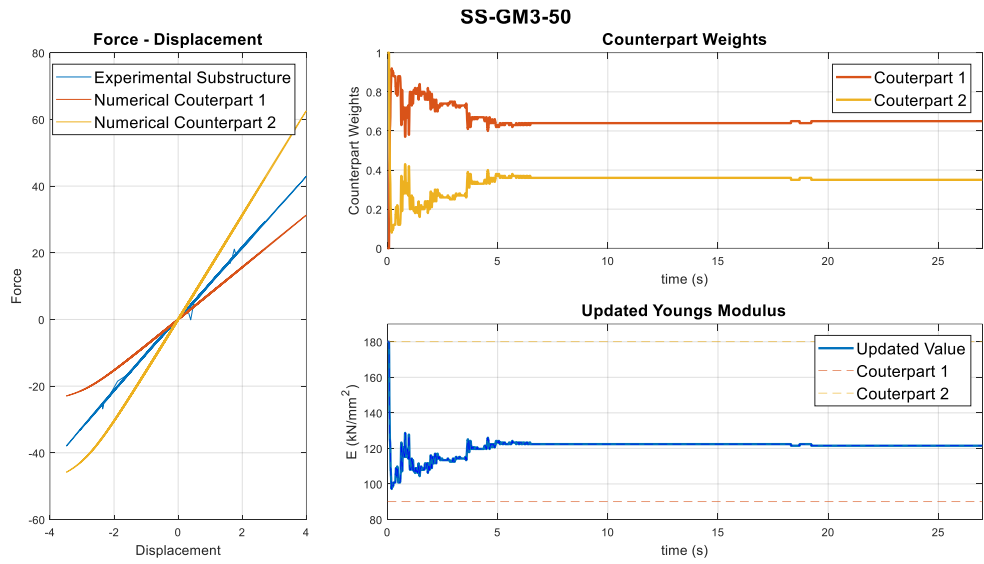


Figure 10-26 Test SS-GM3-50: Updated Elastic Modulus

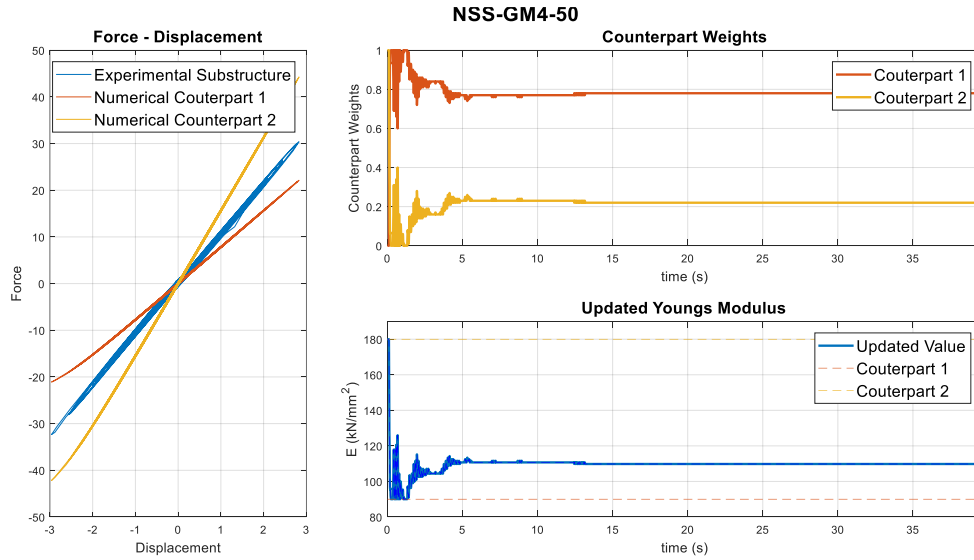


Figure 10-27 Test NSS-GM4-50: Updated Elastic Modulus

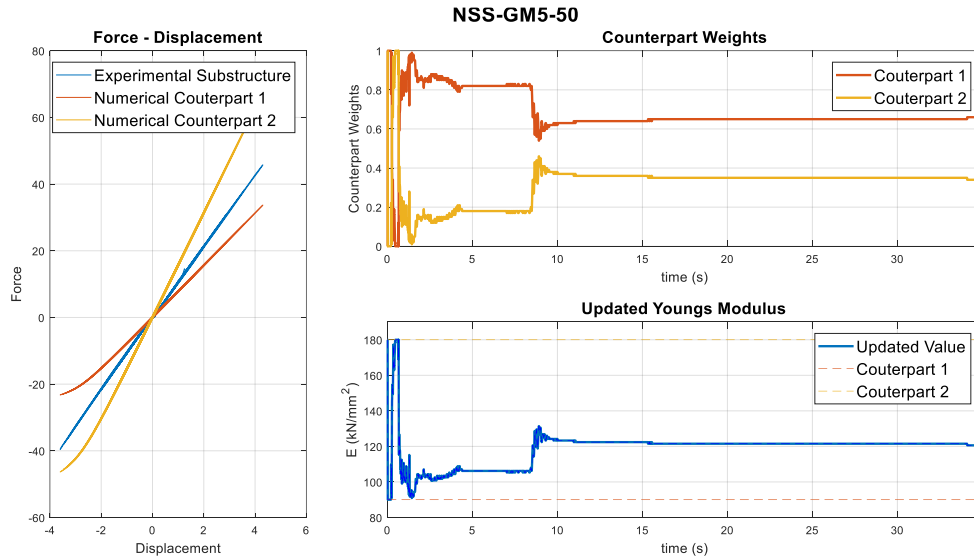


Figure 10-28 Test NSS-GM5-50: Updated Elastic Modulus

The values for elastic modulus recommended by the algorithm range from 108 kN/mm² to 120 kN/mm². Clearly, these are significantly different from the generally assumed values of 200 N/mm² to 210 kN/mm². However, in reality, these calculated values incorporate all differences between the elastic responses of the numerical counterparts and the control substructure, not just the material elastic modulus. Hence, it is more accurate to view these updated elastic modulus values as the effective elastic modulus of the OpenSees model given the differences between the models and the true structure, as opposed to the true value of the elastic modulus of the steel used in the experiment.

Hence, instead of updating the elastic modulus, a term called the Elastic Stiffness Modification Factor (*ESMF*) is proposed. The elastic modulus of each element in the model can be multiplied

by this in order to give a more accurate representation of elastic stiffness. The *ESMF* is a global correction factor that encompasses all differences in modelling stiffness between the control substructure and numerical counterparts rather than just errors in the elastic modulus. There are numerous potential sources of modelling error that can all contribute to these differences in stiffness, for example in the modelling of connections, materials properties, geometry or boundary conditions, while the load history of some of the experimentally tested components may also affect the measured experimental response. Within the model updating algorithm developed it is impossible to separate these sources of uncertainty. This is because the updating algorithm is based on adjusting a modelling parameter so that the response matches the elastic stiffness (i.e. the slope of the force-displacement line) of the control substructure. If separate numerical counterparts representing various sources of modelling uncertainty are used to do this, each parameter will be updated so that changes to that parameter alone lead to the elastic stiffness of the updated substructure matching the control substructure. This is in contrast to the desirable outcome where a combination of modelling parameters are updated so that, together, these updates match the target elastic stiffness. Therefore, because of the limitations of the algorithm, it is necessary to adjust for all potential sources of modelling error using a single global parameter, the *ESMF*.

Table 10-1 presents the calculated values of the *ESMF* for each simulation at the 50% in 50 year intensity level, assuming a true value of 210 kN/mm².

Table 10-1 Values of the elastic modulus proposed by the updating algorithm and the *ESMF*, assuming a true value of $E = 210 \text{ kN/mm}^2$, for the 50% in 50 year hybrid simulations

Analysis Run	E_{Updated} kN/mm ²	Elastic Stiffness Modification Factor ($E = 210 \text{ kN/mm}^2$)
SS-GM1-50	120.6	0.57
SS-GM2-50	110.7	0.54
SS-GM3-50	120.6	0.57
NSS-GM4-50	108.9	0.52
NSS-GM5-50	118.8	0.57

10.4.2 Hybrid Simulations at 10% in 50 Year Ground Motion Intensity

Figures 10-29 to 10-33 show the updated elastic modulus for the simulations carried out at the 10% in 50 year intensity level. In all but one of these, the control substructure is deemed to yield during the course of the test. As for the 50% in 50 year intensity level, the values obtained for the updated elastic modulus are significantly less than the typical values for steel. As before, this is attributable to a combination of several characteristics of the experiment and numerical

models which result in the numerical counterparts being stiffer than the control substructure. The dotted blue lines show the value of elastic modulus obtained when updating is carried out to the end of the simulation, while the solid lines show the value when updating is stopped once yield is deemed to have occurred. Generally these values are reasonably similar, although notable differences are observed in some tests (e.g. SS-GM3-50).

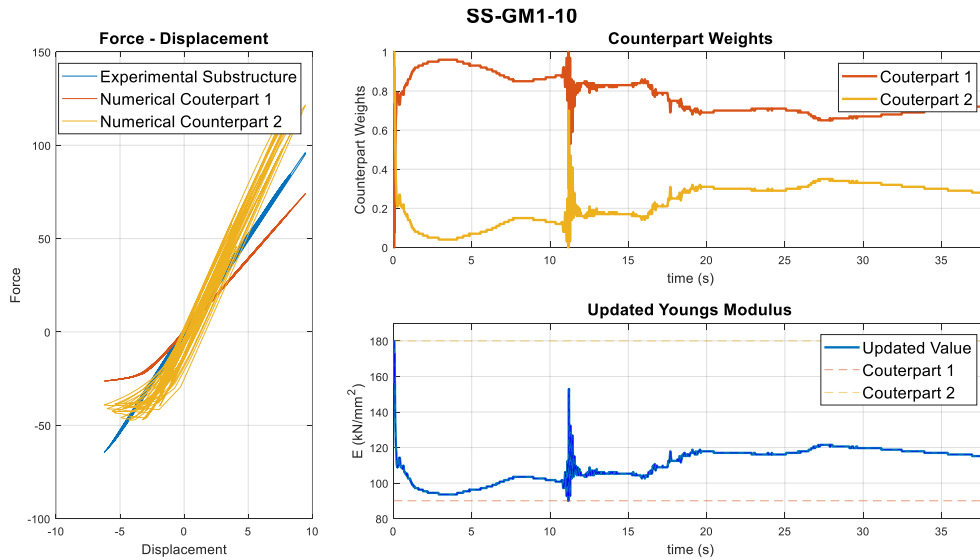


Figure 10-29 Updated elastic modulus from SS-GM1-10; note the experimental substructure is not deemed to have yielded

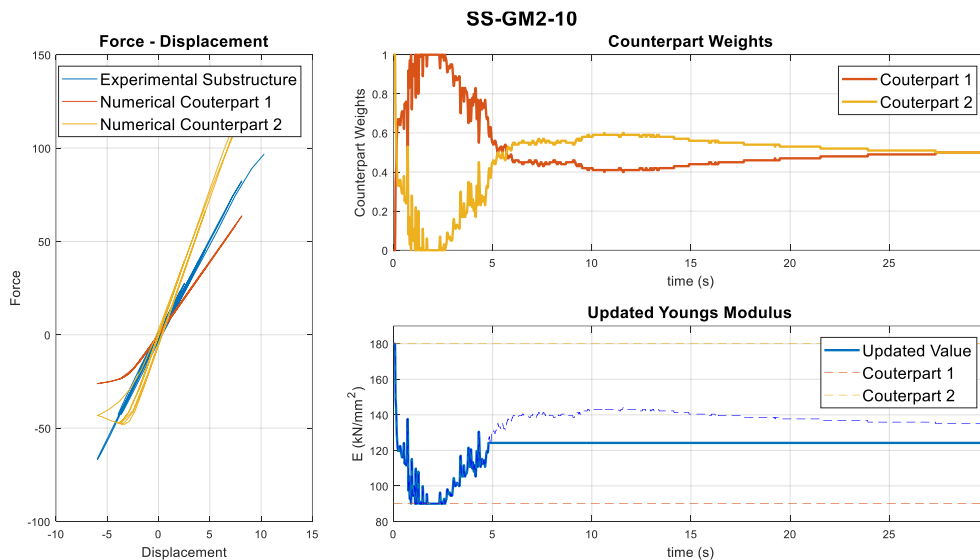


Figure 10-30 Updated elastic modulus from SS-GM2-10

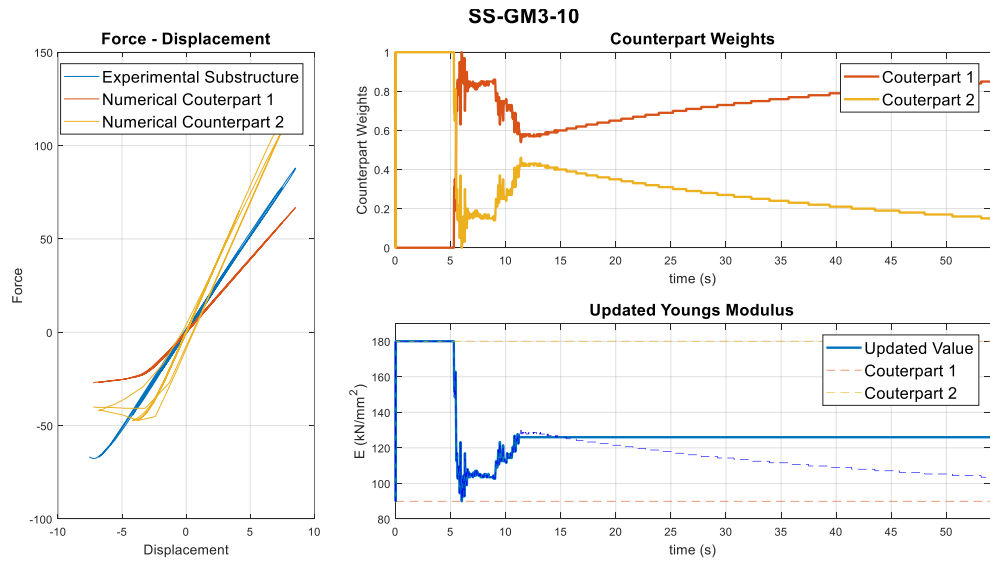


Figure 10-31 Updated elastic modulus from SS-GM3-10

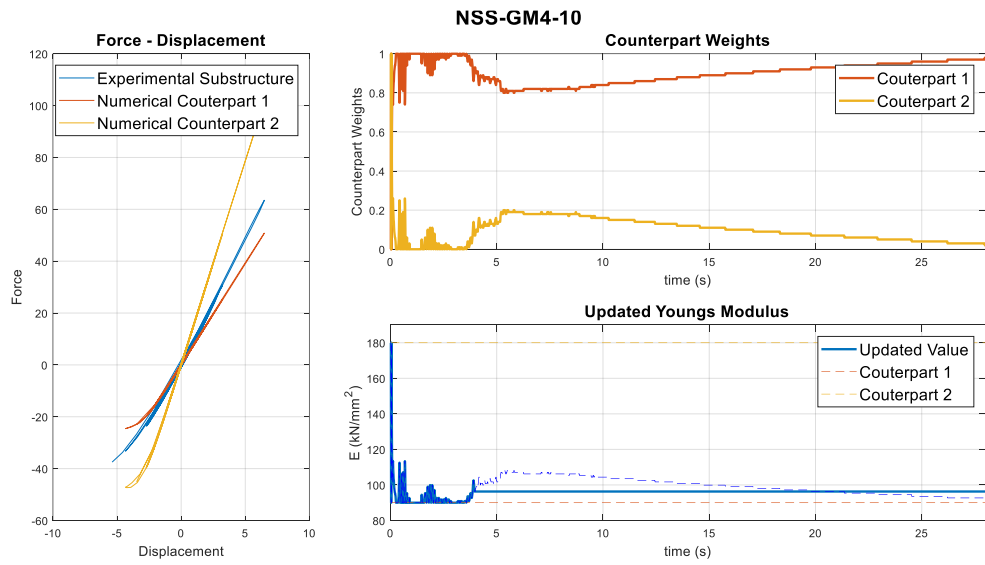


Figure 10-32 Updated elastic modulus from NSS-GM4-10

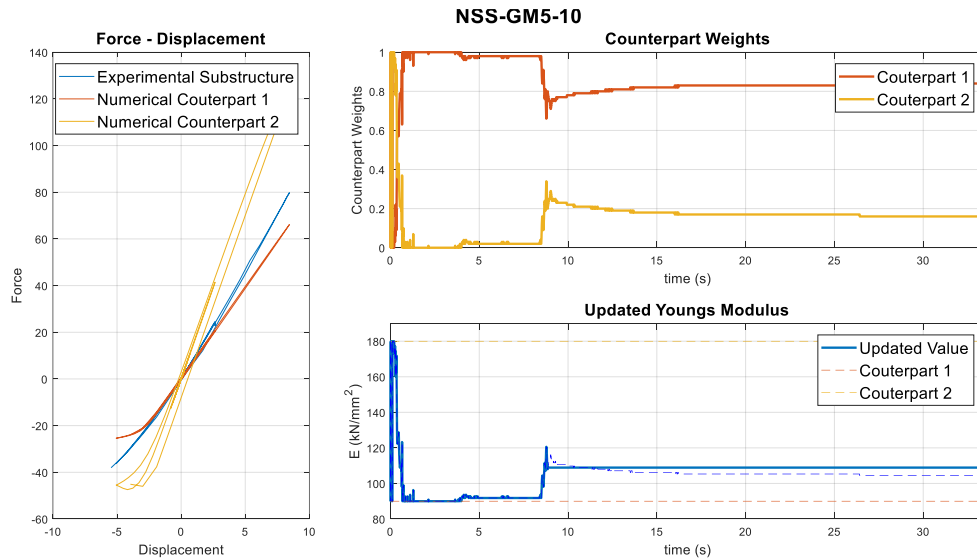


Figure 10-33 Updated elastic modulus from NSS-GM5-10

10.4.2.1 Yield Strength

Figures 10-34 to 10-37 show the updated value for yield strength obtained from the model updating process. In the material coupon tests performed prior to the hybrid simulations an average yield strength value of 294.25 N/mm^2 was obtained. The values obtained from the offline updating are greater than this, with an average of 355 N/mm^2 . This can partly be explained by the shape of the brace section; because of the method of formation cold form steel tubular sections are known to have higher yield strength at the corners and consequently greater yield strength than a coupon machined from one of the faces. However, it is also partly attributable to the fact that the control substructure generally experiences greater forces in the negative direction than the numerical counterparts, irrespective of the yield strength. In general the higher yield strength counterparts compare better to the experimental results while the brace is in compression, meaning these tend to be more heavily weighted in the updating algorithm. However, this is partly compensated for by the fact that the lower yield strength counterparts compare better to the experimental results while the brace is in tension.

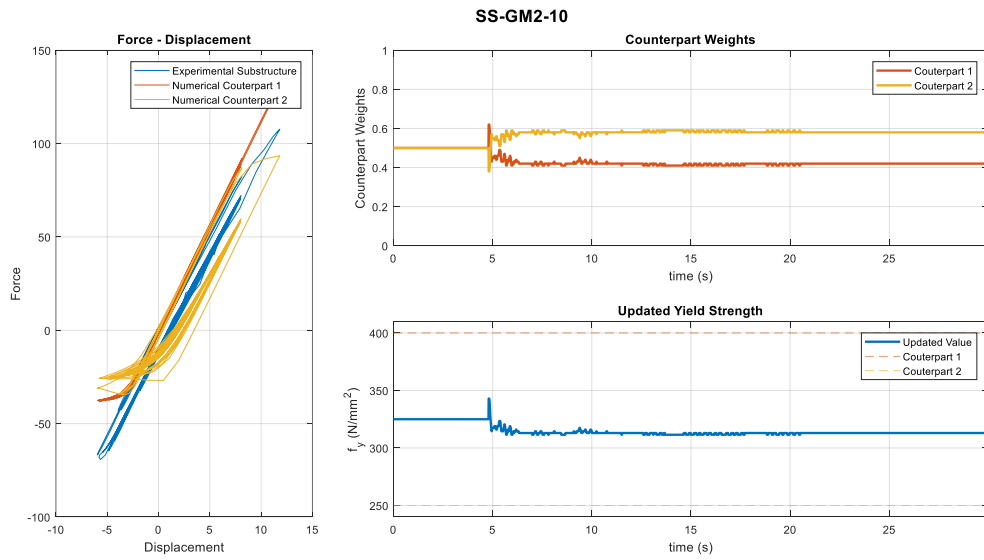


Figure 10-34 Updated yield strength from SS-GM2-10

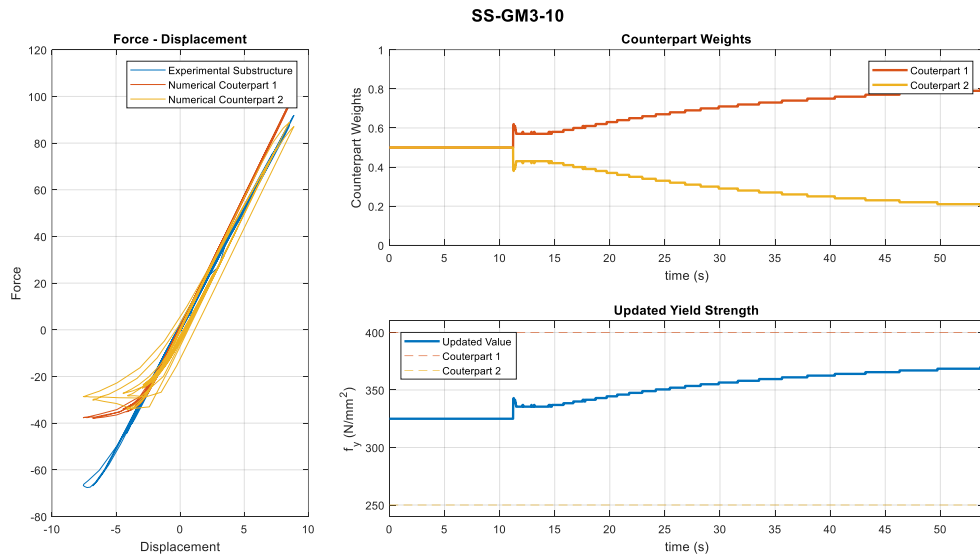


Figure 10-35 Updated yield strength from SS-GM3-10

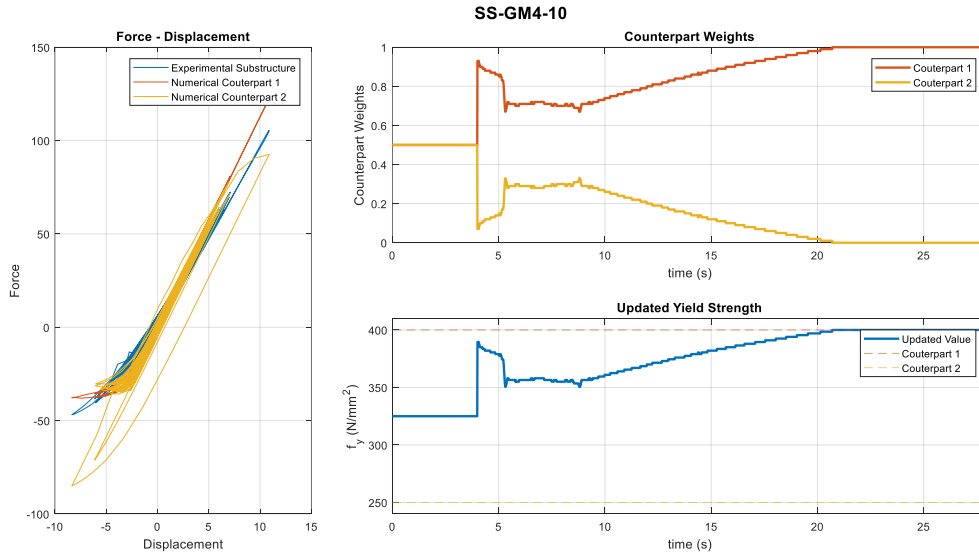


Figure 10-36 Updated yield strength from NSS-GM4-10

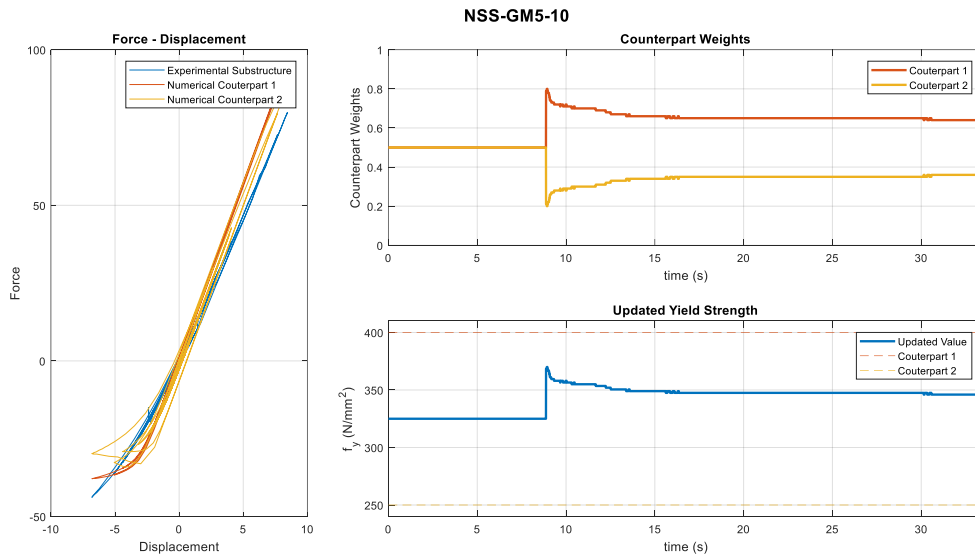


Figure 10-37 Updated yield strength from NSS-GM5-10

10.4.2.2 Initial Camber

Figures 10-38 to 10-41 show the updated values for the brace initial camber obtained from the updating algorithm; the values at the end of the simulation are given in Table 10-2. From this it can be seen that the value of the initial camber recommended by the updating algorithm is generally around 0.2% of the initial brace length. This agrees reasonably well with the value of 0.1% of brace length, as recommended by Uriz et al. (2008), used in the hybrid simulations and elsewhere in this research, and the value 0.3% proposed by Wijesundara (2009). However, it is less than the and 0.8% of brace length used Ryan et al. (2017).

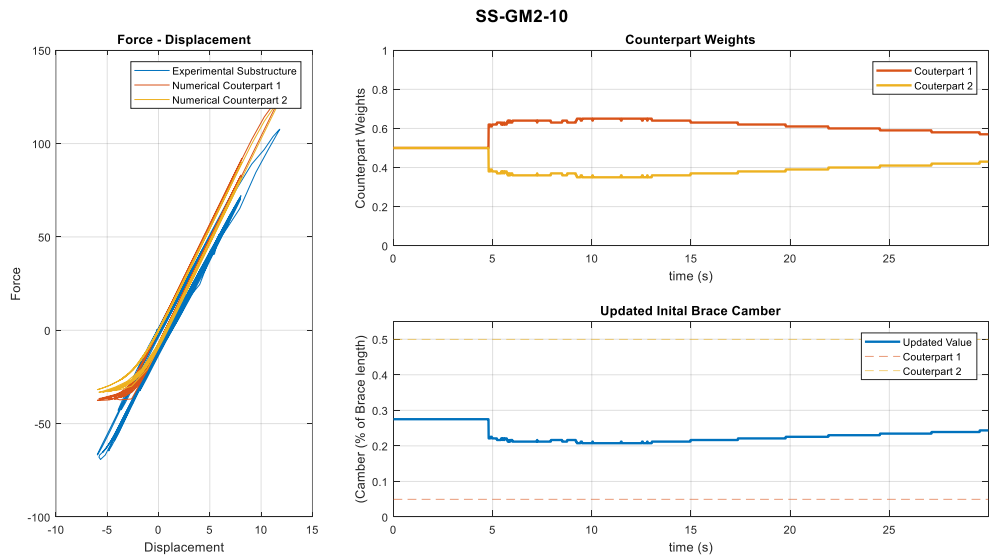


Figure 10-38 Updated initial brace camber from SS-GM2-10

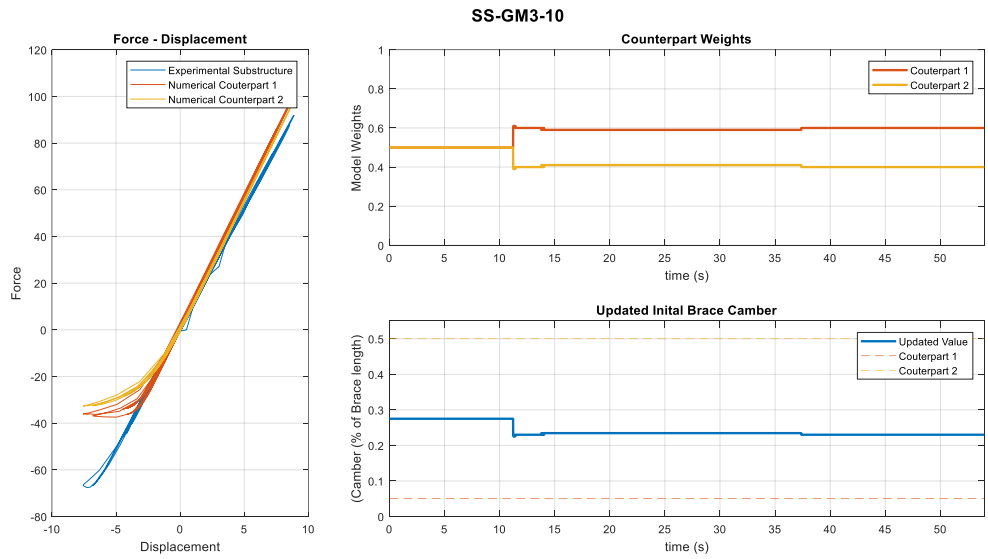


Figure 10-39 Updated initial brace camber from SS-GM3-10

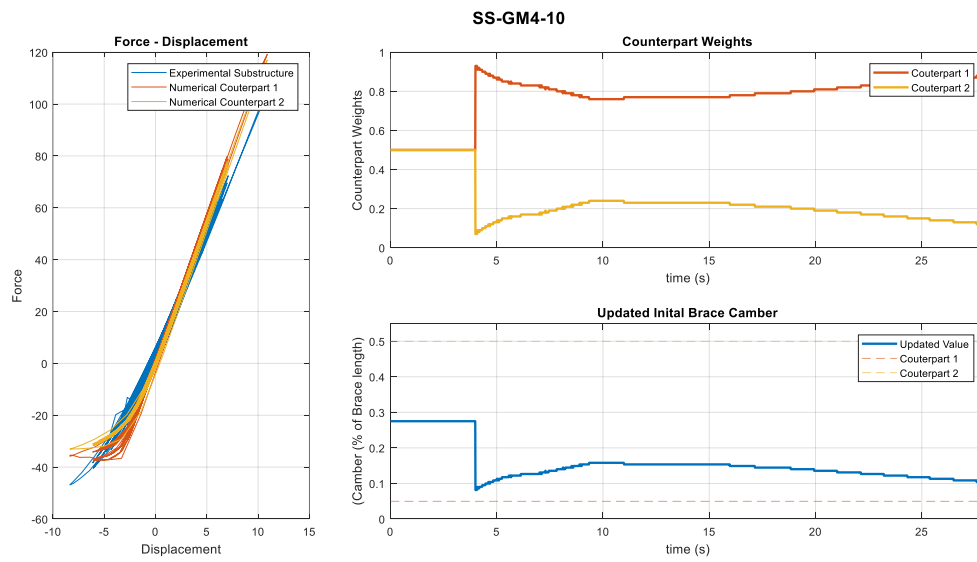


Figure 10-40 Updated initial brace camber from NSS-GM4-10

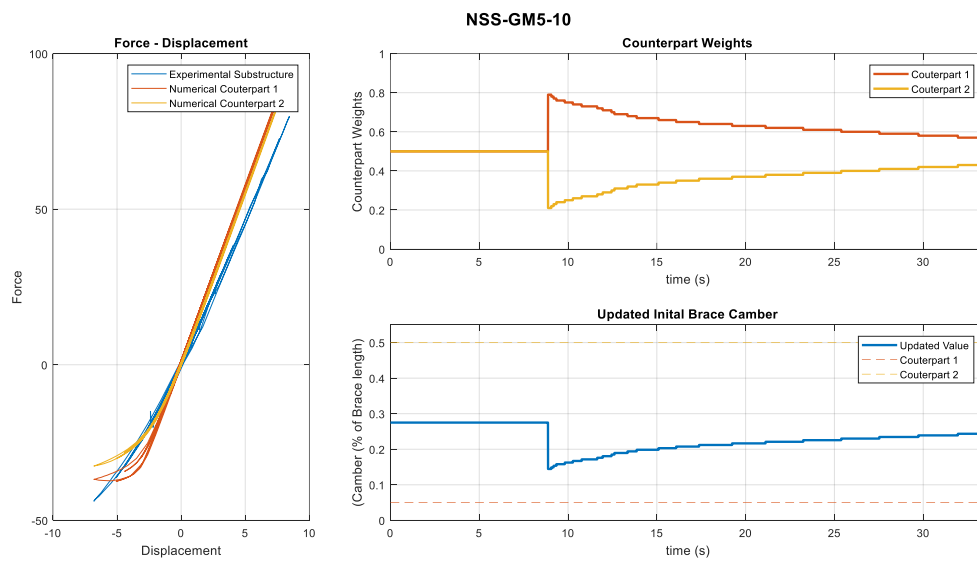


Figure 10-41 Updated initial brace camber from NSS-GM5-10

Table 10-2 Values of the elastic modulus (when updating is stopped at the point of yield and when updating is allowed to continue throughout the test), yield strength and initial brace camber proposed by the updating algorithm and the *ESMF*, assuming a true value of $E = 210 \text{ kN/mm}^2$, for the 10% in 50 year hybrid simulations

Analysis Run	$E_{\text{Updated,}}$ (kN/mm^2) (Updating stopped at yield)	$E_{\text{Updated,}}$ (kN/mm^2) (Updating Continued throughout test)	f_y (N/mm^2)	Initial Brace Camber (% of brace length)	Elastic Stiffness Modification Factor ($E = 210 \text{ kN/mm}^2$)
SS-GM1-10	117	117	N/A	N/A	0.56
SS-GM2-10	124	135	313	0.24%	0.59
SS-GM3-10	126	105	367	0.23%	0.60
NSS-GM4-10	96	90	400	0.10%	0.46
NSS-GM5-10	109	104	346	0.24%	0.52

10.4.2.3 Discussion

A useful way to examine the influence of various parameters on model accuracy is to present the force-displacement hysteresis plots for the numerical counterparts for a parametric analysis where the initial camber employed is varied. Figures 10-42 and 10-43 do this for the SS GM3-10 experiment. In Figure 10-42 the elastic modulus is set as 126 kN/mm^2 ; i.e. the value recommended by the updating algorithm, whereas in Figure 10-43 a more conventional value of 200 kN/mm^2 is adopted.

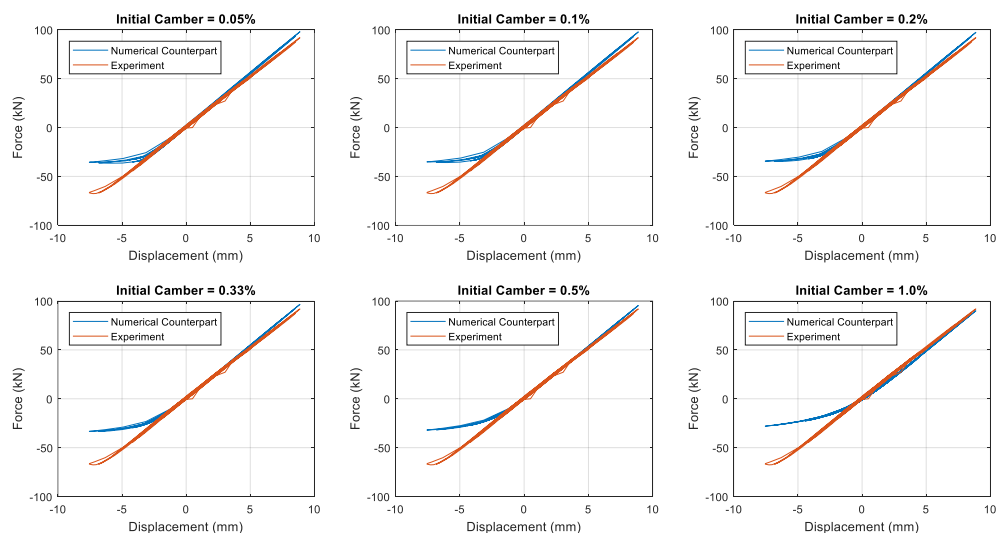


Figure 10-42 SS GM3-10; variation in numerical counterpart response with initial camber: $E = 126 \text{ kN/mm}^2$

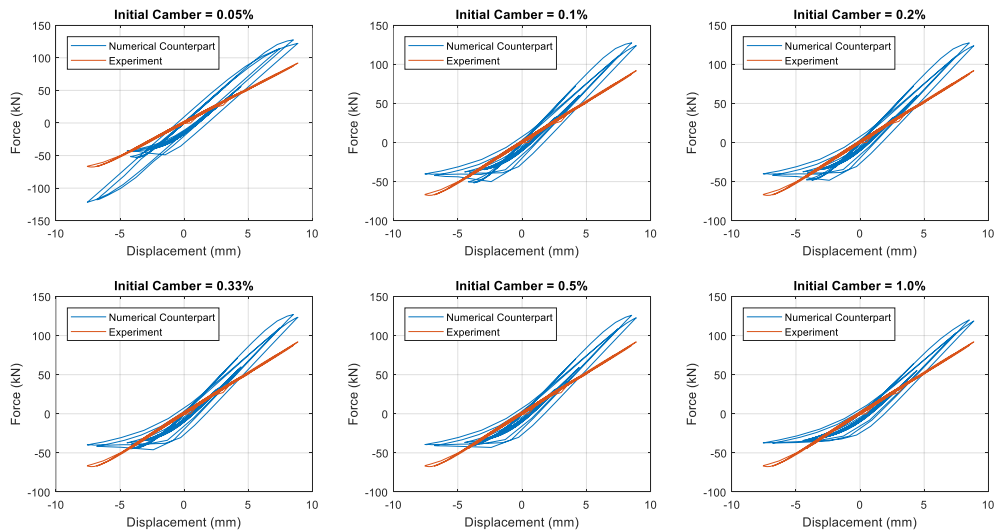


Figure 10-43 SS GM3-10; variation in numerical counterpart response with initial camber: $E = 200 \text{ kN/mm}^2$

A number of trends are noticeable from comparison of Figures 10-42 and 10-43. Firstly, it is clear that a much better match with the experimental data is obtained using the updated, albeit unrealistic, elastic modulus in Figure 10-42. In particular, it is notable that the numerical counterparts with the reduced elastic modulus don't exhibit the same degree of nonlinear behaviour as those modelled using $E = 200 \text{ kN/mm}^2$. This is because of the extra force required to reach the target displacement for the stiffer counterparts induces nonlinear response. This is illustrated conceptually in Figure 10-44. When the results of hybrid simulation were examined in Chapter 9, it was noted that the numerical substructures tended to exhibit a greater degree of nonlinear behaviour than the experimental substructure. Updating the elastic modulus appears to correct this, demonstrating the potential benefits of model updating for improving the accuracy of hybrid simulations.

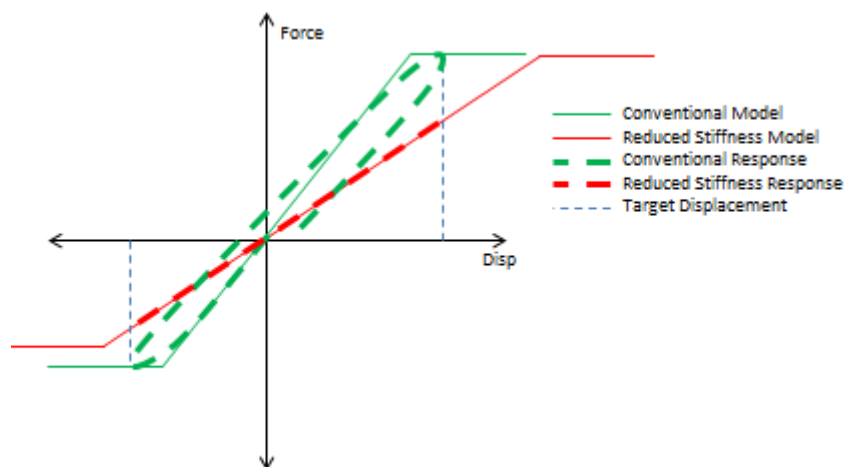


Figure 10-44 Illustration of the reason for greater nonlinear behaviour in a model with conventional stiffness compared to a model with reduced stiffness

Although not as obvious, it is also noticeable that the higher, more realistic, elastic modulus value (Figure 10-43) gives a better prediction of the peak force in the negative direction, i.e. when the brace is in compression. This is reasonable as the critical buckling load of a strut is proportional to its elastic modulus, and if the elastic modulus is artificially reduced, a corresponding underestimation in buckling load is to be expected. Thus the more realistic elastic modulus appears gives a closer match for peak compressive force but a better overall match with the experimental data is achieved using the updated value.

The value of initial camber used in the numerical counterpart is clearly less influential than the value of elastic modulus, especially for the lower elastic modulus analysis. A lower value for initial brace camber generally gives a slightly improved prediction of the peak compression force whereas a higher initial camber appears to give a better match for positive displacement, i.e. when the brace is in tension. However, neither of these trends is particularly strong or noticeable in comparison to the influence of elastic modulus. Ultimately, there is arguably not enough nonlinear or buckling behaviour exhibited by the test structure to definitively calibrate the brace initial camber. Ideally, such strongly nonlinear behaviour would have been obtained for the tests at the 2% in 50 year intensity level, but this was not possible because of the failure of the component in the test system.

10.4.3 Recommendations for Numerical Modelling

The main idea to emerge from the offline model updating is that the conventionally modelled numerical counterparts appear to be too stiff. This basic conclusion is in agreement with other work that has attempted to calibrate similar numerical models of CBFs using experimental data (e.g. Ryan et al. (2017)). It appears to be possible to achieve a better match between the experimental and numerical force-displacement hysteresis by employing an Elastic Stiffness Modification Factor (*ESMF*) that reduces the elastic stiffness of the numerical models by factoring down the elastic modulus of the material used. It is important to emphasise that the purpose of updating the elastic stiffness is not to identify the true value of the elastic modulus of steel, but to account for global modelling uncertainty. The model updating scheme adopted here suggests a value of approximately 0.6 for the *ESMF*, although intuitively reducing the elastic modulus to 60% of the conventionally assumed value appears extreme and it is possible that the load history of some of the components in the experimental tests contribute to this low value.

For the range of earthquakes examined, the initial camber of the bracing member does not appear to be as influential on the response as the elastic stiffness. Nevertheless, an average value of 0.2% of brace length is recommended by the model updating scheme from a limited

number of tests. However, it is possible that an improved value of this parameter could be obtained by performing updating for more extreme earthquake events where a greater degree of nonlinear response is observed.

10.5 Conclusion

The Chapter details the development and verification of a model updating algorithm for hybrid simulation designed to improve OpenSees modelling accuracy. The method uses experimentally measured data and sets of numerical counterparts to update modelling parameters. One of the primary advantages to this algorithm over previously developed methods is that it can be implemented entirely within the OpenSees-OpenFresco framework. This allows for reasonably straightforward combination with regular hybrid simulations, without requiring time to be spent developing and managing further links between different software. From a numerical modelling viewpoint, the algorithm also allows modelling parameters to be isolated and examined individually, which can provide useful information about the optimum value of these parameters to use in future analytical models. However, there are limitations in the algorithm's ability to find unique updating solutions in the elastic range, meaning global correction factor accounting for multiple sources of uncertainty is required.

Although initially intended to be deployed for Online Model Updating, damage to the test system meant this was not possible during the course of this research. Instead the algorithm was validated using numerical analyses, which showed that the updating strategy is capable of identifying correct parametric values of elastic modulus and yield strength. Offline Model Updating was then employed to examine the results of previous hybrid simulations that had been performed without updating. The optimum values of elastic modulus, yield strength and initial brace camber were investigated. It was shown that the numerical counterpart models employed consistently overestimate the stiffness of the test structure. A parameter, termed the Elastic Stiffness Correction Factor (*ESMF*), was proposed to correct for this excessive stiffness. A value of approximately 0.6 was recommended for the *ESMF*.

11 Review of Experimental Results

11.1 Introduction

The overarching aim of this Chapter is to re-examine the conclusions reached in the Chapters 6 and 7 through numerical modelling, in light of the experimental results presented in Chapters 9 and 10. The experimental data presented in Chapter 9 suggests that the conventional modelling assumptions used in Chapters 6 and 7, generally return results that are reasonably, but not entirely accurate. The offline model updating undertaken in Chapter 10 suggests that improved model accuracy can be achieved by applying optimised modelling parameters, in particular through reducing the model stiffness. In terms of performance assessment, two main questions arise from this. Firstly, what impact to these modelling inaccuracies have on calculated performance measures, and secondly to what extent do numerical models using optimised modelling parameters change these measures? Furthermore, the experimental results obtained present an opportunity for the *EDP* prediction models developed in Chapter 7 to be tested.

The Chapter begins by using results of shake table tests from a previous research project to further investigate the ability of numerical models to predict *EDPs* for performance assessment. This is done using both conventional models and models with reduced elastic stiffness based on the analysis of hybrid simulation results, allowing the accuracy of various models to be assessed. Using both experimental and numerical *EDPs*, performance assessment is then carried out to examine the impact of these *EDPs*, and the associated inaccuracies, on performance metrics. The impact of reduced stiffness models is then further assessed using some of the case study buildings previously examined in Chapter 6. Finally, the rapid prediction methods developed in Chapter 7 are assessed, and improved, using experimental results.

11.2 Investigation of Modelling Accuracy using BRACED Shaking Table

Tests

In Chapters 9 and 10, model accuracy was assessed by comparing results from numerical models to hybrid simulations. In general, it was concluded that the while reasonable predictions of *EDPs* are obtained, numerical models tend to overestimate structural stiffness. However, these statements about model accuracy assume that the results of the hybrid simulations are perfectly correct, which given the complexity of the method, simplifications to allow implementation and various other potential sources of error, is a questionable assumption. Hence, in order to further explore the accuracy of the models employed, results of previous shaking table tests, referred to as the BRACED Shaking Table Tests, are also considered.

11.2.1 BRACED Shaking Table Tests

The BRACED (Brace Response and Assessment: Computation, Experiments and Design) Project, carried out within the Seismic Engineering Research Infrastructure for European Synergies (SERIES) program, investigated the response of CBFs under seismic excitation (SERIES, 2013). The project involved collaboration between researchers in Trinity College Dublin, National University of Ireland Galway, University of Ljubljana, Imperial College London, University of Liege and the Commissariat à l'Énergie Atomique (CEA). A central element of this project was a series of shake table experiments on full-scale CBFs. Full details of the test setup and results for the entire project have been reported elsewhere (SERIES, 2013, Broderick et al., 2015, Ryan et al., 2017, Salawdeh et al., 2017).

11.2.1.1 Test Frame Details

The BRACED program included over 150 shake table tests performed at CEA Saclay, near Paris. The large platform area and table acceleration capacity allowed the ultimate response of brace members of a realistic size to be investigated. In order to investigate a range of feasible designs, shake table tests were performed for a series of single storey CBFs, illustrated in Figure 11-1, with 12 different combinations of brace members and gusset plate connections. Experiments were performed at ground motion intensity levels approximately corresponding to 2%, 10% and 50% probability of exceedance in 50 years for the Bulk Mail site in Los Angeles, the same site used for the hybrid simulations in this study. However, these analyses were performed using a single ground motion record (from the El Centro earthquake, PEER ID 0006) incrementally scaled to match the Uniform Hazard Spectrum (UHS) at the site, as opposed to the Multiple Stripe (MSA) record selection approach based on the Conditional Spectrum (CS) adopted in this study.

The basic frame setup for the BRACED tests is shown in Figure 11-1. The beam and column assembly illustrated was employed to examine a series of brace and gusset plate combinations. Tests were performed for gusset plates designed using the conventionally employed standard linear clearance model (SLC) and the recently developed balanced design method using an elliptical clearance (EC) zone in the gusset plate (Roeder et al., 2011). Gusset plates connected to beam only, as well as more conventional connections to both the beam and column, were tested. Details of the various combinations of brace sections and gusset plate designs and connection methods tested as part of the experimental program are given in Table 11-1.

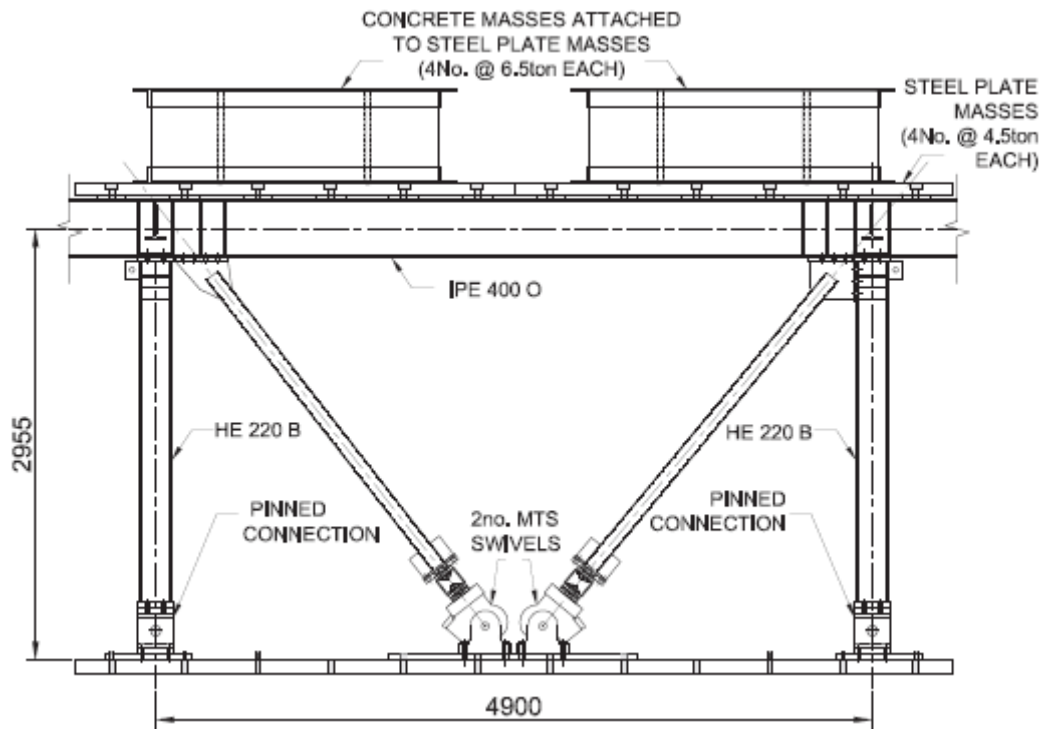


Figure 11-1 Schematic of test frame used in BRACED shaking table tests. Both gusset connection types are shown in the one diagram; the gusset plate on the left is connected to the beam only, while on the right a more conventional connection to both the beam and column is shown. In reality, the same connection was employed for both gusset plates in each individual test.

Table 11-1 Details of the BRACED shake table test

Test	Designation	Brace (SHS or RHS)	Gusset Plate		
			Connected to:	Design Method	t_{plate} (mm)
1	S1-CA-G1	80x80x3.0	Beam & Column	SLC	12
2	S3-CA-G1	80x40x3.0	Beam & Column	SLC	8
3	S4-CA-G1	60x60x3.0	Beam & Column	SLC	8
4	S2-CA-G1	100x50x3.0	Beam & Column	SLC	12
5	S1-CA-G2	80x80x3.0	Beam & Column	EC	5
6	S2-CA-G2	100x50x3.0	Beam & Column	EC	4
7	S3-CA-G2	80x40x3.0	Beam & Column	EC	4
8	S1-CB-G1	80x80x3.0	Beam only	SLC	12
9	S2-CB-G1	100x50x3.0	Beam only	SLC	12
10	S4-CB-G2	60x60x3.0	Beam only	EC	4
11	S2-CB-G2	100x50x3.0	Beam only	EC	4
12	S3-CB-G2	80x40x3.0	Beam only	EC	4

11.2.2 OpenSees Modelling of BRACED Tests

In order to assess the modelling assumptions adopted in previous Chapters, models of the experimentally tested frames were developed in OpenSees using these modelling practices. Results from the experiments were compared with the *EDPs* calculated numerically from NLTHA.

Previous studies have attempted to develop OpenSees models that accurately capture the experimental response of the test frame by varying different parameters, such as the brace initial camber or the beam-column connection model (Hunt, 2013, Ryan et al., 2017). However, the focus of this work is slightly different; the aim is to assess the accuracy of conventionally employed modelling assumptions, such as those adopted elsewhere in literature (e.g. in Del Gobbo et al. (2018)) and in previous chapters in this study, as opposed to optimising modelling parameters in order to achieve a good match with experimental results. In that sense the numerical modelling undertaken here can be partly thought of as a 'blind' prediction of the shake table response.

For this study, where the impact of gusset plate design on CBF response is not a particular focus, comparison was only made between experimental and numerical results for the tests performed with gusset plate connected to both the beam and column, i.e. tests 1 to 7 as listed in Table 11-1. The experiments where the gusset plate is only connected to the beam are not of particular interest for this study as this is not a conventionally employed connection method and these test runs have proved particularly challenging to model accurately in previous work (Ryan et al., 2017).

As before, the brace elements were modelled using two force based nonlinearBeamColumn elements with 3 integration points per element and an initial camber displacement of 0.1% of the brace length (Uriz et al., 2008), with gusset plate connections modelled using nonlinear rotational springs (Hsiao et al., 2012). The beams and columns were modelled using force based nonlinearBeamColumn elements with 3 integration points and discretized fiber sections. The Steel02 material, which represents the Giuffre-Menegotto-Pinto model, with the yield strength as measured in the experimental program, was used for all components. In order to capture the out of plane buckling facilitated by the swivel connections at the bottom of the brace, it is necessary to model the test frame in three dimensions. The boundary conditions at the base of the frame are modelled as recommended by Ryan et al. (2017), which means allowing rotation about the z axis at the base of both the columns and swivels, as well as about the x axis for the swivels. For all analysis 5% Rayleigh damping proportional to the mass and tangent stiffness matrix at the first and third modes was applied.

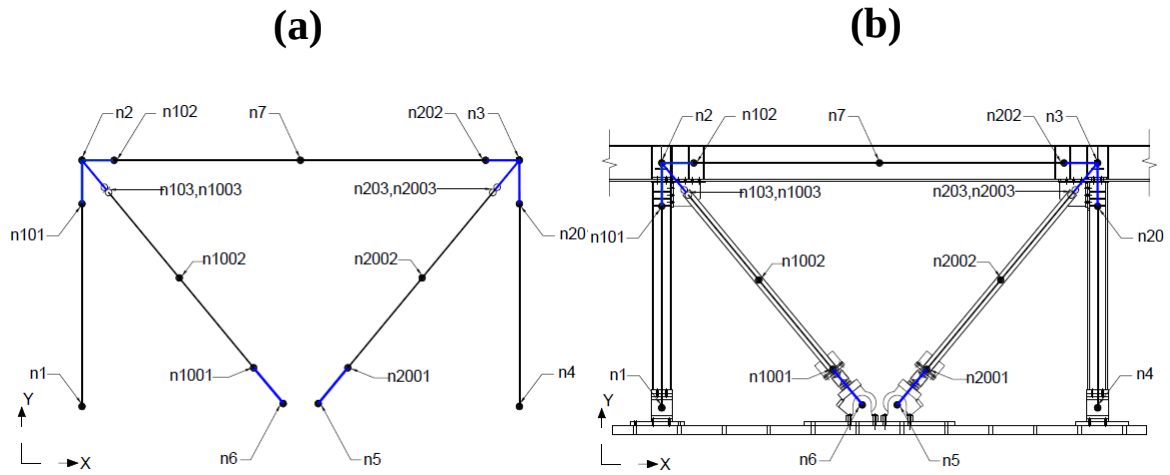


Figure 11-2 (a) OpenSees model of the BRACED test frame and (b) model superimposed on schematic of test structure

The actual ground motions caused by the shaking table during the tests were measured and recorded as they differed slightly from the prescribed motions. These recorded ground motions were applied to OpenSees models in order to perform NLTHA. Figures 11-3 and 11-4 compare the experimentally recorded and numerically calculated response for two sample test runs. The force-displacement hysteresis from the experimental test is approximated as the product of the roof acceleration and the seismic mass; clearly there is a significant amount of noise in this data, however the stiffness of the test structure and whether yielding or significant buckling occurs can be generally appreciated. The peak storey drift and floor accelerations recordings from the tests were then compared to the corresponding calculated values in order to assess model accuracy.

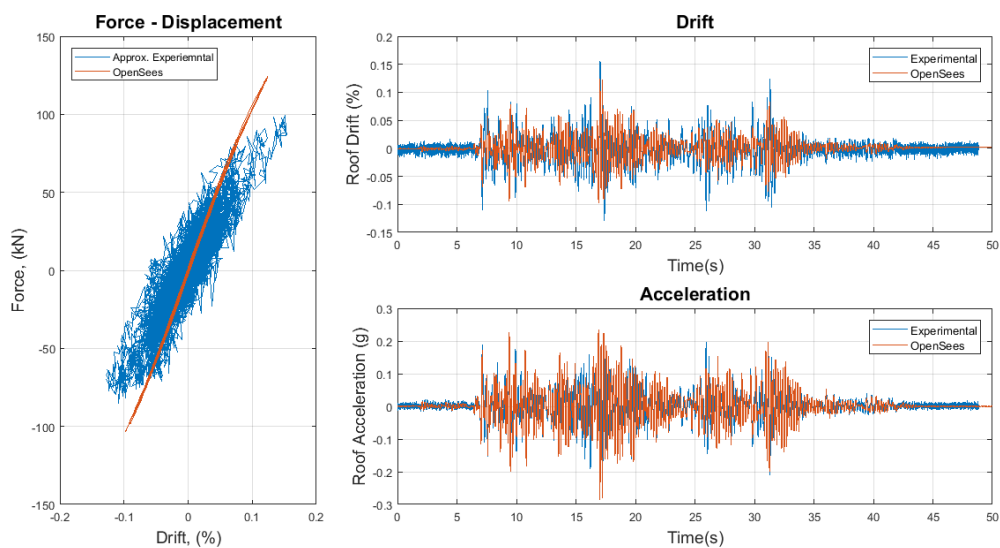


Figure 11-3 Comparison of experimental and numerical response for test run S4-CA-G1 at the 50% in 50 year intensity level

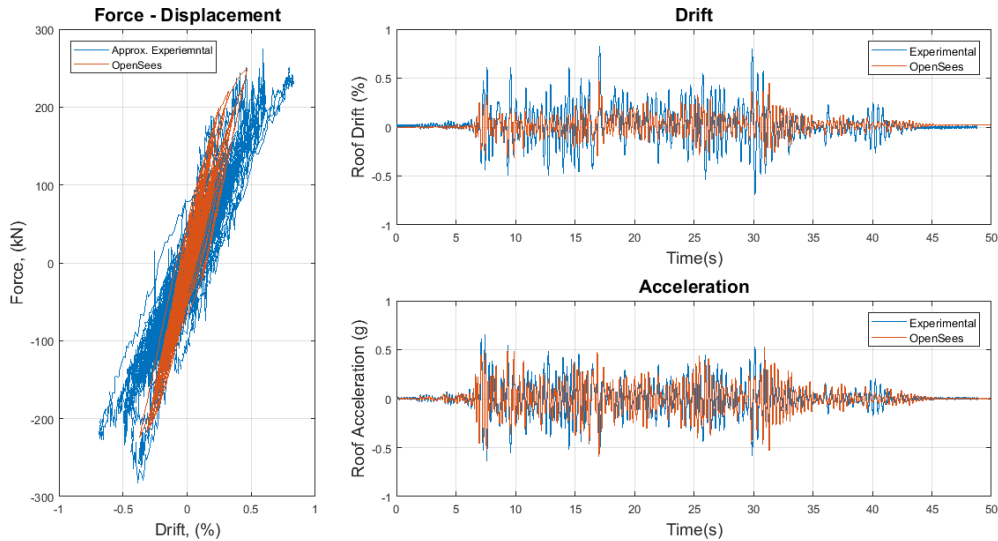


Figure 11-4 Comparison of experimental and numerical response for test run S4-CA-G1 at the 2% in 50 year intensity level

11.2.2.1 EDP Comparison

Tables 11-2 and 11-3 compare the measured and calculated *EDPs*. Figure 11-5 presents the numerically calculated *EDPs* normalized by the corresponding experimentally recorded value. Clearly, the ideal value for these is 1, which would mean the value of the *EDP* calculated in OpenSees equals the experimental value. The results are presented for tests performed at the 50% (SLE), 10% (DLE) and 2% (MCE) in 50 year intensity levels. The response is generally elastic at the SLE, with nonlinear behaviour generally experienced at the DLE and MCE. Analysis was performed for 7 different brace and gusset plate configurations, as detailed in Table 11-1. The dashed lines in Figure 11-5 show the ratio of numerical to experimental *EDPs* for individual brace and gusset plate configurations, with the thick green line showing the mean value.

The numerical storey drift values are always less than the experimental values. This is true irrespective of the ground motion intensity and whether the frames behaved elastically or inelastically. On average the numerical model returns storey drift values of approximately 60% of the measured value. There is a slight reduction in this ratio for inelastic response, i.e. comparing the 10% in 50 year and 50% in 50 year intensity levels; however the main difference is the increase in variation.

In terms of peak floor acceleration, the model over-predicts response at the 50% in 50 year intensity level, with the average ratio of numerical to experimental acceleration being approximately 1.35. However, at the 10% and 2% in 50 year intensity levels the model appears to capture peak floor acceleration response well, with average calculated values within 10% of those observed in the experiment. This is attributable to the fact that the acceleration response

at these intensity levels, where the test structure is behaving inelastically, is strongly influenced by the material yield strength and brace resistance which are amongst the easier properties to model correctly.

Table 11-2 Comparison between experimentally recorded and numerically calculated drift (in %)

Test ID	Experimental			Numerical		
	50% in 50	10% in 50	2% in 50	50% in 50	10% in 50	2% in 50
S4-CA-G1	0.155	0.353	0.829	0.124	0.278	0.470
S1-CA-G1	0.202	0.414	0.100	0.136	0.264	0.590
S3-CA-G1	0.180	0.449	1.085	0.108	0.309	0.864
S2-CA-G1	0.221	0.634	1.536	0.149	0.359	0.594
S1-CA-G2	0.255	0.667	1.807	0.156	0.271	0.601
S2-CA-G2	0.177	0.651	N/A	0.136	0.368	N/A
S3-CA-G2	0.199	0.545	1.598	0.144	0.337	1.215

Table 11-3 Comparison between experimentally recorded and numerically calculated acceleration (in g)

Test ID	Experimental			Numerical		
	50% in 50	10% in 50	2% in 50	50% in 50	10% in 50	2% in 50
S4-CA-G1	0.197	0.416	0.655	0.284	0.486	0.589
S1-CA-G1	0.292	0.502	0.853	0.474	0.669	0.802
S3-CA-G1	0.200	0.438	0.836	0.205	0.483	0.685
S2-CA-G1	0.292	0.622	0.936	0.390	0.662	0.758
S1-CA-G2	0.270	0.690	1.373	0.482	0.645	0.794
S2-CA-G2	0.281	0.640	N/A	0.323	0.644	N/A
S3-CA-G2	0.235	0.465	0.833	0.255	0.498	0.808

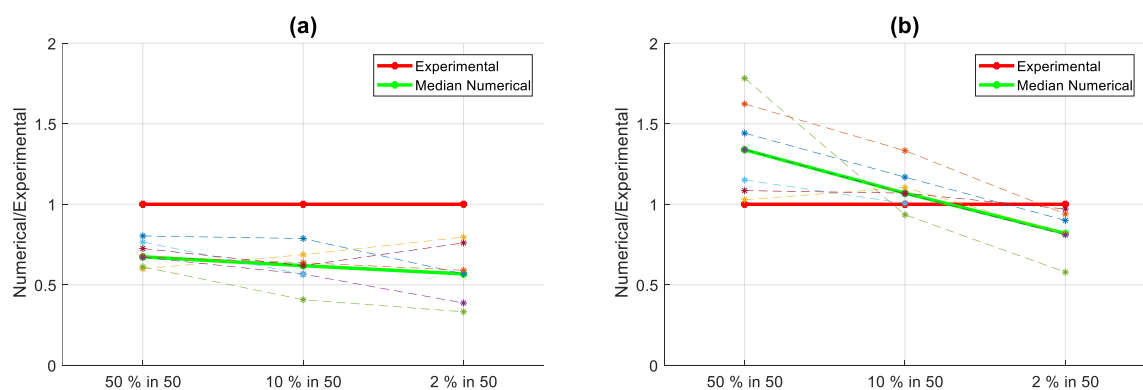


Figure 11-5 (a) Comparison of experimentally recorded peak inter-storey drift with values obtained from OpenSees model; (b) Comparison of experimentally recorded peak floor acceleration with values obtained from OpenSees model

The respective under and over estimation of peak drift and peak elastic acceleration response were shown to be statistically significant. A null hypothesis stating that if an infinite number of experiment-simulation pairs were performed the average ratio of numerical to experimental

EDPs at each intensity level would equal 1 was proposed, i.e. that the apparent biases seen in Figure 11-5 only happened by chance and are simply due to the low number of analyses. The probability of this being the case, i.e. the p value, was calculated using a student's t test and it was shown that for drift at all levels and acceleration at the 50% in 50 year intensity level the p value was greater than 0.05. This is generally taken to mean that the null hypothesis is not statistically significant, in this case meaning that the apparent modelling biases are not due to chance. Therefore it can be concluded that the errors in the modelling process above are significant and would be observed even with an infinite number of tests.

In summary, the comparison between the model results, generated using conventionally adopted modelling assumptions, and the experimental *EDPs* indicate that the structural models employed are generally stiffer than the experimental test structures. This can also be seen by comparing the hysteresis plots in Figures 11-3 and 11-4, and agrees with the conclusions from the hybrid simulation program. From an *EDP* estimation viewpoint, this means that peak drifts tend to be underestimated while peak floor accelerations are overestimated while the structure is behaving elastically.

11.2.3 Updated Modelling Assumptions

The concept of altering the frame stiffness using an Elastic Stiffness Modification Factor (*ESMF*) to alter then elastic modulus of the material used in the model emerged from the offline model updating undertaken in Chapter 10. An average *ESMF* value of approximately 0.6 was proposed by the model updating algorithm employed, although intuitively this value appears extreme. The comparison between the experimental and numerical results for the BRACED shake table tests presented in Section 11.3 also suggests that the numerical models employed are too stiff. Hence, it was decided to rerun the numerical analyses of the BRACED tests with *ESMFs* of 0.6, 0.7, 0.8 and 0.9. For all analyses the brace initial camber was kept constant at 0.1% of the brace length.

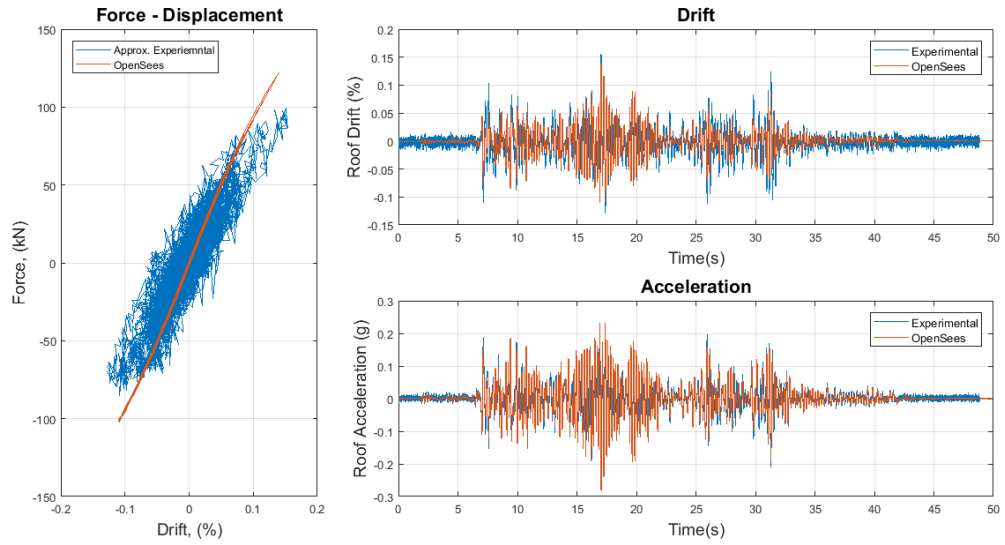


Figure 11-6 Comparison of experimental and numerical response using an *ESMF* of 0.9 for test run S4-CA-G1 at the 50% in 50 year intensity level

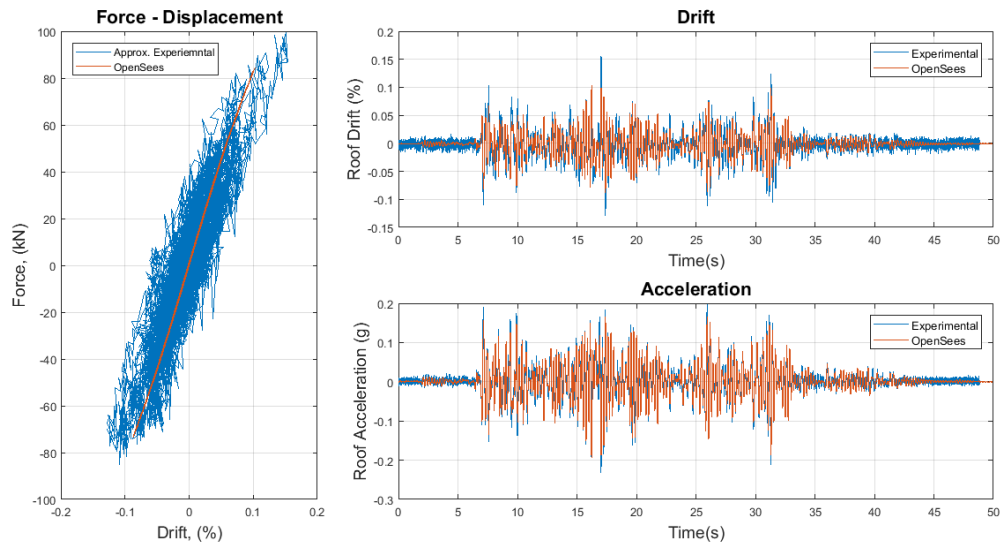


Figure 11-7 Comparison of experimental and numerical response using an *ESMF* of 0.8 for test run S4-CA-G1 at the 50% in 50 year intensity level

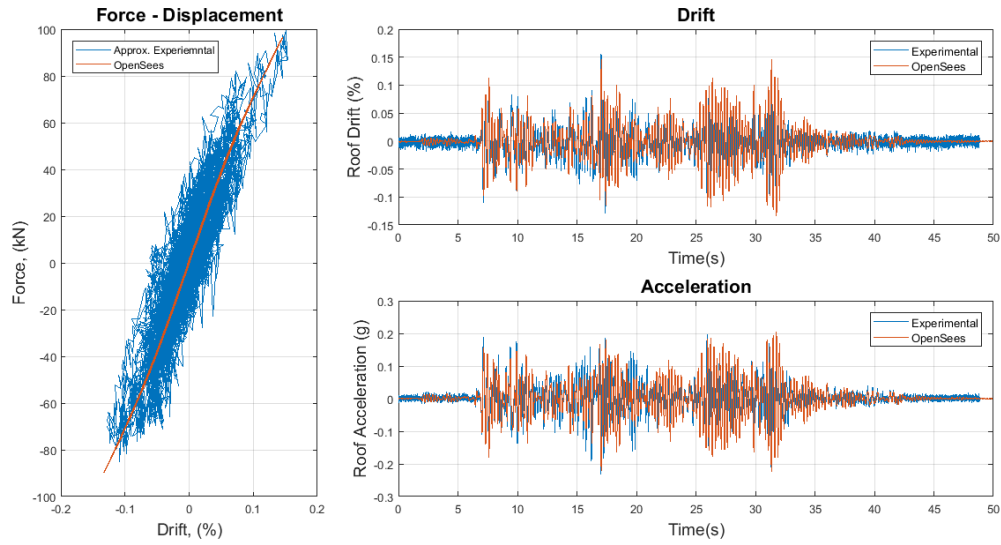


Figure 11-8 Comparison of experimental and numerical response using an *ESMF* of 0.7 for test run S4-CA-G1 at the 50% in 50 year intensity level

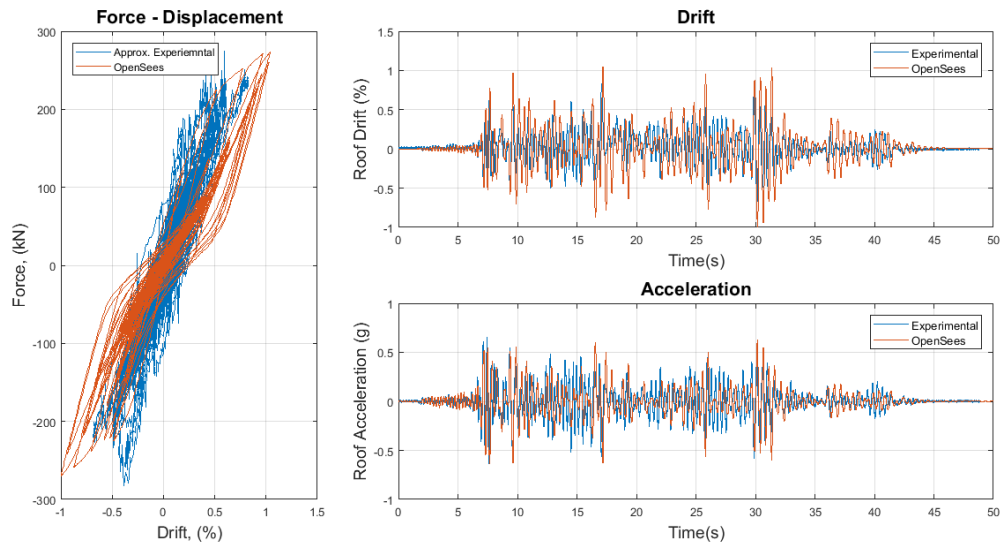


Figure 11-9 Comparison of experimental and numerical response using an *ESMF* of 0.7 for test run S4-CA-G1 at the 2% in 50 year intensity level

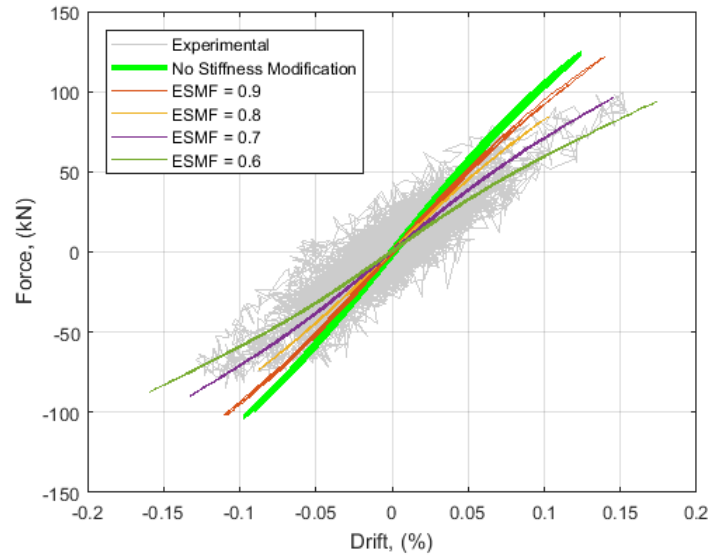


Figure 11-10 S4-CA-G1 at the 50% in 50 year intensity level

Figures 11-6 to 11-9 show these results for the S4-CA-G1 test at the 50% and 2% in 50 year intensity levels with *ESMFs* of 0.7 and 0.8, while Figure 11-10 compares the force-displacement hysteresis plots for each of the *ESMFs* employed. Figure 11-11 compares the median normalized *EDP* values for all the tests using each *ESMF*, as well as the standard unmodified stiffness models, to the *EDPs* measured in the experiments.

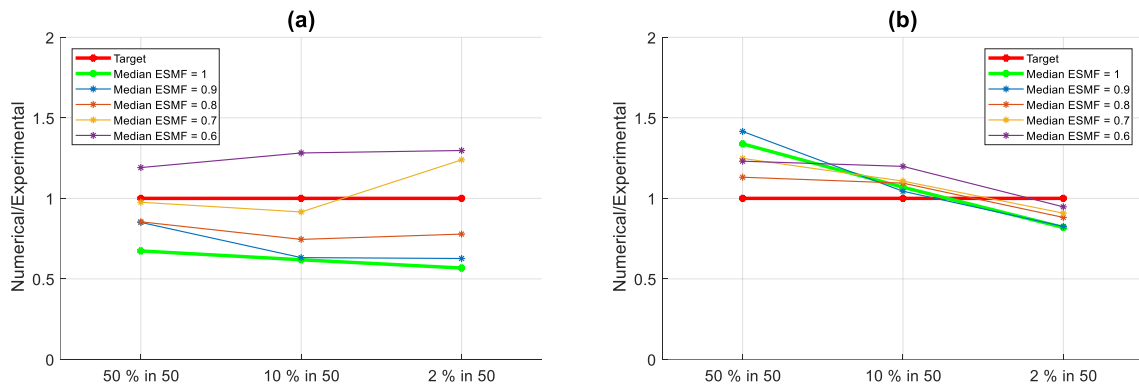


Figure 11-11 Impact of the *ESMF* on (a) median peak inter-storey drift and (b) median peak floor acceleration response

A clear, and expected, trend is evident when the impact of the *ESMF* on median predicted peak inter-storey drift is examined. As the *ESMF* is reduced, i.e. as the structural model becomes more flexible, the predicted drift increases. It is clear that compared to the regular model with no stiffness modification, improved drift prediction can be achieved by reducing the *ESMF* and model stiffness. The closest match to the experimental results is achieved using an *ESMF* of 0.7 for the 50% and 10% in 50 year intensity levels. For the 2% in 50 year intensity level an *ESMF* of

0.8 gives marginally better prediction of peak inter-storey drift, although the improvement compared to an *ESMF* of 0.7 is minimal (less than 1%).

In contrast, the trends for peak floor acceleration are less clearly defined. However, some general observations can be made. Firstly, at the 50% in 50 year intensity level, where response is largely elastic, the median peak floor acceleration is generally lower for the models with reduced stiffness, although the extent of this is not uniform. Using an *ESMF* of 0.8 gives the closest match to the experimental results. At the 10% in 50 year intensity level the best match is achieved using an *ESMF* of 0.9. Acceleration response is overestimated by the lower *ESMF* models. However these low stiffness models give the best prediction of acceleration at the 2% in 50 year intensity level. Overall, on average the *ESMF* of 0.8 gives the closest match to the median experimental peak acceleration response.

11.3 Impact of modelling inaccuracies on Performance Measures

From a performance assessment viewpoint, it is necessary to understand how these apparent modelling inaccuracies are carried through in the performance assessment procedure and how they affect the calculated performance measures like expected repair cost and downtime.

11.3.1 BRACED Test Results

In order to do this, performance assessment was carried out for a single-storey building with the BRACED test frame as the seismic resisting element. This was done using *EDPs* measured in the BRACED experiments and calculated in corresponding OpenSees NLTHA. As before, performance measures were calculated using these structural analysis results within PACT. The type and quantity of non-structural elements included in the building the performance model in PACT were obtained using the FEMA P-58 normative estimates tool.

A single-storey 3 bay by 3 bay building, with lateral resistance provided by the BRACED test structure, as illustrated in Figure 11-12, was used as a case study building for performance assessment. This larger building was used because the seismic mass of the experimental test structure, 44Mg, is larger than would typically be supported in plan by a structure of the size of test frame, whereas using the 3x3 bay structure results in a more realistic mass distribution per unit area (giving a seismic design load of 4.1kN/m²; which is what would be obtained if realistic design gravity actions of 3.2kN/m² permanent and 3kN/m² variable loads were applied). This is relevant because the floor area dictates the quantity of non-structural elements included in the building performance model used for performance assessment in PACT.

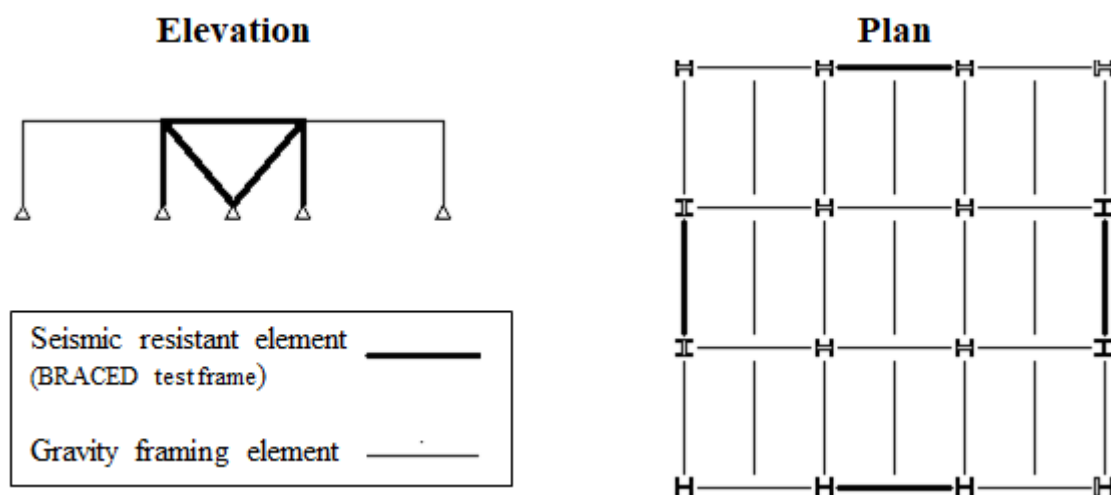


Figure 11-12 Elevation and Plan of case study building for comparison of performance assessments using experimental and numerical *EDPs*; BRACED test frame used as the seismic resistant component in all directions

Performance assessment was carried out using the *EDPs* obtained from the experimental tests and then from the corresponding numerical analyses. In one sense this is an unusual approach, as performance assessment is normally performed using *EDPs* obtained from applying multiple ground motion records to the same frame as opposed to applying the same ground motions to multiple frames with different brace-gusset configurations as was the case here. However, as the focus here is on the difference between performance metrics calculated using experimental and numerical *EDPs*, as opposed to the value of performance metrics themselves, it is simply necessary to obtain two corresponding sets of *EDPs*; one from experimental results and a corresponding one developed from numerical analysis.

The results of the performance assessment are presented in Table 11-3. It can be seen that the use of the numerically-calculated *EDPs* leads to expected repair costs and downtime being underestimated relative to the costs obtained using the experimental *EDPs*.

Table 11-4 Comparison between performance measures obtained using experimental and numerically calculated *EDPs*

Input <i>EDPs</i>	Modelling Uncertainty Factor: β_m	Annualized Repair Cost (\$)	Annualized Downtime (Days)
Experimental	0	2691.02	0.70
Numerical	0	2050.01	0.49
Numerical	0.5	2457.28	0.63

However, as discussed previously, a factor, β_m , can be included in the FEMA-P58 methodology to account for uncertainty in structural modelling. Put simply, this works by broadening the distribution used to develop the response parameters of each earthquake realization used to

assess performance. When the recommended β_m value of 0.5 is included in the assessment for the numerical *EDPs*, the performance metrics estimated using the numerical *EDPs* become closer to those obtained using the experimental *EDPs*, as illustrated in Table 11-3 and Figure 11-13.

The application of β_m widens the distribution of *EDPs* used in the earthquake realizations used to calculate losses. This ultimately results in greater losses, as greater component damage is induced in more of the earthquake realizations used in the performance assessment. Figure 11-13 shows the breakdown of these losses between drift and acceleration sensitive components. From this, it is observed that without accounting for modelling uncertainty, numerical drift dependant losses are less than half (42% of the experimental value) of those calculated using the experimental *EDPs*, while acceleration sensitive losses are slightly greater (120%). Overall losses are 76% of those obtained using experimental *EDPs*. For $\beta_m = 0.5$ drift dependant losses are still underestimated (62%) compared to those determined using the experimental results, with this somewhat negated by an overestimation of acceleration sensitive losses (130%). Ultimately, this means that total losses are 91% of those calculated using the experimental *EDPs* when the model uncertainty is incorporated using β_m .

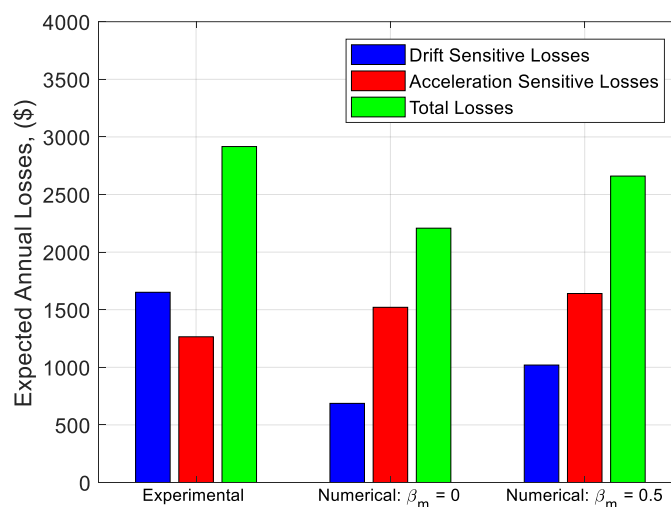


Figure 11-13 Comparison of the present value of expected losses, due to repair costs and downtime, for the case study building calculated using experimental *EDPs*, numerical *EDPs* and numerical *EDPs* with $\beta_m = 0.5$

From this it can be concluded that while some biases exist in the numerically-calculated *EDPs* compared to experimentally-obtained values, these are in line with the expected influence of model uncertainty in FEMA P-58. Nevertheless, the modelling discrepancies suggested by Figure 11-13 demand further investigation.

11.3.1.1 Reduced Stiffness Models

Performance assessment was also carried out using reduced stiffness models with *ESMFs* of 0.7 and 0.8. These analyses were performed twice, with and without the inclusion of modelling uncertainty, i.e. β_m was set to equal 0 and then 0.5. Table 11-5 shows the results of this loss assessment. From this it can be seen that when modelling uncertainty is ignored the *EDPs* from the *ESMF* of 0.7 models give results very similar to those obtained using the experimental *EDPs*. The *ESMF* of 0.8 results slightly underestimate annualized repair cost and downtime; the values obtained are quite similar to those from the regular numerical models using $\beta_m = 0.5$. When modelling uncertainty is considered, the *ESMF* = 0.7 results overestimate the value of performance metrics. However, the *ESMF* = 0.8 *EDPs* return very similar values to those calculated using the experimental *EDPs*.

Table 11-5 Comparison between performance measures obtained using experimental and numerically calculated *EDPs* using *ESMFs* of 0.7 and 0.8

Input <i>EDPs</i>	<i>ESMF</i>	Modelling Uncertainty Factor: β_m	Annualized Repair Cost (\$)	Annualized Downtime (Days)
Experimental	N/A	0	2691.02	0.70
Numerical	0.7	0	2722.35	0.70
Numerical	0.8	0	2365.57	0.59
Numerical	0.7	0.5	3029.12	0.80
Numerical	0.8	0.5	2674.69	0.69

Figures 11-14 and 11-15 present the breakdown of the expected losses into contributions from drift and acceleration dependant components. As before, the use of *EDPs* from the numerical models under predict drift dependant losses compared to the use of experimental *EDPs*. However, in terms of overall losses, this is balanced out by an over-estimation of acceleration dependant losses.

For all of this work it should be borne in mind that the ground acceleration values are identical in the experimental and numerical models. This means that the acceleration dependant losses at this storey level are very similar, and given that the case study building is a single storey building, the overall acceleration dependant losses also tend to be similar. The only differences in acceleration dependant losses are due to differences in the damage experienced a small number of acceleration sensitive components at the roof level, e.g. roof tiles. It is likely that differences in floor acceleration would have more effect on loss assessment results for buildings with a greater number of stories.

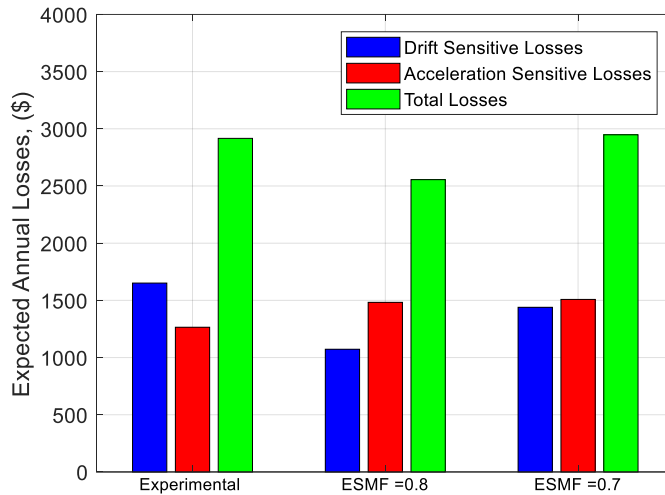


Figure 11-14 Comparison of the present value of expected losses, due to repair costs and downtime, for the case study building calculated using the experimental *EDPs* and numerical *EDPs* from *ESMF* = 0.7 and 0.8 models. Performance measures calculated with $\beta_m = 0$

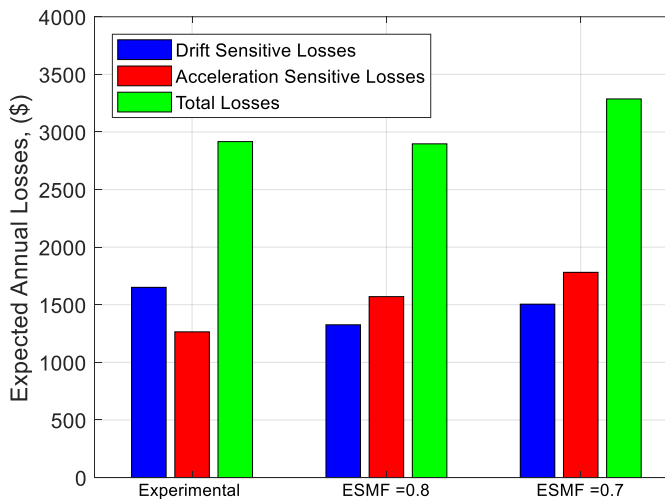


Figure 11-15 Comparison of the present value of expected losses, due to repair costs and downtime, for the case study building calculated using the experimental *EDPs* and numerical *EDPs* from *ESMF* = 0.7 and 0.8 models. Performance measures calculated with $\beta_m = 0.5$

11.4 Performance Assessment Using Updated Modelling Assumptions

The above results prompted further investigation of the impact of the *ESMF* on performance assessment. This was done by repeating the performance assessment of the 5 storey CBF designed using $q = 4$ at the Oakland site, as described in Chapter 6, using structural models with reduced stiffness. Based on the analysis of the BRACED test results it was decided to use an *ESMF* of 0.75. Using an *ESMF* value of 0.75 reduces the model stiffness, with the fundamental period of the structure increasing from 0.38s to 0.44s.

The impact of this on the main *EDPs* for performance assessment, namely peak inter-storey drift and peak floor acceleration, is shown in Figure 11-16. It can be seen that peak drift response is greater for the more flexible model with $ESMF = 0.75$. Peak acceleration response is generally greater for the stiffer model with no stiffness modification. However, at the two lowest intensity levels examined acceleration response is similar for the two models, something which is partly attributable to the reduced seismic hazard at the longer fundamental period. The difference between the peak drift responses of the two models is generally greater than the difference between the peak acceleration responses, something which is evident from the storey-level drift *EDP* plots for the MCE, DLE and SLE presented in Figure 11-17. This trend has been noted before; changing frame stiffness slightly for low or mid-rise CBFs appears to have little impact on peak acceleration response, as these structures tend to lie in the constant acceleration region of the spectrum.

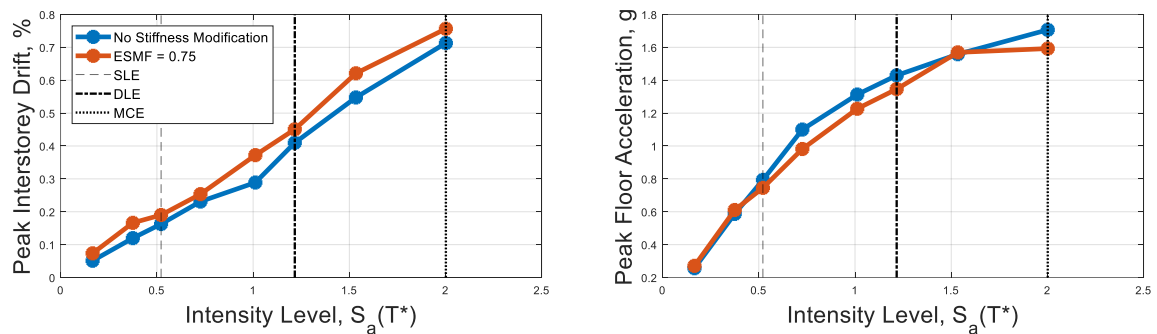


Figure 11-16 Comparison between median peak *EDPs* for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using $ESMF = 0.75$

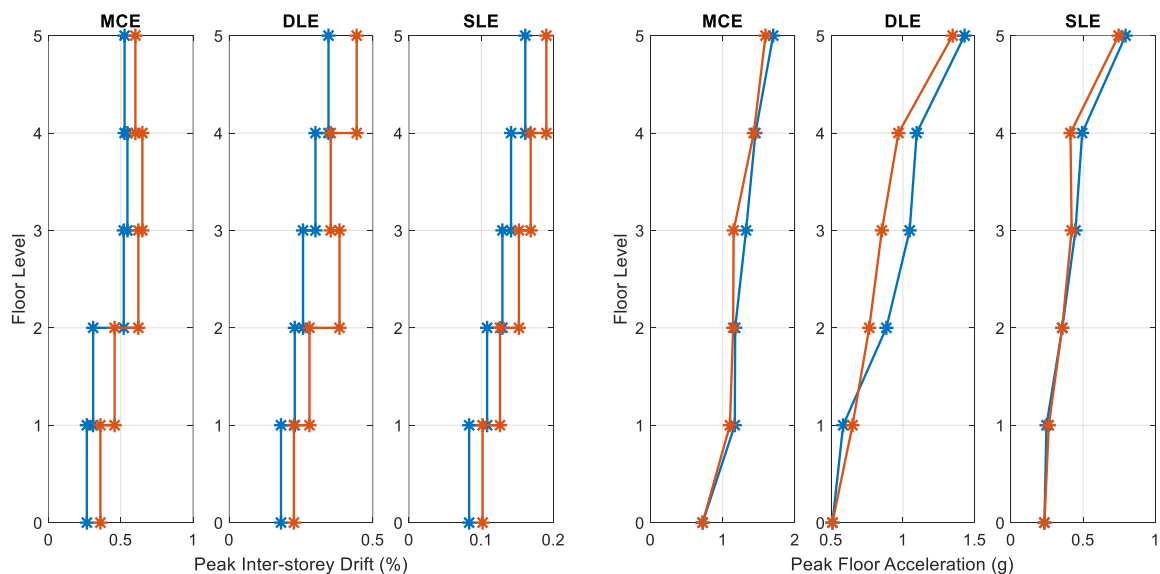


Figure 11-17 Comparison between median *EDPs* for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using $ESMF = 0.75$ at the MCE, DLE and SLE

Figure 11-18 presents the impacts of the *ESMF* on the expected lifetime losses. Overall losses are greater by approximately 16% (17.6% versus 20.5% of initial cost) for the reduced stiffness model. It can be seen that this is primarily due to greater drift dependant losses for the model with modified stiffness, which is reasonable given the greater drifts displayed in Figure 11-12. Losses due to damage to acceleration sensitive components are very similar for the two model types. While peak acceleration is generally greater for the stiffer, unmodified models, it can be seen from Figure 11-16 that at the lowest intensity levels the acceleration response is slightly greater for the *ESMF* = 0.75 model. As these intensity levels have the highest weighting in the lifetime loss calculation, the greater accelerations, and resultant losses, experienced by the stiffer model at the other intensity levels are negated. Collapse losses are similar for the two models, while losses due to excessive residual drifts are slightly greater for the more flexible *ESMF* = 0.75 model, but as was noted previously, the contribution of these to overall lifetime losses for CBFs is minimal.

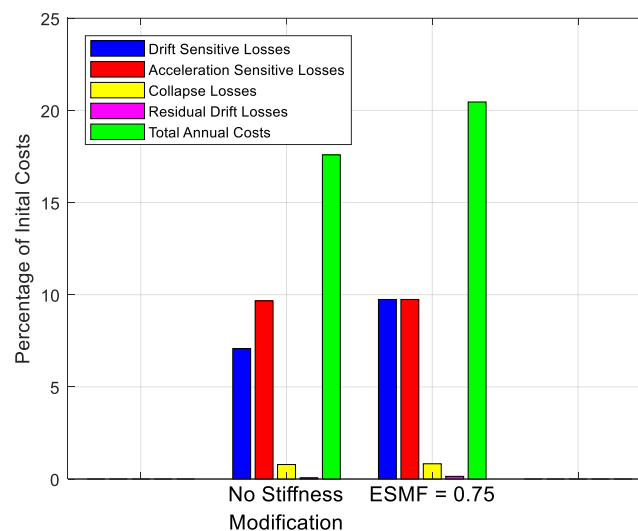


Figure 11-18 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using *ESMF* = 0.75

Figure 11-19 shows the losses for the two alternate models at the MCE, DLE and SLE earthquake intensity levels. There is little difference in the overall losses for the two models at each of these intensity levels. However, in all three cases the drift sensitive losses are greater and acceleration sensitive losses are smaller for the frame with reduced stiffness. Demolition losses due to excessive residual drift become more prominent at the MCE and DLE levels for the modified stiffness model.

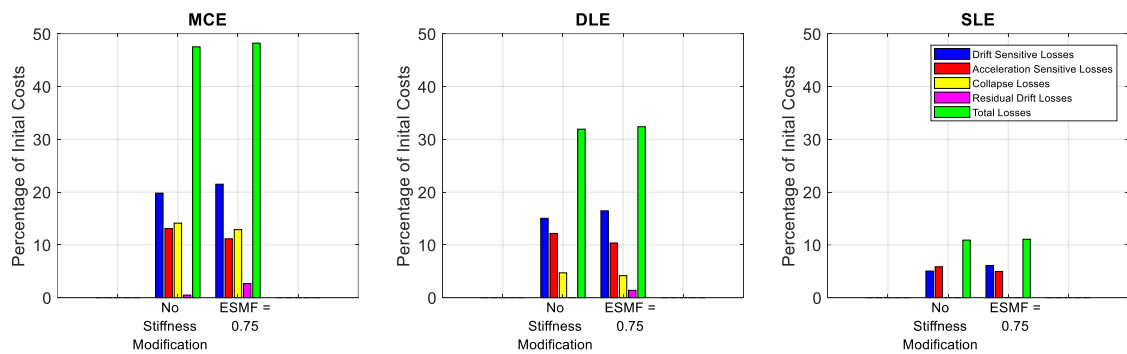


Figure 11-19 Comparison between intensity specific losses for the regular and reduced stiffness models for the 5 storey frames designed using $q = 4$ at the Oakland site

Loss assessment was also carried out using reduced stiffness models for the 5 storey CBFs designed using $q = 2$ at the Oakland site and using $q = 4$ at the Seattle site, as well as for the 10 storey $q = 4$ design at the Oakland site. This allows the impact of the reduced model stiffness on all of the key performance trends noted in Chapter 6 to be examined, namely the variation in losses with behaviour factor, seismic hazard and building height.

Similar trends are observed for the 5 storey frames designed using $q = 2$, as shown in Figure 11-20. Again, drift dependant losses are greater for the reduced stiffness model, while acceleration sensitive losses are almost identical, leading to approximately 17% (13.0% versus 15.1% of initial cost) greater overall losses when the reduced stiffness model is employed.

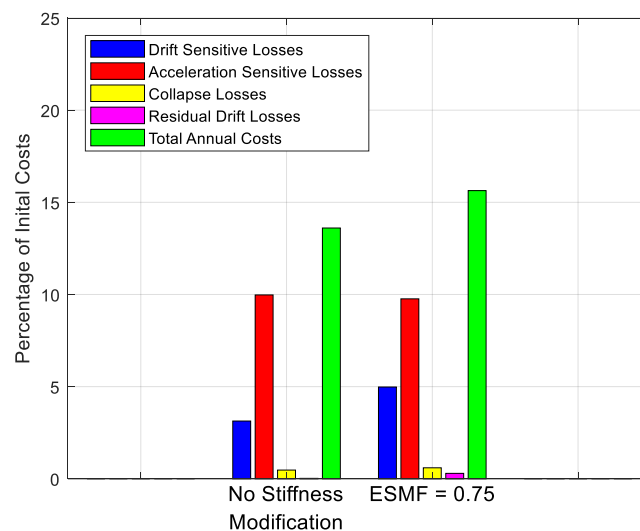


Figure 11-20 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 2$ at the Oakland site for the regular structural model and using $ESMF = 0.75$

For the $q = 4$ frame at the Seattle site there is a small reduction in acceleration dependant losses for the reduced stiffness frame, as shown in Figure 11-21. This negates some of the increase in drift dependant losses, which is similar to that seen for the other designs examined, meaning

that the increase in overall losses for the reduced stiffness model is just 7% (8.21% versus 8.76%).

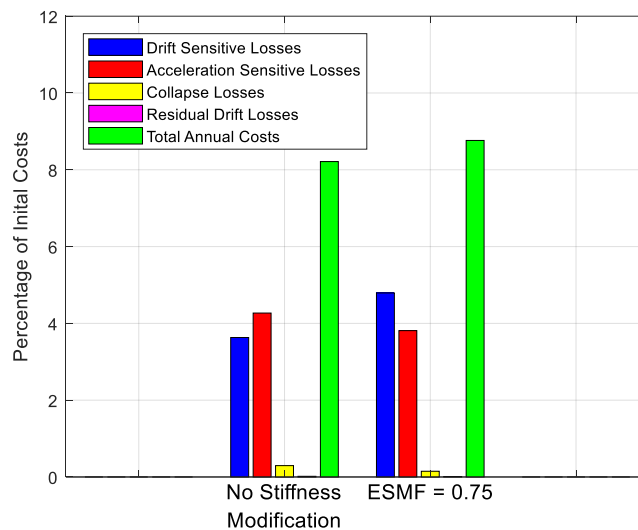


Figure 11-21 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 4$ at the Seattle site for the regular structural model and using $ESMF = 0.75$

As shown in Figure 11-22, overall losses are 12% (37.25% versus 41.57% of initial cost) greater for the reduced stiffness model for the 10 storey CBF designed using $q=4$ for the Oakland site. As before, this is primarily due to an increase in drift dependant losses, although a small reduction in acceleration dependant losses is noticeable for the reduced stiffness model.

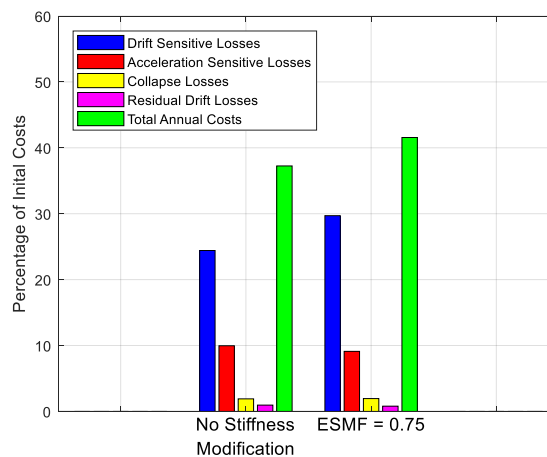


Figure 11-22 Comparison between expected lifetime losses for 5 storey CBF designed with $q = 4$ at the Oakland site for the regular structural model and using $ESMF = 0.75$

In general, similar trends are observed for each frame examined; reducing the model stiffness results in greater inter-storey drift, and therefore greater drift dependant losses. In some cases a small decrease in acceleration dependant losses is noted, but the influence of the model stiffness on acceleration is not as pronounced as it is for drift. This conclusion is similar to that

reached when examining the influence of the behaviour factor on CBF lifetime performance; in fact the influence of using an *ESMF* of 0.75 appears to be quite similar to increasing the behaviour factor by 1.

The analysis of the experimental data from both the hybrid simulation tests conducted in this study and the BRACED tests, suggests that employing an *ESMF* of 0.75 gives an improved estimate of *EDPs*, and therefore gives a more accurate prediction of true performance metrics. If this is the case, it suggests that expected losses are slightly underestimated when regular models are employed due to an underestimation of inter-storey drift. However, the extent of this apparent underestimation is not particularly large, on average approximately 13%, or in absolute terms about 2.5% of initial costs, for the cases considered here. Furthermore, the use of an *ESMF* of 0.75 doesn't greatly impact on the key trends identified in for CBF lifetime losses in Chapter 6. Losses are shown to increase with the behaviour factor, seismic hazard and frame height by similar amounts irrespective of whether the unmodified or modified stiffness models are used. More generally, this would suggest that the calculation of lifetime performance measures is not particularly sensitive to the structural model employed. This agrees with the conclusions of Hwang and Lignos (2017a) about the minimal influence of the inclusion of gravity framing elements in the structural model on lifetime repair costs.

11.5 Assessment of Rapid Performance Assessment Methods Using Experimental Results

In Chapter 7, a series of expressions were developed to predict the drift and acceleration response of CBF structures designed to Eurocode 8 for use in performance assessment. The results of a large number of time history analyses were used to develop regression equations for the global drift modification factor, δ_{mod} and the maximum drift modification factor, ϑ_{mod} :

$$\ln(\delta_{mod}) = -0.195 - 0.018q_{eff} + (0.357 - 0.239q_{eff}) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] \quad 11.1$$

$$\begin{aligned} \ln(\theta_{mod}) = & 2.868 + (0.538 - 0.317q_{eff}) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] \\ & + 0.303 \ln \left[\max \left(1.752 \frac{T_1}{T_m}, 1 \right) \right] - 4.649\gamma \\ & + 0.881\beta_3 - 19.557\beta_k \end{aligned} \quad 11.2$$

Where T_1/T_m is the ratio of fundamental structural period to ground motion mean period, γ is the first mode participation factor, β_3 is relative storey stiffness of the upper to lower half of the frame as defined by Kumar et al. (2013) and β_k is the column to brace stiffness ratio, q_{eff} is the degree of nonlinearity experienced, which in practice essentially represents the ground motion

intensity. The expressions for δ_{mod} or ϑ_{mod} can be used in conjunction with the first mode shape to predict inter-storey drift over the height of a building, with ϑ_{mod} tending to give more conservative values.

An expression to predict the floor acceleration magnification at each floor level was also developed:

$$\ln\left(\frac{PFA}{PGA}\right) = \frac{h_i}{H} \left(1.846 - 0.196q_{eff} - 0.488\frac{h_i}{H} - 0.379\frac{T_1}{T_m} + 0.041NS \right) \quad 11.3$$

where h_i is the height of the i^{th} storey level, H is the overall building height and NS is the number of stories.

Expressions were also developed to estimate the lognormal standard deviation of δ_{mod} , ϑ_{mod} and PFA/PGA :

$$\sigma_{\ln, \delta_{mod}} = 0.35 \quad 11.4$$

$$\sigma_{\ln, \vartheta_{mod}} = 0.18 + 0.06NS \quad 11.5$$

$$\sigma_{\ln, \frac{PFA}{PGA}} = 0.3 \quad 11.6$$

In order to examine the effectiveness of these models, the values of peak drift and acceleration they predict are compared to those recorded in the hybrid simulations discussed in Chapter 9.

The effective behaviour factor, q_{eff} , essentially represents the earthquake intensity level and can be calculated using a structure's lateral strength obtained from a pushover analysis as per Equation 11.7:

$$q_{eff} = \frac{S_a(T_1) \times \gamma \times m}{V_{bs,yield}} \quad 11.7$$

In order to calculate q_{eff} for each hybrid simulation intensity level, $V_{bs,yield}$ was obtained from a numerical pushover analysis performed in OpenSees while γ was also calculated numerically. Spectral acceleration of each ground motion at $T = 0.25s$ was taken as $S_a(T_1)_{NLTHA}$. The values are given in Table 11-6.

Table 11-6 Effective behaviour factor (q_{eff}) used to calculate δ_{mod} and ϑ_{mod} for each intensity level

Intensity Level	$S_a(T_1)$	$V_{BS,yield}$	γ	Mass (Mg)	q_{eff}
50% in 50	0.4775	125.592	0.891	31.5	1.046
10% in 50	1.1882	125.592	0.891	31.5	2.604
2% in 50	2.1104	125.592	0.891	31.5	4.624

11.5.1 Global Drift – δ_{mod}

Figure 11-23 shows the recorded values for δ_{mod} recorded in each of the experiments performed as part of the hybrid simulation testing program. Through Equation 7.10, δ_{mod} was calculated using the experimentally recorded roof drift, q_{eff} from Equation 7.16 and the yield drift obtained from a numerical pushover analysis. The regression model for δ_{mod} developed in Chapter 7, and the associated standard deviations, are also shown. It can be seen that the model predicts roof drift reasonably successfully with all of the δ_{mod} values for the simulations within ± 1 lognormal standard deviation. However, the fall in the predicted δ_{mod} as T_1/T_m decreases below 1 suggests it is possible that global drift for the 50% in 50 year ground motion intensity ($q_{eff} = 1.046$) is underestimated by the model. This is most probably a result of the nature of the regression equation employed. The equation is designed to capture the increase in δ_{mod} observed in this T_1/T_m range for higher q_{eff} values. However, this trend is extended to produce a slight decrease in predicted δ_{mod} for lower behaviour factors, which may not reflect true behaviour. Despite this, the model gives a slightly better prediction than the equal displacements rule proposed by Eurocode 8, which appears to overestimate peak roof drift.

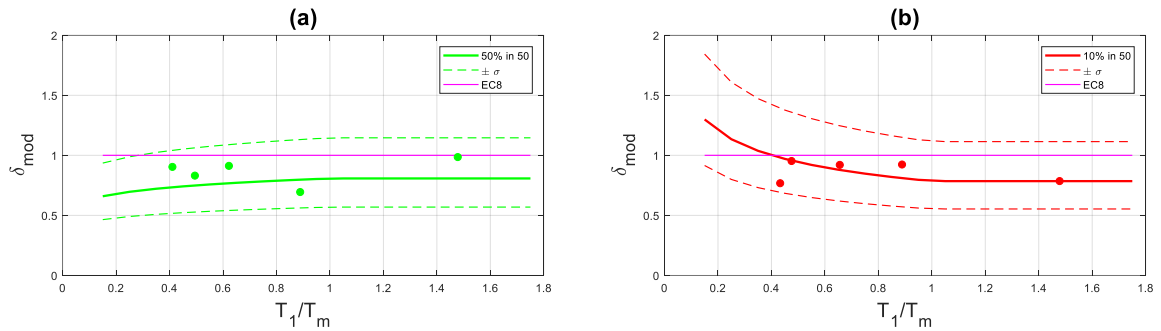


Figure 11-23 Predicted and calculated δ_{mod} values for the Hybrid Simulations

11.5.2 Inter-Storey Drift – θ_{mod}

Figure 11-24 compares the experimentally obtained values of θ_{mod} with the corresponding model developed in Chapter 7. It can be seen that the values of θ_{mod} do not compare well with the values predicted by the model, which consistently underestimates peak inter-storey drift.

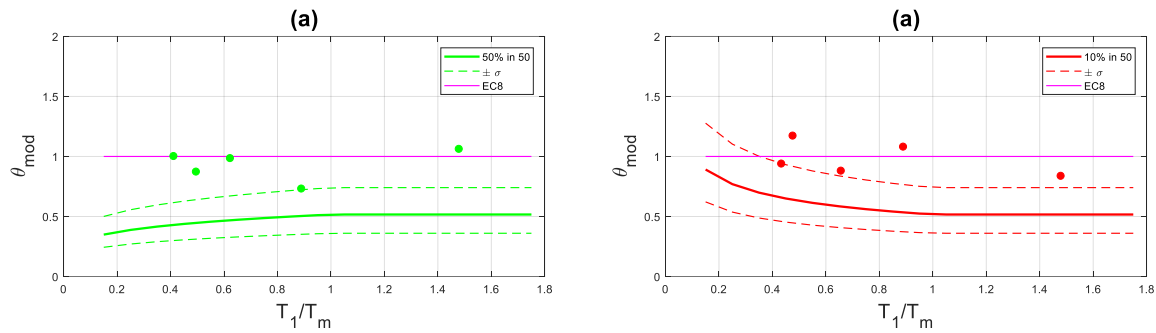


Figure 11-24 Predicted and calculated θ_{mod} values for the Hybrid Simulations

This poor match is attributable to the influence of the β_3 variable, which is outside the range of values for which the regression model is developed. β_3 is calculated as the ratio of the peak inter-storey drift in the upper to lower half of the frame, where inter-storey drift is calculated from the fundamental mode shape, as per Equation 7.3. This provides a way of quantifying higher mode effects due to weaker upper stories; β_3 is high if a flexible upper storey is noted in the mode shape. In the dataset used to develop the equations, which comes from a series of 24 frames designed to Eurocode 8, the β_3 values tend to be greater than 1, with an average value of 1.168. However, for the test frame, the value of β_3 is 0.71. This is because the first storey, i.e. the experimental substructure, was deliberately designed to have much lower brace overstrength than the other stories, resulting in a very flexible storey. A flexible storey in the lower half of the frame reduces the value of β_3 . Due to the data used in its development, the regression model for ϑ_{mod} is not good at handling low β_3 values; as the bottom storeys become more relatively flexible the model actually predicts decreased drift response, which intuitively is incorrect. This leads to the under-prediction of ϑ_{mod} observed in Figure 11-24.

11.5.2.1 Improved Predictive Model

In the MRFs designed to Eurocode 8 examined by Kumar et al. (2013), β_3 is generally less than one; ranging from 0.73 to 1.03. This is because the case study buildings used in that study have a taller, and therefore more flexible, first storey, meaning the most flexible storey is generally in the lower half of the frame and therefore β_3 is generally less than unity. The regression analysis performed in that study showed that ϑ_{mod} decreased as β_3 increases; or viewed slightly differently, ϑ_{mod} decreased as β_3 becomes closer to 1. This is illustrated in Figure 11-26(a). It can be seen that broadly speaking when the range of values included in the two studies is examined it is clear that ϑ_{mod} increases as β_3 moves away from 1. This trend makes sense as a β_3 value of 1 represents a design with stiffness distributed evenly between the top and bottom half of the frames. Therefore, as one half of the frame becomes relatively less stiff, meaning β_3 moves away from 1, the tendency for greater inter-storey drift to occur increases. However, as can be seen in

Figure 11-25 (a) both studies appear to misrepresent the influence of β_3 when the model is extrapolated beyond the range of data used in their development.

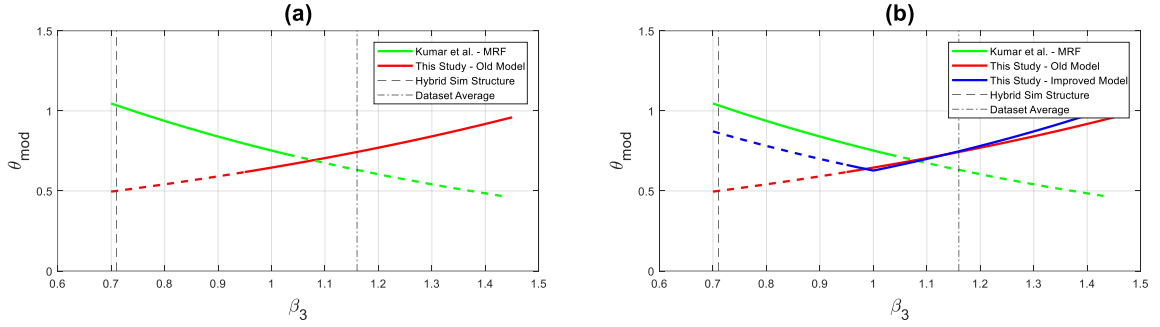


Figure 11-25 (a) Illustration of the impact of β_3 on θ_{mod} . Range of β_3 values used in the particular study are marked by solid lines, extrapolated values are dashed lines (Plot developed for $T_1/T_m=0.7$, $q_{eff}=3$, $\gamma=0.9$, $\beta_k = 0.0108$, but the basic trends shown hold for all values). (b) Illustration of the impact of β_3 on θ_{mod} , including the improved model which uses $|1-\beta_3|$ as a variable instead of β_3 and appears to capture the trends noted in the two studies better.

In order to address this issue the regression analysis was rerun with a reformatted equation. Thus, the β_3 variable was replaced with $|1 - \beta_3|$, i.e. how ‘far away’ β_3 is from 1. This allows the model to account for flexible stories in the lower half as well as the upper half of the frame. Equation 11.8 shows the result of this new regression analysis.

$$\begin{aligned} \ln(\theta_{mod}) = & 3.686 + (0.543 - 0.319q_{eff}) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] \\ & + 0.305 \ln \left[\max \left(1.747 \frac{T_1}{T_m}, 1 \right) \right] - 4.603\gamma \\ & + 1.098|1 - \beta_3| - 20.194\beta_k \end{aligned} \quad 11.8$$

The variation in θ_{mod} with β_3 for the two models, and the MRF model developed by Kumar et al. (2013), is shown in Figure 11-25(b). From this it can be appreciated how the new model appears to capture the general trends observed over a broader range of β_3 values.

Figure 11-26 shows that the new model is practically identical to the old model for the average β_3 value of the dataset used to develop the models, i.e. for a β_3 value of 1.16. However, as demonstrated in Figure 11-27, the difference between the models is clear when a β_3 value of 0.71, the value of the hybrid test structure, is used.

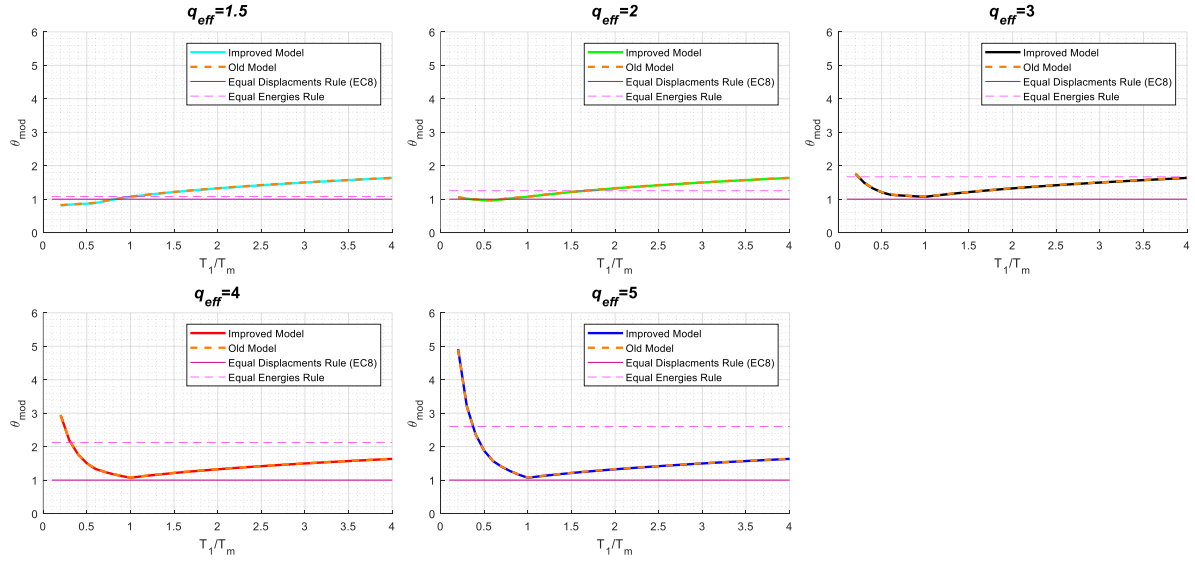


Figure 11-26 Comparison between models developed for ϑ_{mod} (using $\beta_3=1.16$; the average value for the case study frames in the development of the models in Chapter 7)

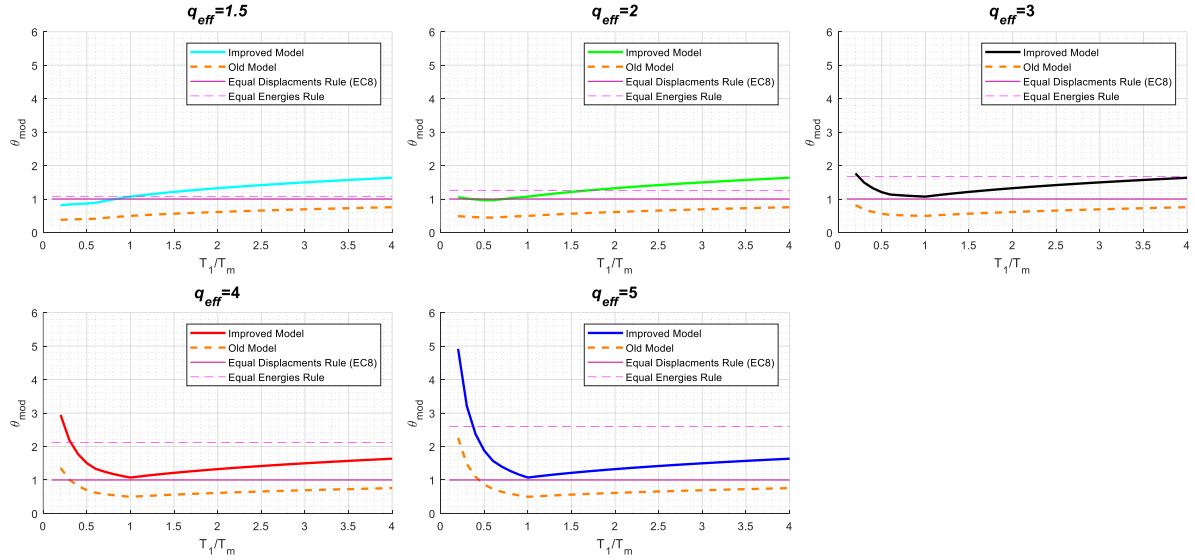


Figure 11-27 Comparison between models developed for (using $\beta_3=0.71$, the value for the hybrid simulation test structure)

Figure 11-28 compares the experimental data to this ‘improved’ model. When the new predictive equation is compared to the results of the hybrid simulations, it can be seen that the match is relatively good; suggesting that the new model offers improved extrapolation abilities for lower β_3 values. All the experimentally recorded values lie within ± 1 lognormal standard deviation, predicted by Equation 7.24, of the mean value. As with the model for δ_{mod} , it is possible that the nature of the regression equation employed means that ϑ_{mod} is slightly underestimated for T_1/T_m values less than 1 for the 50% in 50 year intensity level. Indeed, for this intensity level the equal displacements rule appears to give an improved prediction of drift response.

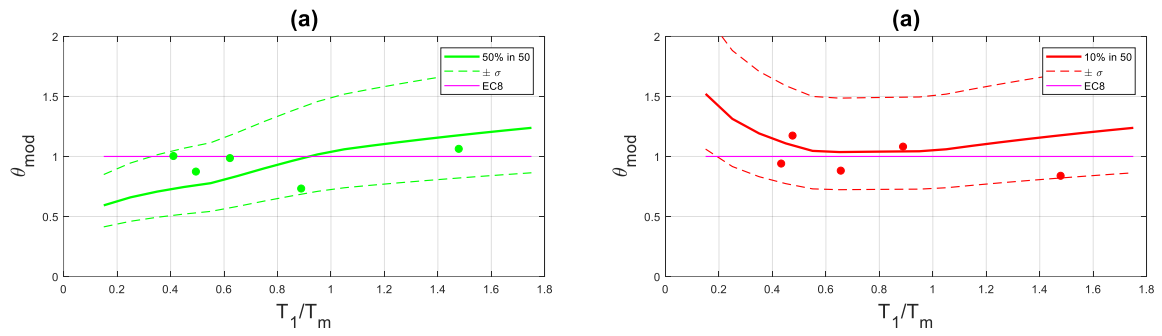


Figure 11-28 Predicted and calculated θ_{mod} values for the Hybrid Simulations, with updated model for θ_{mod}

11.5.2.2 Inter-Storey Drift Profile

Figures 11-29 and 11-30 show the recorded and predicted storey-level drift profile over the height of the test structure for each hybrid simulation. The inter-storey drifts at each storey level are predicted using both δ_{mod} and ϑ_{mod} . Each of these is combined with the first mode shape to develop the inter-storey drift profile, as per Equations 7.20 or 7.21. For the 50% in 50 year simulations, shown in Figure 11-29, the predictions are relatively successful for both methods. However, the maximum inter-storey drift at the ground storey, i.e. the experimental test level is generally slightly under predicted.

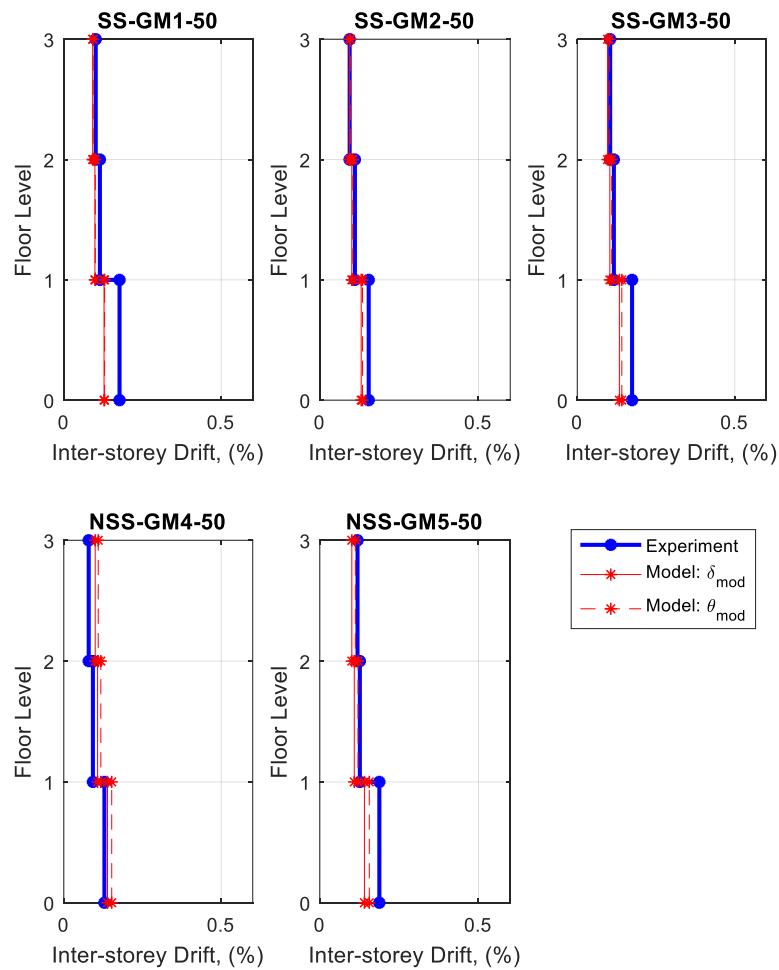


Figure 11-29 Experimentally recorded and modelled inter-storey drift profiles for the 50% in 50 year simulations

There is a more noticeable difference between the modelled and recorded responses for the 10% in 50 year simulations. Both prediction methods generally give a reasonable ‘average’ value for drift response, with the ϑ_{mod} based method being slightly more conservative. There is greater variation in inter-storey drift over the frame height for the experimental results. Inter-storey drift for the top storey is over-predicted in each case, while inter-storey drift at one of the other two levels is generally under-predicted. It is possible that improved predictions could be achieved by accounting for the change in mode shape that occurs with yield.

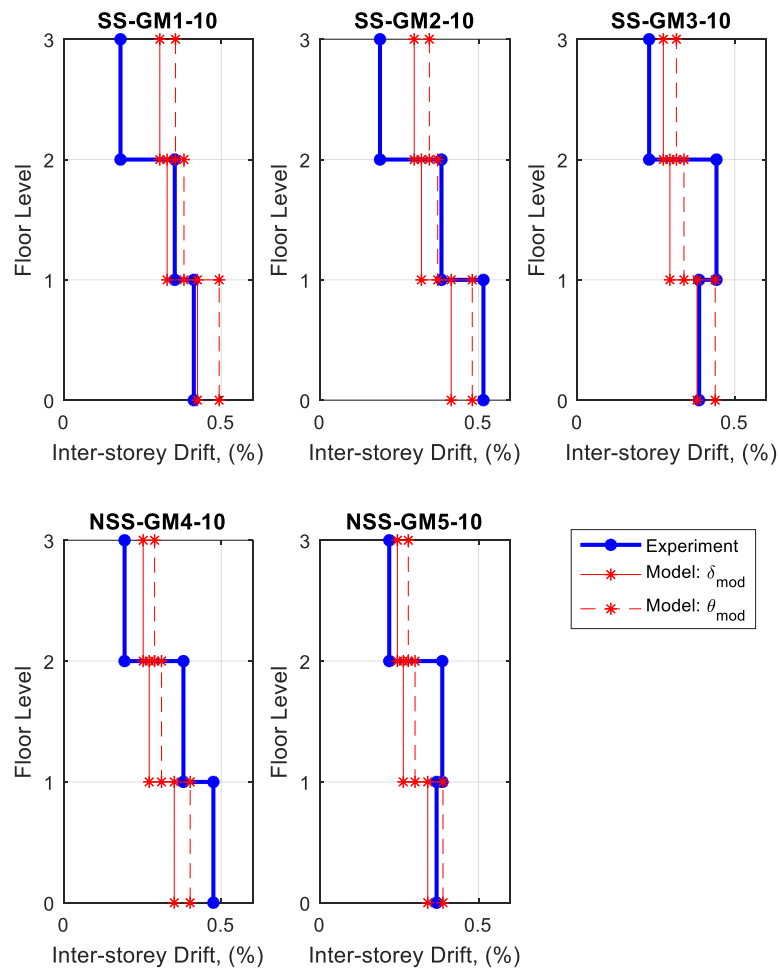


Figure 11-30 Experimentally recorded and modelled inter-storey drift profiles for the 10% in 50 year simulations

11.5.3 Peak Floor Acceleration

Figures 11-31 and 11-32 compare the experimental and modelled floor acceleration magnification. For the 50% in 50 year intensity level, the numerical model gives a very good prediction of the experimental response for 4 out of the 5 analyses. However, for the NSS-GM-5-50 run the acceleration response is severely under predicted.

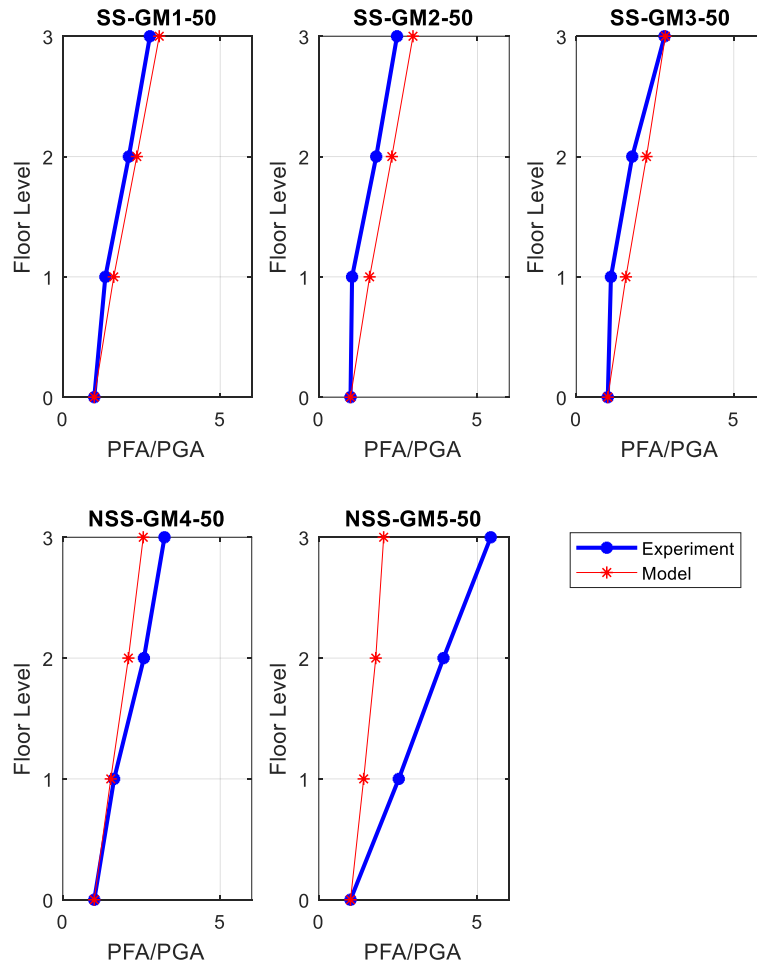


Figure 11-31 Experimentally recorded and modelled floor acceleration magnification profiles for the 50% in 50 year simulations

Likewise, for the 10% in 50 year intensity level, the model gives a good prediction of the response in the majority of cases, as shown in Figure 11-32. However, again, the model severely under-predicts response for the NSS-GM5-10 experiment. It is possible that this is simply attributable to the record-to-record variation associated with the model, but given that this is a scaled version of the ground motion for which with the under-predicted response at the 50% in 50 year intensity level was observed, it is suggestive that some characteristic of this ground motion leads to increased acceleration response and is not picked up by the model. This is analysis run with one of the higher T_1/T_m values; hence, it is possible that model accuracy decreases as T_1/T_m increases. However, it is likely that if such a trend was particularly strong it would have been noted in the examination of model residuals carried out in Chapter 7.

The model predicts an approximately linear variation in PFA/PGA over the frame height. However, the experimental data shows a tendency for accelerations at the roof level to be quite

similar to those at the second floor level. For taller buildings, other studies (for example (Reinoso and Miranda, 2005)), and to an extent the work in Chapter 7, have shown a tendency for increased accelerations at the roof level. In short, it appears that floor acceleration magnification increases at a relevantly constant rate with each storey level, except at the roof level where more variation observed. It is difficult for models which attempt to predict accelerations at each floor level as a function of relative height, such as the one developed here or in FEMA P-58, to capture these variations. This suggests that improved prediction could be achieved by developing separate models for roof acceleration.

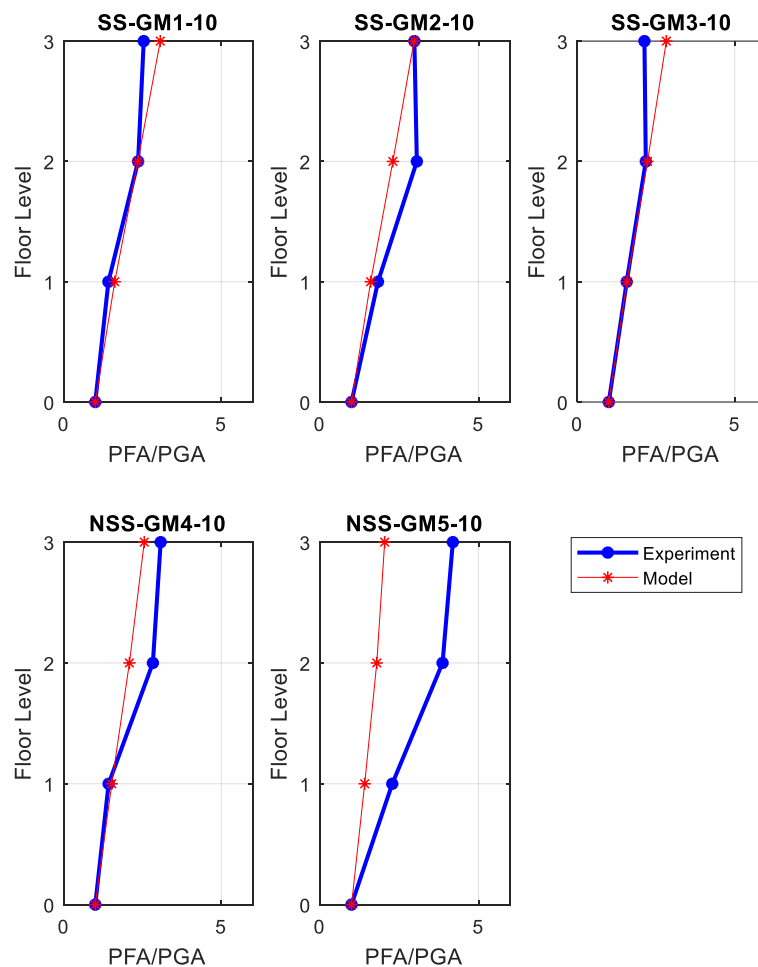


Figure 11-32 Experimentally recorded and modelled floor acceleration magnification profiles for the 10% in 50 year simulations

11.6 Conclusions

The Chapter used experimental results to assess conclusions reached elsewhere in this research. The primary idea to emerge from the hybrid simulation testing program was that the structural models typically employed for CBFs tend to overestimate the structural stiffness. This conclusion

was further assessed using the results of the BRACED shake table tests. It was found that models with reduced stiffness, applied through reducing the elastic modulus, gave more accurate *EDP* predictions than regular models with unmodified stiffness. The *EDPs* obtained from these tests were also used to assess the modelling uncertainty factor, β_m , used in FEMA P-58 performance assessment. The impact of model inaccuracies on expected losses was found to be in line with the expected influence of model uncertainty incorporated in the FEMA P-58 methodology, meaning that performance measures appear to be estimated reasonably well in spite of these modelling inaccuracies.

Some performance assessments previously carried out in Chapter 6 were repeated using reduced stiffness models. The expected losses calculated were generally slightly greater for than those obtained using regular models, primarily due to an increase in drift dependant losses. However, the general conclusions reached about CBF lifetime losses, namely the increase with design behaviour factor, frame height and seismic hazard, were found to remain valid.

Finally, the *EDP* prediction models developed in Chapter 7 for rapid performance assessment were assessed using the experimental results. The models for roof drift, represented by δ_{mod} , and floor acceleration magnification were found to predict experimentally recorded response reasonably successfully. However, the model for peak inter-storey drift, represented by ϑ_{mod} , was shown to be inadequate for relative storey stiffness, β_3 , values outside the range used in the development of the model. An improvement was suggested through using $|1-\beta_3|$, as opposed to β_3 , as a parameter in regression.

In short, the work carried out here suggests that while there is scope to improve the accuracy of numerical models of CBFs, the lifetime performance assessment procedures employed in FEMA P-58, and resulting conclusions, are relatively robust and are not particularly sensitive to modelling error. Further to this, it appears possible to predict CBF response reasonably successfully without the need for NLTHA.

12 Conclusions

12.1 Completed Work

The overall aim of this work was to examine how lifetime performance assessment methodologies can be applied to improve earthquake resistant CBF design and to assess how such methodologies can be improved for CBFs. The first stage of this work involved examining the impact of the design behaviour factor, q , on expected lifetime losses of CBFs designed to Eurocode 8. Two and five storey CBFs were designed to Eurocode 8 using behaviour factors of 1, 2, 3, 4 and 5, while ten storey CBFs were designed using behaviour factors of 2, 3 and 4. Each of these CBFs were designed for two case study sites in the USA; one in Oakland in California with a very severe seismic hazard and a second in Seattle, Washington with a more moderate hazard in line with conditions in Europe. The structures were modelled in OpenSees using commonly adopted modelling practices recommended elsewhere in literature. Nonlinear time history analysis (NLTHA) was carried out for a variety of potential earthquake scenarios using ground motion records selected to match the conditional spectrum at the sites for eight different ground motion intensity levels. The primary outputs of interest, or Engineering Demand Parameters (*EDPs*), from these analyses were peak inter-storey drift and peak floor acceleration. Collapse fragility functions and peak residual drift were also calculated. Using these analysis outputs, the FEMA P-58 performance assessment methodology was employed to estimate expected lifetime losses, calculated as losses due to a combination of expected repair costs and estimated financial losses due to downtime. Overall losses were broken down into contributions from damage to drift and acceleration sensitive components, as well losses due to collapse and demolition due to excessive residual drift. The impact of various parameters on lifetime losses was assessed and the return on investment achieved for various design scenarios was also calculated.

A methodology to carry out performance assessment for CBFs designed to Eurocode 8 without the need to for NLTHA, termed ‘rapid’ performance assessment, was subsequently proposed. This was done by developing regression equations to predict peak drift and acceleration response based on frame lateral strength and stiffness, obtained from a pushover analysis. Thus, the technically challenging and time consuming NLTHA step in performance assessment is avoided, allowing for more practicable application of performance assessment techniques. The regression equations were developed from a large dataset of *EDPs* created by carrying out NLTHA for a set of twenty-four CBFs designed to Eurocode 8 for different seismic hazards and ground conditions. Values describing the expected *EDP* lognormal standard deviation were also

calculated. The values of peak drift and acceleration predicated by these equations were compared to values predicted by models proposed in other studies and design codes. The *EDPs* predicted by the developed regression equations were then used to carry out lifetime performance assessment for some of the CBFs which had previously been assessed using NLTHA-based methods. The *EDPs*, and subsequent performance metrics, were compared to those calculated using NLTHA.

The next goal in the research work was to undertake a substructured hybrid simulation programme, including a model updating component, to further examine the seismic response of CBFs. The first step in this process involved developing and correctly implementing the testing method. A 3 storey CBF with a single storey experimental substructure at the bottom level was designed to be suitable for substructured hybrid simulation given the constraints imposed by the facilities at TCD while also conforming to Eurocode 8. The hybrid simulations were performed using OpenSees as the finite element software and OpenFresco as a middleware allowing communication with the experimental setup. A generic experimental element was developed in OpenFresco to allow a multi-element, multi degree of freedom experimental substructure to be tested using a single actuator. Preliminary tests were performed to verify that the test setup worked correctly. Based on these, changes were made to the predictor-corrector algorithm that controls the actuator movement at each timestep to prevent the experimental control system causing tests to stop prematurely.

Once the system was deemed to be working adequately, a hybrid simulation testing programme, without model updating, was undertaken. Ground motion records were selected to match the conditional spectrum at a case study site at intensity levels with 50% and 10% probability of exceedance in 50 years. Hybrid simulation was performed for the 50% and 10% in 50 year intensity levels. The tests were successfully executed. The results of these simulations were compared to purely numerical simulations performed in OpenSees. Initially, it was intended to carry out further simulations at the 2% in 50 year probability of exceedance intensity level. However, prior to these tests being undertaken, a component in the test system failed and, at the time of writing, has not been replaced. This prevented any more tests being carried out.

An algorithm to implement online model updating during substructured hybrid simulation was developed. This involved using a set of numerical counterparts to optimize various modelling parameters based on data measured during the test. The model updating algorithm was implemented entirely within the OpenSees-OpenFresco framework. Numerical verifications of the updating algorithm were carried out for a series of increasingly complex scenarios, ranging

from the updating of one parameter in a linear response to the updating of multiple parameters for nonlinear response. Initially, the intention was to apply the algorithm during hybrid simulation experiments. However, the failure of the test equipment meant this was not possible. Instead, the algorithm was applied to the results of the hybrid simulations carried out without updating, i.e. offline, as opposed to online, model updating was performed. The algorithm was used to obtain the optimum values of the elastic modulus, yield strength and initial brace camber such that the response of the numerical counterparts best matched the measured response. A factor, termed the elastic stiffness modification factor (*ESMF*), was proposed to capture the difference between the measured structural stiffness and that in numerical models, as highlighted by the algorithm.

Finally, the results of the performance assessment based on NLTHA were reviewed in light of the experimental results. The accuracy of *EDPs* of interest predicted by numerical models was further examined using the results of shake table tests from a previous research project. The ability of FEMA P-58 to account for differences between *EDPs* recorded from the experiments and from NLTHA on performance metric was assessed. Performance assessments that had previously been carried out using modelling assumptions recommended elsewhere in literature were repeated using models with alterations arising from the examination of experimental results. Finally, the ability of the developed regression equations to predict the peak drift and acceleration response observed in the hybrid simulation testing programme was assessed.

12.2 Main Findings

Lifetime Performance Assessment of CBFs Designed to Eurocode 8

- Expected lifetime losses for CBFs designed to Eurocode 8 were shown to increase with the design behaviour factor. Losses were also shown to increase with seismic hazard. For the severe seismic hazard site considered, losses were shown to increase with frame height; however this trend was less evident for a more moderate seismic hazard.
- The results of structural analysis showed that peak drift response is more sensitive to the behaviour factor than peak acceleration response. Consequently, losses due to damage to drift sensitive components were shown to increase with the behaviour factor, while acceleration dependant losses remained relatively constant.
- Acceleration dependant losses tended to be the largest contributor to lifetime losses for the 2 and 5 storey CBFs examined, whereas for drift dependant losses were more important for the 10 storey CBFs. The contributions of losses due to collapse or demolition due to excessive residual drift to lifetime losses were minimal. This is

because the strong, infrequent earthquakes that can lead to collapse are given a low weighting in the lifetime loss calculation due the low probability of occurrence. This low weighting also means that the method by which collapse is defined is not particularly important for the assessment of lifetime losses. The probability of demolition was found to be low for CBFs, which agrees with previous work. However, this is dependent on the assumed value of residual drift requiring demolition which is still debated in literature, and such losses may become more important if a lower residual drift tolerance is assumed. Overall, additional total lifetime cost penalty from using dissipative ($q=4$ or $q=5$) rather than non-dissipative ($q=1$ or $q=2$) design ranges from 2% to 16% of initial costs, with the highest penalty observed for the tallest building at the higher seismicity site.

- Losses suffered by drift-sensitive components, can be limited by varying the stiffness or strength of the structure, which in this study was done through changing the behaviour factor. In contrast, the designer appears to have little control over lifetime losses suffered by acceleration sensitive elements, at least for typical low- and mid-rise CBFs.

Development of 'Rapid' Performance Assessment Methodology

- Peak global drift, quantified by the global drift modification factor, δ_{mod} , was found to be dependent on the ratio of fundamental structural period to mean earthquake period, T_1/T_m , and earthquake intensity represented by the degree of nonlinearity experienced, q_{eff} . These parameters, along with relative storey stiffness, β_3 , the first mode participation factor, γ , and column to brace stiffness, β_k , were also found to influence peak inter-storey drift response, as represented by the maximum drift modification factor, ϑ_{mod} . Floor acceleration magnification, PFA/PGA , was found to be a function of T_1/T_m , q_{eff} , the number of stories, NS , and the relative height of a given storey level, h_i/H .
- Comparison of these equations with the results of both NLTHA and experimental tests demonstrated a good ability to predict *EDPs*. Furthermore, improved ability to predict *EDPs* was evident when the prediction methods developed here specifically for CBFs designed to Eurocode 8 were employed as opposed to the more widely applicable methods proposed in FEMA P-58.
- The proposed rapid performance assessment method allows lifetime performance measures to be calculated with reasonable accuracy whilst avoiding the need for NLTHA. Similar values for expected lifetime losses were obtained when performance assessment

was carried out using *EDPs* obtained from regression equations developed here or calculated through NLTHA.

Hybrid Simulation & Model Updating

- The experiments executed showed that hybrid simulation for a multi-degree of freedom experimental substructure with numerous interfaces with the numerical substructure can be performed using a single actuator by representing the experimental substructure using a generic experimental element in OpenFresco.
- The model updating algorithm was validated through numerical analyses, where it was shown that the updating strategy is capable of identifying correct values of various modelling parameters. The algorithm can be used to examine experimental data and provide information about the optimum value of modelling parameters for use in future analytical models.
- Examination of the results of hybrid simulation, as well as the BRACED shake table test programme, suggest that the models typically employed for numerical analysis tend to overestimate structural stiffness. This means numerical models tend to underestimate peak inter-storey drift and, while the structure remains elastic, overestimate of peak floor acceleration.
- The ability of numerical models to match experimentally measured response, and accurately predict *EDPs* for performance assessment, were improved through reduction of the elastic stiffness using an elastic stiffness modification factor (*ESMF*).
- Modelling inaccuracies appear to be accounted for reasonably well within the FEAM P-58 performance assessment methodology. Similar lifetime losses were obtained using *EDPs* recorded from shake table tests and *EDPs* from numerical analyses when a modelling uncertainty was incorporated in the performance assessment.

12.3 Recommendations for Future Work

Lifetime Performance Assessment of CBFs

The results of performance assessment showed that the designer has the ability to limit expected lifetime losses by increasing frame stiffness, as this results in reduced drift dependant losses. However, losses to acceleration sensitive components appear more difficult to control, at least for the type of CBFs examined here. However, it is possible that these may be limited thorough the use of damping or other special devices. Therefore, it would be beneficial to carry out performance assessment examining the impact of dampers on lifetime losses. In particular,

the return on investment achieved through using such devices for various design scenarios should be investigated.

The development of collapse fragility functions for structural models of CBFs using distributed plasticity models with no deteriorating components is also deserving of further research. While there are advantages to such modelling techniques, they are incapable of capturing the cyclic strength degradation that occurs in various components under high intensity earthquakes. This means that it is challenging to develop the collapse fragility functions used in lifetime performance assessment. Instead, models of bracing members that incorporate fatigue and concentrated plasticity models of the beams and columns that allow models of plastic hinges with strength deterioration to be implemented, can be used to attempt to capture loss of component strength, and ultimately, collapse. Collapse fragility functions developed using IDA based collapse assessment from such models are generally seen to be the most robust method of quantifying collapse probability. Such fragility functions could be developed and compared to those obtained here. This would allow various potential definitions of collapse applicable for models with no strength degradation, such as numerical non-convergence as used in this study or alternative indicators like a maximum inter-storey drift or brace ductility limit, to be assessed. It is unlikely that recalculating collapse fragility would have an impact on the results of lifetime loss assessment as response at low or medium intensity earthquakes should not be changed, however the value of losses given the occurrence of higher intensity earthquakes may be affected.

Performance assessment here was carried out using PACT, which contains fragility functions and consequence functions developed for typical building components in the USA developed as part of FEMA P-58. This can introduce an element of incompatibility in loss assessment, for example in this study damage and losses to bracing elements were calculated based on functions developed for braced frames designed to AISC, even though the CBFs were designed to Eurocode 8. The development of such functions for both structural and non-structural components typically employed in Europe will ultimately be necessary for fully compatible performance assessment for Eurocode 8 based design.

The role of the modelling uncertainty term, β_m , within the FEMA P-58 performance assessment procedure needs to be further investigated and potentially refined. Although this study showed that the recommended level of modelling uncertainty appears to be in line with the difference noted between numerical and experimental results, it is possible that improvements could be made by adapting how it is employed within the performance assessment methodology. In

particular, the results of this study suggest that improvements could be made by allowing different values of β_m for different *EDPs*; as opposed to the current implementation where the same value is used for all *EDPs*. Similarly, it may be beneficial to explore ways to account for situations when the *EDPs* are thought to be overestimated by numerical models. Finally, further guidance could be developed on how modelling uncertainty should change with earthquake intensity.

More generally, the overall goal of performance assessment procedures is to improve and potentially optimize structural design. Research is needed in to how best to apply various performance objectives within design codes.

Rapid Performance Assessment Methodology

There are several clear refinements that can be made to regards to the ‘rapid’ performance assessment procedure developed for CBFs designed to Eurocode 8. Most obviously, the range of applicability of the procedure could be improved by incorporating taller CBFs in the dataset used to develop the equations, possibly involving the development of separate equations for taller buildings. Secondly, the method by which peak inter-storey drift at each floor level is calculated could be further researched and improved. The method proposed here involves using the fundamental mode shape to distribute either the predicted inter-storey drift or the roof drift over the frame height. However, this fails to account for both the change of mode shape as the structure yields and the influence of higher modes. Therefore, there is scope to improve the prediction of inter-storey drift through the development of methods to account for these, or through the development of new equations that predict inter-storey drift at each floor level. With regards to the prediction of peak floor acceleration, it is possible that developing separate equations to predict roof acceleration may improve the ability to accurately predict response. The results of other studies have suggested that for taller buildings, compared to other floor levels, higher floor acceleration magnifications tend to be experienced at the top stories, while the results of hybrid simulation here suggest a slight reduction at the roof level for low rise CBFs. As they are concentrated at the top storey, these trends are difficult to capture in regression equations that attempt to describe floor acceleration over the entire building height, meaning that separate equations for the roof level may give improved *EDP* predictions.

Hybrid Simulation and Model Updating

With regards to the hybrid simulation testing programme, clearly, the completion of the all the simulations and the online implementation of the model updating algorithm is desirable.

The model updating algorithm can potentially be improved by the use of unscented Kalman Filter (*UKF*) for parameter optimization, using a similar framework to that recently reported by Wu et al. (2018). Due to the limitations of the tcl language, this would be challenging to implement entirely within the OpenSees-OpenFresco environment. However, it may be possible to implement the *UKF* in the OpenSees source code. Alternatively, the newly developed OpenSeesPy (Zhu et al., 2018) Python interpreter could be employed. Indeed, for any type work involving OpenSees, the greater functionality of Python compared to tcl provides scope for new research that builds upon the core function of OpenSees as a finite element program, for example in the area of optimization.

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Appendix A Step-by-step Summary of Performance Assessment Procedure

1. Building design & structural modelling
 - a. Select site of case study building
 - i. Obtain ground acceleration with 10% in 50 year probability of exceedance on type A ground ($V_{s,30} > 800\text{m/s}$, as defined by Eurocode 8) from USGS (United States Geological Survey, 2014); this is used as a_g to create the EC8 design spectrum
 - b. Design structure according to EC8
 - i. Initially done using approximate T_1 value given in EC8
 - ii. EC8 type C ground conditions assumed
 - c. Create structural model in OpenSees
 - i. Perform Eigenvalue analysis to obtain T_1
 - ii. Review design using calculated T_1 ; generally the CBFs examined lie in the constant acceleration region of the design spectrum, so in most cases here the design using the estimated T_1 is not altered
 - d. Repeat steps a, b and c for all behaviour factors for the frame height for the site (e.g. all 5 storey frames at the Oakland site)
2. Select ground motion records for NLTHA
 - a. Deaggregate the seismic hazard at the site for the intensity level of interest to obtain the mean causal earthquake magnitude, M , and source-site distance, R , using USGS online tool (United States Geological Survey, 2008)
 - b. Using these, calculate the conditional spectrum (CS), using MATLAB code provided by Jayaram et al. (2011)
 - i. The conditioning period, T^* , is the average of the T_1 values for the frames of the height under examination at the site (e.g. the average T_1 of the 5 storey CBFs designed using $q=1,2,3,4,5$ frame at the Oakland site is 0.30s; hence the CS is calculated using this as T^*)
 - c. Scale and select 10 records to match the calculated CS using MATLAB code provided by Jayaram et al. (2011)
 - i. For this study only records with $V_{s,30} =$, i.e. EC8 type C ground, considered
 - ii. Only records with a magnitude of the causal magnitude obtained from deaggregation ± 1.5 were considered

- iii. Only records with a source-site distance within 15km of the causal source-site distance obtained from deaggregation were considered
 - d. Do this for 2%, 5%, 10%, 15%, 30%, 50%, 75% and 95% probabilities of exceedance in 50 years
- 3. Perform NLTHA
 - a. Separate analyses using the two horizontal components of each ground motion record
 - b. Obtain
 - i. Peak inter-storey drift between each level
 - ii. Peak absolute floor acceleration at each floor level
 - iii. Peak residual drift
 - 1. Record peak residual inter-storey drift by allowing NLTHA to run for 10s after the end of the ground motion
 - 2. Calculate peak residual inter-storey drift based on peak inter-storey drift as per Equation 6.1
- 4. Develop collapse fragility function
 - a. Collapse defined as numerical non-convergence in NLTHA
 - b. For each frame, fit lognormal distribution to the number of non-converged runs at each intensity level
 - c. If the number of non-converging analyses is insufficient (sufficiency is arbitrarily defined as requiring at least one non-convergence at 3 separate intensity levels):
 - i. Select 10 records to match the CS at 1% in 50 year intensity level
 - ii. Carry out NLTHA and monitor number of non-convergences
 - iii. If still insufficient number of collapses, repeat step ii) using records selected to match the CS at 0.5% in 50 year intensity level
 - iv. If still insufficient number of collapses, incrementally scale records selected to match the CS at 0.5% in 50 year intensity level by increasing the $S_a(T^*)$ value in steps of 0.5g until a sufficient number of non-convergences are obtained. This IDA style approach is necessary above the 0.5% in 50 year intensity level as deaggregation data is not available, and hence the CS cannot be calculated, for these highly unlikely events
 - d. Minimum value of 0.6 imposed for lognormal standard deviation of collapse fragility function to account for uncertainty in method
- 5. Create building performance model in PACT
 - a. Type and quantity of structural elements dictated by structural design

- b. Type, quantity and variation in non-structural components obtained from normative quantities estimation tool
- 6. Input *EDPs*, collapse fragility functions and seismic hazard information into *PACT*
 - a. Exclude *EDPs* from any non-converged runs
 - i. It is recommended to have at least 7 converged analyses at each intensity level (FEMA, 2012a). Although it never occurred in this research, if less than 7 converged analyses are obtained, it is therefore advisable to select further ground motion records and carry out NLTHA in order to obtain *EDPs* from enough converged analyses.
 - b. Input seismic hazard curve
 - i. $S_a(T^*)$ value at each intensity level
 - ii. Complete hazard curve using
 - 1. Maximum S_a value corresponding to the $S_a(T^*)$ value with a 1% in 50 year probability of exceedance.
 - 2. Minimum S_a value = 0.05g; obtain corresponding probability from exceedance from USGS (United States Geological Survey, 2014)
- 7. Carry out performance assessment calculation using *PACT*
- 8. Analyse Results
 - a. Record annualized repair costs, downtime and other performance metrics of interest
 - b. Use 'Drill Down' feature to extract results of each realization
 - i. Analyse results as desired, e.g. to break down losses into contributions from different performance groups

Appendix B Details of Structural Designs

Table B-1 Design Details for 2 Storey CBFs

Level	Oakland Site; $a_g = 0.5g$			Seattle Site; $a_g = 0.3g$		
	Beam	Column	Brace	Beam	Column	Brace
	q=1			q=1		
2	305x165x40	305x305x198	300x200x12	305x165x40	305x305x137	140x140x12.5
1	533x312x182	305x305x198	500x300x12	457x191x106	305x305x137	300x200x12
	q=2			q=2		
2	305x165x40	254x254x132	140x140x10	305x165x40	203x203x86	120x120x6.3
1	457x191x89	254x254x132	250x150x12.5	305x127x48	203x203x86	140x140x10
	q=3			q=3		
2	305x165x40	254x254x132	140x140x6.3	305x165x40	203x203x71	120x120x5
1	406x178x60	254x254x132	200x120x10	305x165x40	203x203x71	140x140x6.3
	q=4			q=4		
2	305x165x40	254x254x107	140x140x5	305x165x40	203x203x60	120x120x4
1	305x165x40	254x254x107	140x140x8	305x165x40	203x203x60	140x140x5
	q=5			q=5		
2	305x165x40	254x254x89	120x120x5	305x165x40	203x203x60	120x120x4
1	305x165x40	254x254x89	140x140x6.3	305x165x40	203x203x60	120x120x5

Table B-2 Design Details for 5 Storey CBFs

Level	Oakland Site; $a_g = 0.5g$			Seattle Site; $a_g = 0.3g$		
	Beam	Column	Brace	Beam	Column	Brace
	q=1			q=1		
5	610x305x238	203x203x71	200x150x8	457x191x106	203x203x46	140x140x7.1
4	610x305x238	305x305x137	300x200x12.5	457x191x106	254x254x89	250x150x12
3	610x305x238	356x406x340	450x250x12	457x191x106	356x368x202	300x200x12.5
2	610x305x238	356x406x340	500x300x12.5	457x191x106	356x368x202	400x200x12
1	610x305x238	356x406x467	500x300x16	457x191x106	356x406x287	500x300x10
	q=2			q=2		
5	457x191x133	203x203x71	120x120x7.1	406x178x67	203x203x46	120x120x5
4	457x191x133	305x305x198	250x150x10	406x178x67	203x203x86	200x120x10
3	457x191x133	305x305x198	300x200x12	406x178x67	305x305x198	250x150x10
2	457x191x133	356x406x340	500x300x8	406x178x67	305x305x198	250x150x12
1	457x191x133	356x406x340	400x200x12	406x178x67	305x305x240	300x200x10
	q=3			q=3		
5	406x178x85	203x203x60	120x120x5	305x165x46	152x152x51	120x120x4
4	406x178x85	305x305x158	140x140x8.8	305x165x46	203x203x60	200x120x6
3	406x178x85	305x305x158	140x140x12.5	305x165x46	254x254x167	140x140x8.8
2	406x178x85	356x406x287	250x150x12	305x165x46	254x254x167	250x150x8
1	406x178x85	356x406x287	250x150x12	305x165x46	305x305x198	250x150x8
	q=4			q=4		
5	356x171x57	203x203x60	120x120x4	305x165x40	152x152x44	150x100x3
4	356x171x57	305x305x137	140x140x6.3	305x165x40	203x203x46	120x120x6.3
3	356x171x57	305x305x137	140x140x8.8	305x165x40	254x254x89	140x140x7.1
2	356x171x57	356x406x235	200x120x10	305x165x40	254x254x89	140x140x8.8
1	356x171x57	356x406x235	140x140x12.5	305x165x40	305x305x118	140x140x8.8
	q=5			q=5		
5	305x127x48	203x203x60	120x120x4	305x165x40	152x152x37	100x100x3.6
4	305x127x48	305x305x118	140x140x6.3	305x165x40	152x152x51	100x100x7.1
3	305x127x48	305x305x118	200x120x8	305x165x40	254x254x89	140x140x7.1
2	305x127x48	356x368x202	200x120x10	305x165x40	254x254x89	140x140x8
1	305x127x48	356x368x202	140x140x12.5	305x165x40	254x254x107	140x140x8.8

Table B-3 Design Details for 10 Storey CBFs

	Oakland Site; $a_g = 0.5g$			Seattle Site; $a_g = 0.3g$		
Level	Beam	Column	Level	Beam	Column	Level
	q=2			q=2		
10	356x127x33	152x152x44	100x100x5	356x127x33	152x152x37	80x80x4
9	356x127x33	203x203x86	200x100x8	356x127x33	203x203x86	100x100x7.1
8	406x140x46	305x305x118	400x200x6	356x127x33	203x203x113	120x120x8.8
7	406x178x60	305x305x198	250x150x12	305x165x40	254x254x167	180x100x10
6	457x152x74	356x406x287	450x250x8	305x165x40	305x305x198	140x140x12.5
5	406x178x85	356x406x393	500x300x8	305x165x46	305x305x283	250x150x10
4	457x191x98	356x406x467	400x200x12	356x171x51	356x406x340	250x150x10
3	457x191x98	356x406x551	450x250x10	356x171x57	356x406x467	250x150x12
2	457x191x106	356x406x677	500x300x10	356x171x67	356x406x551	250x150x12
1	457x191x106	356x406x818	500x300x10	406x178x60	356x406x634	250x150x12.5
	q=3			q=3		
10	305x165x40	203x203x60	80x80x4	356x127x33	152x152x30	80x80x3
9	305x165x40	203x203x71	90x90x8.8	356x127x33	203x203x46	120x80x5
8	305x165x40	305x305x137	140x140x8	356x127x33	203x203x86	90x90x8
7	305x165x40	305x305x158	200x120x10	305x165x40	203x203x127	120x120x8
6	305x165x46	356x406x235	400x200x6	305x165x40	254x254x167	180x100x8
5	305x165x54	356x406x287	300x200x8	305x165x40	305x305x198	200x120x8
4	356x171x57	356x406x393	250x150x12	305x165x40	356x406x235	140x140x10
3	406x178x60	356x406x467	250x150x12.5	305x165x40	356x406x287	200x100x10
2	356x171x67	356x406x551	300x200x10	305x165x40	356x406x340	200x120x10
1	356x171x67	356x406x634	300x200x10	305x165x40	356x406x467	200x120x10
	q=4			q=4		
10	305x165x40	152x152x30	80x80x3	356x127x33	152x152x30	70x70x3
9	305x165x40	203x203x52	90x90x6.3	356x127x33	152x152x51	100x100x4
8	305x165x40	254x254x89	140x80x8	356x127x33	203x203x60	90x90x7.1
7	305x165x40	254x254x132	180x100x8	305x165x40	203x203x100	140x80x8
6	305x165x40	356x368x153	200x120x8	305x165x40	254x254x107	120x120x8
5	305x165x40	356x406x235	200x100x10	305x165x40	305x305x137	140x140x8
4	305x127x48	356x406x287	140x140x12.5	305x165x40	356x368x177	180x80x10
3	305x127x48	356x406x340	140x140x12.5	305x165x40	356x368x202	200x120x8
2	356x171x45	356x406x393	250x150x10	305x165x40	356x406x235	180x100x10
1	305x165x54	356x406x467	250x150x10	305x165x40	356x406x287	140x140x10

Appendix C Details of Selected Ground Motion Records

C.1 2 Storey CBFs

Table C-1 Seismic hazard deaggregation details for the two storey CBFs used to construct the Conditional Spectrum (CS) for ground motion record selection; showing magnitude, M, source-site distance R and spectral acceleration at the conditioning period, $S_a(T^*)$

	Oakland Site; $T^*=0.2s$			Seattle Site; $T^*=0.235s$		
Intensity Level	M	R, (km)	$S_a(T^*)$, (g)	M	R (km)	$S_a(T^*)$, (g)
2% in 50 Years	6.13	7.9	2.0272	6.25	6.7	1.5813
5% in 50 Years	6.03	9.1	1.5712	6.16	8.3	1.14064
10% in 50 Years	5.96	10.1	1.2541	6.09	9.3	0.85443
15% in 50 Years	5.91	11.2	1.0409	6.03	10.3	0.7035
30% in 50 Years	5.85	13.3	0.75706	5.95	12.6	0.468455
50% in 50 Years	5.79	16.3	0.54012	5.89	15.6	0.31228
70% in 50 Years	5.75	20	0.39632	5.84	19.7	0.24804
95% in 50 Years	5.70	29.75	0.1799	5.77	30.7	0.097715

Table C-2 Ground motion records selected to match the CS for NLTHA for the two storey CBFs

Oakland Site			Seattle Site		
Event Name	PEER ID	Scale Factor	Event Name	PEER ID	Scale Factor
2% in 50 Years			2% in 50 Years		
"Managua_ Nicaragua-01"	95	1.85602	"Imperial Valley-06"	191	4.74953
"Imperial Valley-06"	174	1.852631	"Chalfant Valley-02"	552	4.480285
"Westmorland"	315	5.325906	"Northridge-01"	998	1.425114
"Chalfant Valley-02"	558	1.723093	"Northridge-01"	1007	1.853236
"Whittier Narrows-01"	633	5.352604	"Northridge-01"	1038	3.528269
"Whittier Narrows-01"	683	3.006523	"Northridge-01"	1049	1.372879
"Northridge-01"	952	1.277135	"Northridge-01"	1076	3.450734
"Duzce_ Turkey"	1614	5.621182	"Chi-Chi_ Taiwan"	1524	1.181778
"Chi-Chi_ Taiwan-02"	2386	7.75761	"Chi-Chi_ Taiwan-05"	3020	5.031337
"Chi-Chi_ Taiwan-06"	3475	2.111172	"Chi-Chi_ Taiwan-05"	3188	4.491388
5% in 50 Years			5% in 50 Years		
"Imperial Valley-06"	95	1.438525	"Chalfant Valley-02"	191	3.425981
"Westmorland"	174	1.435899	"Northridge-01"	591	6.620582
"Morgan Hill"	315	4.127892	"Northridge-01"	1007	1.336796
"Whittier Narrows-01"	458	2.23585	"Northridge-01"	1024	2.428161
"Whittier Narrows-01"	633	4.148585	"Northridge-01"	1076	2.48912
"Northridge-01"	683	2.330233	"Northridge-01"	1524	0.852452
"Duzce_ Turkey"	952	0.989855	"Chi-Chi_ Taiwan"	2997	3.665167
"Chi-Chi_ Taiwan-02"	1614	4.356749	"Chi-Chi_ Taiwan-05"	3020	3.629257
"Chi-Chi_ Taiwan-06"	2386	6.012607	"Chi-Chi_ Taiwan-05"	3183	5.237933
"Imperial Valley-06"	3475	1.636283	"Imperial Valley-06"	3404	6.231946
10% in 50 Years			10% in 50 Years		
"Managua_ Nicaragua-01"	95	1.148202	"Imperial Valley-06"	191	2.566332
"Imperial Valley-06"	174	1.146105	"Whittier Narrows-01"	990	1.190903
"Westmorland"	315	3.2948	"Northridge-01"	1024	1.818886
"Morgan Hill"	458	1.78461	"Northridge-01"	1076	1.864548
"Whittier Narrows-01"	633	3.311317	"Northridge-01"	1302	3.570691
"Whittier Narrows-01"	683	1.859945	"Chi-Chi_ Taiwan"	1524	0.638555
"Northridge-01"	952	0.790082	"Chi-Chi_ Taiwan-05"	3005	7.223549
"Duzce_ Turkey"	1614	3.477469	"Chi-Chi_ Taiwan-05"	3183	3.923628
"Chi-Chi_ Taiwan-02"	2386	4.799141	"Chi-Chi_ Taiwan-05"	3340	6.418941
"Chi-Chi_ Taiwan-06"	3475	1.306048	"Chi-Chi_ Taiwan-06"	3404	4.668223

15% in 50 Years			15% in 50 Years		
"Managua_ Nicaragua-01"	95	0.953005	"Imperial Valley-06"	56	2.99699
"Imperial Valley-06"	174	0.951265	"Northridge-01"	603	7.839119
"Westmorland"	315	2.734676	"Northridge-01"	950	2.878379
"Morgan Hill"	458	1.481222	"Northridge-01"	990	0.980537
"Whittier Narrows-01"	633	2.748385	"Chi-Chi_ Taiwan"	1024	1.49759
"Whittier Narrows-01"	683	1.54375	"Chi-Chi_ Taiwan"	1076	1.535187
"Northridge-01"	952	0.655766	"Chi-Chi_ Taiwan-05"	1524	0.525758
"Duzce_ Turkey"	1614	2.886291	"Chi-Chi_ Taiwan-05"	3005	5.947552
"Chi-Chi_ Taiwan-02"	2386	3.983275	"Chi-Chi_ Taiwan-06"	3020	2.238377
"Chi-Chi_ Taiwan-06"	3475	1.084017	"Chi-Chi_ Taiwan-06"	3404	3.843609
30% in 50 Years			30% in 50 Years		
"Managua_ Nicaragua-01"	95	0.693133	"Whittier Narrows-01"	618	2.967507
"Imperial Valley-06"	174	0.691867	"Loma Prieta"	733	1.056401
"N. Palm Springs"	514	1.120267	"Northridge-01"	990	0.652932
"Chalfant Valley-02"	558	0.643491	"Northridge-01"	1038	1.045238
"Whittier Narrows-01"	633	1.998936	"Chi-Chi_ Taiwan"	1301	2.089178
"Whittier Narrows-01"	683	1.122789	"Chi-Chi_ Taiwan"	1302	1.957689
"Northridge-01"	952	0.476947	"Chi-Chi_ Taiwan-05"	2947	0.884932
"Duzce_ Turkey"	1614	2.099236	"Chi-Chi_ Taiwan-05"	3219	3.533782
"Chi-Chi_ Taiwan-02"	2386	2.897088	"Chi-Chi_ Taiwan-06"	3278	0.893652
"Chi-Chi_ Taiwan-06"	3475	0.788419	"Chi-Chi_ Taiwan-06"	3340	3.519288
50% in 50 Years			50% in 50 Years		
"Imperial Valley-06"	160	0.326804	"San Fernando"	64	5.493989
"Imperial Valley-06"	174	0.493609	"Loma Prieta"	733	0.704215
"Westmorland"	318	2.711189	"Northridge-01"	1038	0.696774
"N. Palm Springs"	514	0.799248	"Northridge-01"	1088	0.605572
"N. Palm Springs"	529	0.377311	"Chi-Chi_ Taiwan"	1301	1.392682
"Chalfant Valley-02"	558	0.459095	"Chi-Chi_ Taiwan"	1302	1.305029
"Northridge-01"	952	0.340275	"Chi-Chi_ Taiwan"	1524	0.233381
"Duzce_ Turkey"	1614	1.497688	"Chi-Chi_ Taiwan-05"	2947	0.589911
"Chi-Chi_ Taiwan-02"	2386	2.06691	"Chi-Chi_ Taiwan-05"	3212	1.252269
"Chi-Chi_ Taiwan-06"	3475	0.562493	"Chi-Chi_ Taiwan-06"	3278	0.595723
70% in 50 Years			70% in 50 Years		
"Imperial Valley-06"	174	0.362192	"San Fernando"	64	4.363805
"Westmorland"	318	1.98937	"N. Palm Springs"	520	3.053655
"Morgan Hill"	460	1.158597	"Loma Prieta"	733	0.559349
"N. Palm Springs"	514	0.586458	"Northridge-01"	1038	0.553438
"N. Palm Springs"	529	0.276857	"Northridge-01"	1066	1.706224
"Chalfant Valley-02"	558	0.336867	"Chi-Chi_ Taiwan"	1302	1.036567
"Duzce_ Turkey"	1614	1.098948	"Hector Mine"	1783	1.058165
"Chi-Chi_ Taiwan-02"	2386	1.516622	"Chi-Chi_ Taiwan-05"	2947	0.468559
"Chi-Chi_ Taiwan-03"	2635	1.937789	"Chi-Chi_ Taiwan-05"	2981	1.259691
"Chi-Chi_ Taiwan-06"	3475	0.412737	"Chi-Chi_ Taiwan-06"	3278	0.473176
95% in 50 Years			95% in 50 Years		
"Imperial Valley-06"	162	0.324359	"N. Palm Springs"	520	1.202983
"Imperial Valley-06"	174	0.164408	"Northridge-01"	1066	0.672165
"Morgan Hill"	456	0.464197	"Chi-Chi_ Taiwan-02"	2170	2.159575
"Morgan Hill"	457	0.464216	"Chi-Chi_ Taiwan-02"	2232	1.805763
"N. Palm Springs"	514	0.266209	"Chi-Chi_ Taiwan-05"	2944	0.740308
"N. Palm Springs"	529	0.125673	"Chi-Chi_ Taiwan-05"	2947	0.184588
"Whittier Narrows-01"	622	0.549992	"Chi-Chi_ Taiwan-05"	2954	0.368503
"Whittier Narrows-01"	681	0.25255	"Chi-Chi_ Taiwan-05"	2982	0.49195
"Northridge-01"	1082	0.196236	"Chi-Chi_ Taiwan-05"	3212	0.391845
"Chi-Chi_ Taiwan-06"	3475	0.187352	"Chi-Chi_ Taiwan-05"	3235	1.069901

C.2 5 Storey CBFs

Table C-3 Seismic hazard deaggregation details for the five storey CBFs used to construct the Conditional Spectrum (CS) for ground motion record selection; showing magnitude, M, source-site distance R and spectral acceleration at the conditioning period, $S_a(T^*)$

	Oakland Site; $T^*=0.3s$			Seattle Site; $T^*=0.37s$		
Intensity Level	M	R, (km)	$S_a(T^*)$, (g)	M	R, (km)	$S_a(T^*)$, (g)
2% in 50 Years	6.26	7.4	2.002	6.25	6.7	1.510
5% in 50 Years	6.16	8.3	1.534	6.16	8.3	1.081
10% in 50 Years	6.09	9.3	1.217	6.09	9.3	0.804
15% in 50 Years	6.03	10.3	1.012	6.03	10.3	0.663
30% in 50 Years	5.95	12.6	0.726	5.95	12.6	0.440
50% in 50 Years	5.89	15.6	0.522	5.89	15.6	0.292
70% in 50 Years	5.84	19.7	0.375	5.84	19.7	0.199
95% in 50 Years	5.77	30.7	0.166	5.77	30.7	0.092

Table C-4 Ground motion records selected to match the CS for NLTHA for the five storey CBFs

Oakland Site			Seattle Site		
Event Name	PEER ID	Scale Factor	Event Name	PEER ID	Scale Factor
2% in 50 Years			2% in 50 Years		
"Managua_ Nicaragua-01"	95	1.838846	"Coalinga-01"	326	4.697895
"Westmorland"	314	6.450164	"Coalinga-01"	354	6.481443
"Whittier Narrows-01"	616	3.521044	"Whittier Narrows-01"	604	3.040663
"Whittier Narrows-01"	622	4.919742	"Northridge-01"	971	3.221737
"Whittier Narrows-01"	674	5.137793	"Northridge-01"	998	1.363371
"Landers"	848	1.814748	"Northridge-01"	1065	4.095579
"Northridge-01"	1044	0.932976	"Kobe_ Japan"	1118	2.001656
"Duzce_ Turkey"	1602	1.331534	"Chi-Chi_ Taiwan-03"	2469	4.996838
"Chi-Chi_ Taiwan-03"	2628	1.455626	"Chi-Chi_ Taiwan-05"	3026	7.898426
"Chi-Chi_ Taiwan-06"	3473	2.277846	"Chi-Chi_ Taiwan-05"	3032	3.33686
5% in 50 Years			5% in 50 Years		
"Westmorland"	314	4.94389	"Coalinga-01"	342	2.423066
"Morgan Hill"	456	4.353154	"Chalfant Valley-02"	555	7.592945
"Morgan Hill"	457	3.565609	"Whittier Narrows-01"	604	2.195207
"Whittier Narrows-01"	616	2.698793	"Northridge-01"	971	2.325933
"Whittier Narrows-01"	622	3.77086	"Kobe_ Japan"	1118	1.445095
"Whittier Narrows-01"	634	3.471219	"Chi-Chi_ Taiwan"	1275	4.796959
"Landers"	848	1.390959	"Chi-Chi_ Taiwan"	1380	3.901029
"Northridge-01"	1044	0.715103	"Chi-Chi_ Taiwan"	1586	4.61484
"Chi-Chi_ Taiwan-03"	2628	1.115701	"Chi-Chi_ Taiwan-05"	3020	3.965541
"Chi-Chi_ Taiwan-06"	3470	6.614863	"Chi-Chi_ Taiwan-05"	3032	2.409046
10% in 50 Years			10% in 50 Years		
"Westmorland"	314	3.920372	"Whittier Narrows-01"	603	7.717053
"Morgan Hill"	456	3.451934	"Northridge-01"	971	1.714789
"Morgan Hill"	457	2.827432	"Northridge-01"	990	1.098277
"Whittier Narrows-01"	616	2.14007	"Northridge-01"	1065	2.179896
"Whittier Narrows-01"	622	2.99019	"Kobe_ Japan"	1118	1.065393
"Whittier Narrows-01"	634	2.752583	"Chi-Chi_ Taiwan"	1256	3.603241
"Landers"	848	1.102993	"Chi-Chi_ Taiwan"	1380	2.876025
"Northridge-01"	1044	0.567057	"Chi-Chi_ Taiwan"	1586	3.40228
"Chi-Chi_ Taiwan-03"	2628	0.884721	"Chi-Chi_ Taiwan-05"	3027	3.309814
"Chi-Chi_ Taiwan-06"	3470	5.245408	"Chi-Chi_ Taiwan-06"	3330	5.747154

15% in 50 Years			15% in 50 Years		
"Managua_ Nicaragua-01"	95	0.929391	"Whittier Narrows-01"	603	6.368777
"Mammoth Lakes-01"	232	1.488824	"Northridge-01"	971	1.415191
"Westmorland"	314	3.260048	"Northridge-01"	990	0.906393
"Morgan Hill"	456	2.870511	"Kobe_ Japan"	1118	0.879254
"Morgan Hill"	457	2.351196	"Chi-Chi_ Taiwan"	1256	2.973705
"Whittier Narrows-01"	622	2.48654	"Chi-Chi_ Taiwan"	1380	2.373543
"Whittier Narrows-01"	634	2.288954	"Chi-Chi_ Taiwan"	1586	2.807855
"Landers"	848	0.917211	"Chi-Chi_ Taiwan-05"	2995	5.554948
"Northridge-01"	954	1.970005	"Chi-Chi_ Taiwan-05"	3027	2.731544
"Northridge-01"	1044	0.471546	"Chi-Chi_ Taiwan-06"	3330	4.743047
30% in 50 Years			30% in 50 Years		
"Managua_ Nicaragua-01"	95	0.667286	"Morgan Hill"	468	3.146651
"Mammoth Lakes-01"	232	1.068948	"Whittier Narrows-01"	603	4.219603
"Westmorland"	314	2.340654	"Landers"	855	1.497735
"Morgan Hill"	456	2.060973	"Northridge-01"	968	0.73798
"New Zealand-02"	587	1.310117	"Northridge-01"	990	0.600526
"Whittier Narrows-01"	622	1.78529	"Chi-Chi_ Taiwan"	1256	1.970214
"Whittier Narrows-01"	634	1.643427	"Chi-Chi_ Taiwan-02"	2393	4.736498
"Landers"	848	0.658541	"Chi-Chi_ Taiwan-03"	2612	5.637273
"Northridge-01"	954	1.414427	"Chi-Chi_ Taiwan-05"	2995	3.680405
"Northridge-01"	1044	0.338561	"Chi-Chi_ Taiwan-06"	3306	1.068008
50% in 50 Years			50% in 50 Years		
"Managua_ Nicaragua-01"	95	0.47923	"Morgan Hill"	468	2.091084
"Mammoth Lakes-01"	232	0.767695	"Whittier Narrows-01"	603	2.804106
"Westmorland"	314	1.681006	"Loma Prieta"	746	1.507394
"Morgan Hill"	456	1.480146	"Landers"	855	0.995309
"New Zealand-02"	587	0.940897	"Kocaeli_ Turkey"	1170	2.414563
"Whittier Narrows-01"	589	0.820228	"Hector Mine"	1766	0.943416
"Landers"	848	0.47295	"Chi-Chi_ Taiwan-02"	2393	3.147605
"Northridge-01"	954	1.01581	"Chi-Chi_ Taiwan-04"	2722	2.041016
"Northridge-01"	1044	0.243147	"Chi-Chi_ Taiwan-05"	2995	2.445785
"Chi-Chi_ Taiwan-02"	2391	1.034824	"Chi-Chi_ Taiwan-05"	3210	0.97111
70% in 50 Years			70% in 50 Years		
"Managua_ Nicaragua-01"	95	0.344599	"San Fernando"	56	0.946238
"Mammoth Lakes-01"	232	0.552025	"San Fernando"	92	2.993243
"Morgan Hill"	456	1.064325	"Loma Prieta"	746	1.027299
"New Zealand-02"	587	0.676569	"Landers"	855	0.67831
"Whittier Narrows-01"	589	0.589799	"Hector Mine"	1766	0.642944
"Whittier Narrows-01"	619	0.562054	"Chi-Chi_ Taiwan-03"	2475	2.214442
"Loma Prieta"	766	0.39688	"Chi-Chi_ Taiwan-03"	2597	2.979122
"Landers"	848	0.340083	"Chi-Chi_ Taiwan-05"	3166	1.416896
"Northridge-01"	954	0.730436	"Chi-Chi_ Taiwan-05"	3179	1.298313
"Chi-Chi_ Taiwan-02"	2391	0.744109	"Chi-Chi_ Taiwan-05"	3210	0.661818
95% in 50 Years			95% in 50 Years		
"Mammoth Lakes-01"	232	0.244164	"San Fernando"	56	0.436506
"Westmorland"	315	0.742158	"Little Skull Mtn_ NV"	1743	2.155221
"New Zealand-02"	587	0.29925	"Hector Mine"	1785	0.441203
"Whittier Narrows-01"	589	0.260872	"Chi-Chi_ Taiwan-03"	2595	2.978432
"Whittier Narrows-01"	619	0.2486	"Chi-Chi_ Taiwan-03"	2597	1.374288
"Whittier Narrows-01"	633	0.643682	"Chi-Chi_ Taiwan-05"	3166	0.653623
"Whittier Narrows-01"	690	0.273474	"Chi-Chi_ Taiwan-05"	3221	0.302345
"Loma Prieta"	767	0.144532	"Chi-Chi_ Taiwan-05"	3235	0.899591
"Northridge-01"	982	0.084349	"Chi-Chi_ Taiwan-05"	3246	1.244753
"Northridge-01"	1052	0.182541	"Chi-Chi_ Taiwan-06"	3302	0.31013

C.3 10 Storey CBFs

Table C-5 Seismic hazard deaggregation details for the ten storey CBFs used to construct the Conditional Spectrum (CS) for ground motion record selection; showing magnitude, M, source-site distance R and spectral acceleration at the conditioning period, $S_a(T^*)$

	Oakland Site; $T^*=0.60s$			Seattle Site; $T^*=0.75s$		
Intensity Level	M	R, (km)	$S_a(T^*)$, (g)	M	R, (km)	$S_a(T^*)$, (g)
2% in 50 Years	6.15	7.00	1.671	7.44	38.53	0.845
5% in 50 Years	6.10	7.70	1.282	7.43	46.28	0.579
10% in 50 Years	6.05	8.52	0.996	7.37	51.81	0.413
15% in 50 Years	6.02	9.26	0.819	7.31	54.72	0.328
30% in 50 Years	5.97	11.12	0.573	7.12	59.58	0.207
50% in 50 Years	5.91	13.72	0.396	6.91	64.19	0.132
70% in 50 Years	5.88	17.44	0.271	6.72	69.37	0.087
95% in 50 Years	5.91	24.36	0.199	6.4	82.01	0.039

Table C-6 Ground motion records selected to match the CS for NLTHA for the ten storey CBFs

Oakland Site			Seattle Site		
Event Name	PEER ID	Scale Factor	Event Name	PEER ID	Scale Factor
2% in 50 Years			2% in 50 Years		
"Westmorland"	319	1.812141	"Imperial Valley-06"	166	3.577733
"Chalfant Valley-02"	558	1.580965	"Coalinga-01"	328	3.669929
"Whittier Narrows-01"	633	7.944881	"Loma Prieta"	732	1.269815
"Loma Prieta"	752	1.559147	"Loma Prieta"	772	2.5746
"Landers"	864	2.898674	"Chi-Chi_ Taiwan"	1288	3.746948
"Landers"	881	2.429504	"Chi-Chi_ Taiwan"	1289	3.115583
"Northridge-01"	960	1.42756	"Chi-Chi_ Taiwan"	1350	2.654102
"Northridge-01"	1044	1.073982	"Chi-Chi_ Taiwan-03"	2490	5.455263
"Kobe_ Japan"	1116	2.568926	"Chi-Chi_ Taiwan-03"	2615	7.551173
"Chi-Chi_ Taiwan-02"	2395	7.280256	"Chi-Chi_ Taiwan-03"	2656	4.266533
5% in 50 Years			5% in 50 Years		
"Imperial Valley-06"	160	0.828293	"Imperial Valley-06"	166	2.449794
"Westmorland"	319	1.390718	"Coalinga-01"	328	2.512924
"Landers"	881	1.86451	"Loma Prieta"	751	2.973649
"Northridge-01"	1063	0.968523	"Loma Prieta"	761	2.302069
"Northridge-01"	1083	3.276275	"Loma Prieta"	800	3.552798
"Double Springs"	1099	7.933208	"Northridge-01"	1092	3.28295
"Duzce_ Turkey"	1602	0.980749	"Kobe_ Japan"	1110	1.211427
"Chi-Chi_ Taiwan-02"	2387	6.749237	"Chi-Chi_ Taiwan"	1265	1.816961
"Chi-Chi_ Taiwan-02"	2395	5.587194	"Chi-Chi_ Taiwan"	1266	2.684117
"Chi-Chi_ Taiwan-06"	3473	2.50209	"Chi-Chi_ Taiwan"	1289	2.133345
10% in 50 Years			10% in 50 Years		
"Imperial Valley-06"	160	0.643155	"Coalinga-01"	328	1.792402
"N. Palm Springs"	529	1.548082	"Northridge-01"	1092	2.341641
"Chalfant Valley-02"	558	0.942107	"Kocaeli_ Turkey"	1177	1.645314
"Whittier Narrows-01"	633	4.734407	"Chi-Chi_ Taiwan"	1190	3.247497
"Landers"	881	1.447758	"Chi-Chi_ Taiwan"	1265	1.29599
"Northridge-01"	1004	0.719757	"Chi-Chi_ Taiwan"	1266	1.914509
"Northridge-01"	1063	0.75204	"Chi-Chi_ Taiwan"	1272	6.6475
"Northridge-01"	1082	1.429569	"Chi-Chi_ Taiwan"	1283	1.605021
"Double Springs"	1099	6.15999	"Chi-Chi_ Taiwan"	1289	1.521658
"Chi-Chi_ Taiwan-04"	2734	1.082778	"Chi-Chi_ Taiwan"	1586	4.061352

15% in 50 Years			15% in 50 Years		
"Imperial Valley-06"	160	0.529232	"Whittier Narrows-01"	328	1.425843
"N. Palm Springs"	529	1.273868	"Northridge-01"	761	1.306203
"Chalfant Valley-02"	558	0.775231	"Northridge-01"	1115	1.530248
"Whittier Narrows-01"	633	3.895797	"Kobe_ Japan"	1190	2.583362
"Landers"	881	1.191315	"Chi-Chi_ Taiwan"	1265	1.030951
"Northridge-01"	1004	0.592266	"Chi-Chi_ Taiwan"	1266	1.522979
"Northridge-01"	1063	0.61883	"Chi-Chi_ Taiwan"	1283	1.276783
"Northridge-01"	1082	1.176348	"Chi-Chi_ Taiwan-05"	1289	1.210469
"Chi-Chi_ Taiwan-04"	2734	0.890984	"Chi-Chi_ Taiwan-05"	1586	3.230777
"Chi-Chi_ Taiwan-06"	3473	1.598692	"Chi-Chi_ Taiwan-06"	3349	3.211505
30% in 50 Years			30% in 50 Years		
"Imperial Valley-06"	160	0.370045	"Kobe_ Japan"	1115	0.966252
"Mammoth Lakes-01"	230	0.967957	"Chi-Chi_ Taiwan"	1190	1.631224
"Morgan Hill"	456	4.021621	"Chi-Chi_ Taiwan"	1304	0.727932
"Chalfant Valley-02"	558	0.542051	"Chi-Chi_ Taiwan"	1569	1.430987
"Whittier Narrows-01"	595	1.075315	"Chi-Chi_ Taiwan"	1573	2.003241
"Whittier Narrows-01"	633	2.723986	"Hector Mine"	1766	1.120219
"Northridge-01"	949	0.899625	"Hector Mine"	1783	1.238943
"Northridge-01"	1004	0.414119	"Chi-Chi_ Taiwan-04"	2886	3.477973
"Northridge-01"	1063	0.432693	"Chi-Chi_ Taiwan-05"	2950	1.816794
"Northridge-01"	1082	0.822516	"Chi-Chi_ Taiwan-06"	3348	4.200731
50% in 50 Years			50% in 50 Years		
"Westmorland"	317	1.434749	"San Fernando"	56	2.465795
"Morgan Hill"	456	2.779307	"Loma Prieta"	799	0.282998
"N. Palm Springs"	529	0.615557	"Northridge-01"	1043	0.991464
"Whittier Narrows-01"	595	0.743141	"Chi-Chi_ Taiwan"	1345	1.024584
"Whittier Narrows-01"	633	1.882523	"Hector Mine"	1783	0.787114
"Whittier Narrows-01"	674	1.323604	"Hector Mine"	1785	0.993862
"Northridge-01"	1004	0.286194	"Hector Mine"	1838	2.214687
"Northridge-01"	1042	0.891792	"Chi-Chi_ Taiwan-03"	2573	5.028403
"Northridge-01"	1087	0.163352	"Chi-Chi_ Taiwan-06"	3448	2.496935
"Duzce_ Turkey"	1615	1.697305	"Chi-Chi_ Taiwan-06"	3452	1.954298
70% in 50 Years			70% in 50 Years		
"Imperial Valley-06"	158	0.675278	"San Fernando"	56	1.630757
"Mammoth Lakes-01"	231	0.538052	"Loma Prieta"	799	0.187161
"Westmorland"	317	0.982807	"Kocaeli_ Turkey"	1163	0.522883
"Morgan Hill"	456	1.903833	"Hector Mine"	1783	0.520559
"N. Palm Springs"	529	0.421659	"Hector Mine"	1785	0.657292
"Whittier Narrows-01"	595	0.509054	"Hector Mine"	1838	1.464687
"Whittier Narrows-01"	674	0.906672	"Chi-Chi_ Taiwan-03"	2634	3.197434
"Northridge-01"	1004	0.196044	"Chi-Chi_ Taiwan-04"	2727	1.626504
"Duzce_ Turkey"	1615	1.162659	"Chi-Chi_ Taiwan-05"	2957	1.107556
"Chi-Chi_ Taiwan-02"	2399	5.552799	"Chi-Chi_ Taiwan-06"	3451	1.08916
95% in 50 Years			95% in 50 Years		
"Imperial Valley-06"	160	0.069922	"Tabas_ Iran"	140	0.386236
"Imperial Valley-06"	162	0.318199	"N. Palm Springs"	520	1.79124
"Mammoth Lakes-01"	231	0.214761	"Landers"	860	0.340085
"Morgan Hill"	456	0.759905	"Landers"	887	1.493865
"N. Palm Springs"	514	0.711237	"Northridge-01"	980	0.468332
"N. Palm Springs"	529	0.168303	"Northridge-01"	1097	0.792
"Loma Prieta"	767	0.182585	"Chi-Chi_ Taiwan"	1344	0.270112
"Northridge-01"	983	0.117273	"Chi-Chi_ Taiwan-04"	2768	3.876234
"Double Springs"	1099	0.669695	"Chi-Chi_ Taiwan-05"	2981	0.586341
"Duzce_ Turkey"	1615	0.464069	"Chi-Chi_ Taiwan-06"	3485	0.736454

C.4 2 Storey MRFs

Table C-7 Seismic hazard deaggregation details for the two storey MRFs used to construct the Conditional Spectrum (CS) for ground motion record selection; showing magnitude, M, source-site distance R and spectral acceleration at the conditioning period, $S_a(T^*)$

Intensity Level	Oakland Site; $T^*=0.40s$		
	M	R, (km)	$S_a(T^*)$, (g)
2% in 50 Years	6.29	7.20	1.968
5% in 50 Years	6.20	8.00	1.516
10% in 50 Years	6.14	9.00	1.193
15% in 50 Years	6.08	9.90	1.015
30% in 50 Years	6.00	12.15	0.717
50% in 50 Years	5.94	15.25	0.494
70% in 50 Years	5.88	19.50	0.334
95% in 50 Years	5.81	31.13	0.166

Table C-8 Ground motion records selected to match the CS for NLTHA for the two storey MRFs

Oakland Site		
Event Name	PEER ID	Scale Factor
2% in 50 Years		
"Imperial Valley-06"	174	2.431184
"Coalinga-01"	367	2.31121
"Morgan Hill"	460	6.314099
"New Zealand-02"	587	2.254163
"Whittier Narrows-01"	613	7.218333
"Superstition Hills-02"	728	2.950982
"Loma Prieta"	741	1.189644
"Loma Prieta"	768	2.338296
"Landers"	848	2.266276
"Landers"	881	3.89362
5% in 50 Years		
"Imperial Valley-06"	160	0.781686
"Imperial Valley-06"	174	1.872802
"Coalinga-01"	367	1.780384
"Morgan Hill"	460	4.86391
"Whittier Narrows-01"	613	5.560464
"Loma Prieta"	741	0.916413
"Loma Prieta"	768	1.801249
"Landers"	848	1.74577
"Landers"	881	2.999354
"Northridge-01"	1063	0.914442
10% in 50 Years		
"Imperial Valley-06"	160	0.614882
"Imperial Valley-06"	162	2.499511
"Coalinga-01"	367	1.400467
"Morgan Hill"	460	3.825998
"Whittier Narrows-01"	613	4.373913
"Loma Prieta"	741	0.720859
"Loma Prieta"	768	1.416879
"Landers"	848	1.373239
"Landers"	881	2.35932
"Northridge-01"	1063	0.719309

15% in 50 Years		
"Managua_ Nicaragua-01"	95	1.023158
"Friuli_ Italy-01"	125	1.285898
"Imperial Valley-06"	162	2.127361
"Corinth_ Greece"	313	1.657333
"Coalinga-01"	367	1.191953
"Morgan Hill"	460	3.256349
"Whittier Narrows-01"	592	2.095297
"Loma Prieta"	741	0.613531
"Landers"	881	2.008043
"Northridge-01"	1063	0.612212
30% in 50 Years		
"Corinth_ Greece"	313	1.171212
"Morgan Hill"	460	2.301213
"Whittier Narrows-01"	592	1.480715
"Whittier Narrows-01"	633	2.395159
"Whittier Narrows-01"	674	2.173252
"Loma Prieta"	741	0.433573
"Loma Prieta"	767	0.98759
"Northridge-01"	983	0.388052
"Northridge-01"	1063	0.432641
"Duzce_ Turkey"	1615	1.840751
50% in 50 Years		
"Imperial Valley-06"	167	1.640099
"Morgan Hill"	460	1.58446
"Whittier Narrows-01"	592	1.01952
"Whittier Narrows-01"	633	1.649145
"Whittier Narrows-01"	674	1.496355
"Loma Prieta"	741	0.298529
"Loma Prieta"	767	0.679988
"Northridge-01"	960	0.48358
"Northridge-01"	983	0.267186
"Northridge-01"	1063	0.297887
70% in 50 Years		
"Imperial Valley-06"	167	1.108067
"Morgan Hill"	460	1.070477
"Whittier Narrows-01"	592	0.688798
"Whittier Narrows-01"	633	1.114179
"Whittier Narrows-01"	674	1.010952
"Loma Prieta"	741	0.201689
"Loma Prieta"	767	0.459407
"Northridge-01"	960	0.326712
"Northridge-01"	983	0.180514
"Northridge-01"	1063	0.201256
95% in 50 Years		
"Imperial Valley-06"	167	0.550962
"Mammoth Lakes-01"	232	0.314626
"Whittier Narrows-01"	592	0.342489
"Whittier Narrows-01"	633	0.554001
"Whittier Narrows-01"	674	0.502673
"Loma Prieta"	741	0.100286
"Loma Prieta"	767	0.22843
"Northridge-01"	959	0.186561
"Northridge-01"	983	0.089756
"Northridge-01"	1044	0.132363

Appendix D Sample OpenSees Code for NLTHA

D.1 5 Storey CBF; $q = 4$; Oakland Site

```
#####
##### -----
# Model of 5 storey CBF, q=4, Oakland site
# Created by: John Hickey TCD
# Last Edited on: 18/10/2016
# -----
#
# Set Up & Source Definition
#####
#####
# wipe; # clear memory of past model definitions
# model BasicBuilder -ndm 2 -ndf 3; # Define the model builder, ndm = #dimension, ndf
# = #dofs
# source WSection.tcl; # procedure for creating steel I section
# source HSSsection.tcl; # procedure for creating standard steel HSS section
# source RHSsection.tcl; # procedure for creating standard steel RHS section
#####
#####
# Define Analysis Type and create output folder
#####
#####
# Define type of analysis: "pushover" = pushover; "dynamic" = dynamic; "eigen" = eigen;
static_Lateral
set analysisType "dynamic";

puts "Running 2D Oakland q4"
source variables.tcl; #

puts "Earthquake read in as"
puts $GMfileH
puts "scaling read in as"
puts $GMfact
puts "dt read in as"
puts $time_step
puts "time step read in as"
puts $deltaT
puts "duration read in as"
puts $tFinal

#####
#####
# Define Building Geometry, Nodes, and Constraints
#####
#####
# define structure-geometry parameters
set WBay 6000.; # bay width in mm
set HStory 3600.; # 1st story height in mm

set NStory 5; # number of stories above ground level ----- you can
change this.
set NBay 3; # number of bays (max 9) -----you can
change this.

set brace_wp_x 611.
set brace_wp_y 366.

set beam_half 250.
set column_half 200.

set gusset_length_x 424.
set gusset_length_y 254

set fixed_braced_x [expr $beam_half + 0.75*($gusset_length_x)]
set fixed_braced_y [expr $column_half + $gusset_length_y]
set fixed_tab_y [expr $beam_half]
set fixed_tab_x [expr $column_half]

set alpha [expr (atan($HStory/$WBay))]
```

```

set PI [expr 2.*asin(1.0)]

# brace effective lenght, number of elements per brace, and initial imperfection
set Leff [expr ($WBay)/(cos($alpha))- $brace_wp_x/cos($alpha)-
($brace_wp_x)/cos($alpha)] ;#effective brace lenght, works for 45 degree angles
set noEle 8; # number of elements per brace
set p_ratio 1000.; #2000 corresponds to 0.05% of the effective lenght
set p [expr 1./$p_ratio*$Leff]; #imperfection of the brace
set eleL [expr $Leff/$noEle]

#####
#Define Floors and Piers

#Add/delete as appropriate
#####
set Pier1 0.0; # leftmost column line
set Pier2 [expr $Pier1 + $WBay];
set Pier3 [expr $Pier2 + $WBay];
set Pier4 [expr $Pier3 + $WBay];
set Pier5 [expr $Pier4 + $WBay];

set Floor1 0.0; # ground floor
set Floor2 [expr $Floor1 + $HStory];
set Floor3 [expr $Floor2 + $HStory];
set Floor4 [expr $Floor3 + $HStory];
set Floor5 [expr $Floor4 + $HStory];
set Floor6 [expr $Floor5 + $HStory];
set Floor7 [expr $Floor6 + $HStory];
set Floor8 [expr $Floor7 + $HStory];
set Floor9 [expr $Floor8 + $HStory];

# define BEAM COLUMN JOINT NODE COORDINATES
# nodeID a0b; level=a, pier=b, ie 303 = third level, third pier
for {set level 1} {$level <=[expr $NStory+1]} {incr level 1} {
  set Z 0
  set Y [expr ($level-1)*$HStory];
  for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
    set X [expr ($pier-1)*$WBay];
    set nodeID [expr $level*100+$pier]
    node $nodeID $X $Y; # actually define node
  }
}

puts "In file you think you're in"
set testmass 17.07896628 ;#in Mg
set roofmass 12.45511307 ;#in Mg
set selfmass 0.0 ;#in Mg
set m [expr $testmass + $selfmass]
set m_roof [expr $roofmass + $selfmass]

for {set level 2} {$level <=[expr $NStory]} {incr level 1} {
  for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
    if {$pier==1 || $pier== $NBay+1} {
      set nodeID [expr $level*100+$pier]
      mass $nodeID [expr (3*$m/4)] 0 0;
    } else {
      set nodeID [expr $level*100+$pier]
      mass $nodeID [expr (3*$m/2)] 0 0;
    }
  }
}

set level [expr $NStory+1]
for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
  if {$pier==1 || $pier== $NBay+1} {
    set nodeID [expr $level*100+$pier]
    mass $nodeID [expr (3*$m_roof/4)] 0 0;
  } else {
    set nodeID [expr $level*100+$pier]
    mass $nodeID [expr (3*$m_roof/2)] 0 0;
  }
}

#Define Extra Nodes in Columns

```



```

# define extra nodes in the columns
# nodes at the beam-column conection
#Node numbering Level-Pier-1 = below joint
#Node numbering Level-Pier-2 = above joint

#these need to be defined individually until you come up with a way of identifying how a
gridline is braced
# | / | \ |
# | / | \ |
# | / | \ |
# | / | \ |
# | / | \ |
# | / | \ |
# | / | \ |
#Vertical GL1
node 112 $Pier1 [expr $Floor1+$fixed_braced_y]

for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
  if {$level ==[expr $NStory+1]} {
    set nodeID [expr (($level*100)+11)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier1 [expr $floor-$fixed_tab_y]
  } else {

    set nodeID [expr (($level*100)+11)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier1 [expr $floor-$fixed_tab_y]

    set nodeID [expr (($level*100)+12)]
    node $nodeID $Pier1 [expr $floor+$fixed_braced_y]
  }
}

#Vertical GL2

for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
  if {$level ==[expr $NStory+1]} {
    set nodeID [expr (($level*100)+21)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier2 [expr $floor-$fixed_braced_y]
  } else {

    set nodeID [expr (($level*100)+21)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier2 [expr $floor-$fixed_braced_y]

    set nodeID [expr (($level*100)+22)]
    node $nodeID $Pier2 [expr $floor+$fixed_tab_y]
  }
}

#VGL3
for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
  if {$level ==[expr $NStory+1]} {
    set nodeID [expr (($level*100)+31)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier3 [expr $floor-$fixed_braced_y]
  } else {

    set nodeID [expr (($level)*100)+31]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier3 [expr $floor-$fixed_braced_y]

    set nodeID [expr (($level*100)+32)]
    node $nodeID $Pier3 [expr $floor+$fixed_tab_y]
  }
}

#VGL4
node 142 $Pier4 [expr $Floor1+$fixed_braced_y]

```

```

for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
  if {$level ==[expr $NStory+1]} {
    set nodeID [expr (($level*100)+41)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier4 [expr $floor-$fixed_tab_y]
  } else {

    set nodeID [expr (($level*100)+41)]
    set floor [expr ($level-1)*$HStory]
    node $nodeID $Pier4 [expr $floor-$fixed_tab_y]

    set nodeID [expr (($level*100)+42)]
    node $nodeID $Pier4 [expr $floor+$fixed_braced_y]
  }
}

#Define Extra Nodes in Beams
#Level-gridline-4,5 left
#Level-gridline-6,7 right
#level2

for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
  set floor [expr ($level-1)*$HStory]

  set nodeID [expr (($level*100)+16)]
  node $nodeID [expr $Pier1+$fixed_braced_x] $floor
  set nodeID [expr (($level*100)+24)]
  node $nodeID [expr $Pier2-$fixed_braced_x] $floor
  set nodeID [expr (($level*100)+26)]
  node $nodeID [expr $Pier2+$fixed_tab_x] $floor
  set nodeID [expr (($level*100)+27)]
  node $nodeID [expr $Pier2+$fixed_tab_x] $floor
  set nodeID [expr (($level*100)+34)]
  node $nodeID [expr $Pier3-$fixed_tab_x] $floor
  set nodeID [expr (($level*100)+35)]
  node $nodeID [expr $Pier3-$fixed_tab_x] $floor
  set nodeID [expr (($level*100)+36)]
  node $nodeID [expr $Pier3+$fixed_braced_x] $floor
  set nodeID [expr (($level*100)+44)]
  node $nodeID [expr $Pier4-$fixed_braced_x] $floor
}

puts "here"
#define braces
#copy this for each braced bay

#set brace_wp_x 200
#set brace_wp_y 200

# nodes of the left brace of storey 1

#bay 1
for {set level 1} {$level <=[expr $NStory]} {incr level 1} {
  set shift [expr $level*1000]
  set orient "pos"
  set base_floor [expr ($level-1)*$HStory]
  set top_floor [expr ($level)*$HStory]
  set Xi [expr $Pier1+$brace_wp_x]; # x coordinate of node 3101
  set Yi [expr $base_floor+$brace_wp_y]; # y coordinate of node 3101
  set Zi 0.0
  set eleL [expr $Leff/$noEle]
  for { set i 1 } { $i <= [expr $noEle+1] } { incr i } {
    #local coordinates
    set xm [expr $eleL*(i-1)]
    set zm 0
    set ym [expr 4.*$p/$Leff*$xm*(1.-$xm/$Leff)]; ;
    #global coordinates
    if {$orient == "pos"} {
      set Xm [expr $Xi+cos($alpha)*$xm-$ym*sin($alpha)];
      set Ym [expr $Yi+sin($alpha)*$xm+$ym*cos($alpha)];
      #set Ym [expr $Yi+sin($alpha)*$xm];
      set Zm [expr $Zi+$zm];
    } else {
      set Xm [expr $Xi+cos($alpha)*$xm];

```

```

        set Ym [expr $Yi-sin($alpha)*$xm];
        set Zm [expr $Zi-$zm];
    }
    if { $i == [expr $noEle+1] } {
        set Xm [expr $Pier2-$brace_wp_x]; # x coordinate of the last node
        set Ym [expr $top_floor-$brace_wp_y]; # y coordinate of of the last node
        set Zm 0.0
    }
    #define nodes
    node [expr $shift+100+$i] $Xm $Ym
    #puts "node: [expr $shift+100+$i] $Xm $Ym $Zm"

    #define extra nodes at start and end for springs
    if { $i==1 } {
        node [expr $level*100+10+8] $Xm $Ym
    }
    if { $i==[expr $noEle+1] } {
        node [expr ($level+1)*100+20+9] $Xm $Ym
    }
}

#bay 3
for {set level 1} {$level <=[expr $NStory]} {incr level 1} {
    set shift [expr $level*1000]
    set orient "neg"
    set base_floor [expr ($level-1)*$HStory]
    set top_floor [expr ($level)*$HStory]
    set Xi [expr $Pier4-$brace_wp_x];
    set Yi [expr $base_floor+$brace_wp_y];
    set Zi 0.0
    set eleL [expr $Leff/$noEle]
    for { set i 1 } { $i <= [expr $noEle+1] } { incr i } {
        #local coordinates
        set xm [expr $eleL*($i-1)]
        set zm 0.0
        set ym [expr 4.*$p/$Leff*$xm*(1.-$xm/$Leff)];
        #global coordinates
        if {$orient == "pos"} {
            set Xm [expr $Xi+cos($alpha)*$xm];
            set Ym [expr $Yi+sin($alpha)*$xm];
            set Zm [expr $Zi+$zm];
        } else {
            set Xm [expr $Xi-cos($alpha)*$xm+$ym*sin($alpha)];
            set Ym [expr $Yi+sin($alpha)*$xm+$ym*cos($alpha)];
            #set Ym [expr $Yi+sin($alpha)*$xm];
            set Zm [expr $Zi-$zm];
        }
        if { $i == [expr $noEle+1] } {
            set Xm [expr $Pier3+$brace_wp_x]; # x coordinate of the last node
            set Ym [expr $top_floor-$brace_wp_y]; # y coordinate of of the last node
            set Zm 0.0
        }
        #define nodes
        node [expr $shift+300+$i] $Xm $Ym $Zm

        #define extra nodes at start and end for springs
        if { $i==1 } {
            node [expr $level*100+40+8] $Xm $Ym
        }
        if { $i==[expr $noEle+1] } {
            node [expr ($level+1)*100+30+9] $Xm $Ym
        }
    }
}

#Define EqualDOF for shear tab beam column connections

# define constraints for pinned beam-to-column connection; fixed in all directions bar z
rotation

```

```

for {set level 2} {$level <= $NStory} {incr level 1} { ;#added a +1 to NStory here
    set Inode [expr ($level*100)+26]
    set Jnode [expr ($level*100)+27]
    equalDOF $Inode $Jnode 1 2

    set Inode [expr ($level*100)+34]
    set Jnode [expr ($level*100)+35]
    equalDOF $Inode $Jnode 1 2
}

# equalDOF 326 327 1 2 3 4 5
# equalDOF 334 335 1 2 3 4 5
# equalDOF 336 337 1 2 3 4 5
# equalDOF 344 345 1 2 3 4 5

#Gusset Plate Springs

#bay1

for {set level 1} {$level <= $NStory} {incr level 1} {
    set Inode [expr ($level*100)+18]
    set Jnode [expr ($level*1000)+101]
    equalDOF $Inode $Jnode 1 2

    set Inode [expr (($level+1)*100)+29]
    set Jnode [expr (((level)*1000)+(101+$noEle))]
    equalDOF $Inode $Jnode 1 2
}

#bay3

for {set level 1} {$level <= $NStory} {incr level 1} {
    set Inode [expr ($level*100)+48]
    set Jnode [expr ($level*1000)+301]
    equalDOF $Inode $Jnode 1 2

    set Inode [expr (($level+1)*100)+39]
    set Jnode [expr ($level*1000)+(301+$noEle)]
    equalDOF $Inode $Jnode 1 2
}

# BOUNDARY CONDITIONS
#fix base of frame
fix 101 1 1 1;
fix 102 1 1 1;
fix 103 1 1 1;
fix 104 1 1 1;

#fix other joint nodes in z and rotation

puts "Nodes done"

#####
#####
#           Define Materials and Sections
#####
#####

set matID_BC 1
set matID_Beam 10
set matID_fatBC 2
set matID_Brace 3
set matID_fatBrace 4
set matID_GT 5
set matID_RT 6
set matID_gusset 7
set matID_RH_tors 8
set matID_sheartab 9

# define material for nonlinear beams and columns

```

```

set Es 210000.0; # modulus of elasticity for steel
set Fy 355.0; # yield stress of steel
set b 0.008; # strain hardening ratio
uniaxialMaterial Steel02 $matID_BC $Fy $Es $b 20 0.925 0.15 0.0005 0.01 0.0005 0.01
uniaxialMaterial Fatigue $matID_fatBC $matID_BC

set Es 210000.0; # modulus of elasticity for steel
set Fy 355.0; # yield stress of steel
set b 0.008; # strain hardening ratio
uniaxialMaterial Steel02 $matID_Beam $Fy $Es $b 20 0.925 0.15 0.0005 0.01 0.0005 0.01

# define material for braces

set Fy_b 355.0; # yield stress of steel
set E0 0.19 ; #Suhaib parameters
set m -0.458
uniaxialMaterial Steel02 $matID_Brace $Fy_b $Es $b 20 0.925 0.15 0.0005 0.01 0.0005
0.01
#uniaxialMaterial Fatigue $matID_fatBrace $matID_Brace -E0 $E0 -m $m -min -1.0 -max
0.017
uniaxialMaterial Fatigue $matID_fatBrace $matID_Brace -E0 $E0 -m $m

#Define Hsiao material for gusset plate spring - assume constant at each level
set My 7066678.438;
set Krot 184828206.4;
set b 0.01
uniaxialMaterial Steel02 $matID_gusset $My $Krot $b 20 0.925 0.15 0.0005 0.01 0.0005
0.01

#Define Torsion material
uniaxialMaterial Elastic $matID_RH_tors 10.e15

# define material for ghost truss
uniaxialMaterial Elastic $matID_GT 500.0

# define material for rigid truss elements
uniaxialMaterial Elastic $matID_RT $Es

# define sections
#-----
set secTagC1 11
set secTagC2 12
set secTagC3 13
set secTagC4 14
set secTagC1side 15
set secTagC2side 16
set secTagC3side 17
set secTagC4side 18
set secTagB 20
set secTagBr1 31
set secTagBr2 32
set secTagBr3 33
set secTagBr4 34
set secTagBr5 35
set secTagBr6 36
set secTagBr7 37
set secTagBr8 38
set secTagGP 40

# command: WSection3D secID matID matTorsion d bf tf tw nfdw nftw nbf nftf J
# column: W14x176

#columns
#c1
# 356x406x235
set cd1 381 ;#column depth
set cbf1 394.8 ;#column breath
set ctf1 30.2 ;# column flange thickness
set ctw1 18.4 ;#column web thickness

# 305x305x137
set cd2 320.5 ;#column depth

```

```

set cbf2      309.2    ;#column breath
set ctf2      21.7     ;# column flange thickness
set ctw2      13.8     ;#column web thickness

# 203x203x60
set cd3 209.6 ;#column depth
set cbf3 205.8 ;#column breath
set ctf3 14.2 ;# column flange thickness
set ctw3 9.4 ;#column web thickness

# 152x152x37
set cd4 161.8
set cbf4 154.4
set ctf4 11.5
set ctw4 8

set cd5 406.4 ;#column depth
set cbf5 418.5 ;#column flange breath
set ctf5 67.5 ;# column flange thickness
set ctw5 42.1 ;#column web thickness

#Beam
# 356x171x57
set bd 358 ;#beam depth
set bbf 172.2 ;#beam breath
set btf 13 ;# beam flange thickness
set btw 8.1 ;#beam web thickness

#calculate areas & I's; used later for rigid links
set acol1 [expr (((cd1-2*ctf1)*ctw1)+2*(cbf1*ctf1))] ;#area if column 1
set acol2 [expr (((cd2-2*ctf2)*ctw2)+2*(cbf2*ctf2))]
set acol3 [expr (((cd3-2*ctf3)*ctw3)+2*(cbf3*ctf3))]
set acol4 [expr (((cd3-2*ctf4)*ctw4)+2*(cbf4*ctf4))]
set abeam [expr (((bd-2*btf)*btw)+2*(bbf*btf))]

set Icol1 [expr (((cbf1*cd1**3)-($cbf1-$ctw1)*((cd1-2*ctw1)**3))/12)] ;#area if
column 1
set Icol2 [expr (((cbf2*cd2**3)-($cbf2-$ctw1)*((cd2-2*ctw2)**3))/12)]
set Icol3 [expr (((cbf3*cd3**3)-($cbf3-$ctw1)*((cd3-2*ctw3)**3))/12)]
set Icol4 [expr (((cbf4*cd4**3)-($cbf4-$ctw1)*((cd4-2*ctw4)**3))/12)]
set Ibeam [expr (((bbf*bd**3)-($bbf-$btw)*((bd-2*btw)**3))/12)]

set Iycol1 [expr ((2*($ctf1*cbf1**3)-($cd1-2*ctf1)*((ctw1)**3))/12)]
set Iycol2 [expr ((2*($ctf2*cbf2**3)-($cd2-2*ctf2)*((ctw2)**3))/12)]
set Iycol3 [expr ((2*($ctf3*cbf3**3)-($cd3-2*ctf3)*((ctw3)**3))/12)]
set Iycol4 [expr ((2*($ctf4*cbf4**3)-($cd4-2*ctf4)*((ctw4)**3))/12)]
set Iybeam [expr ((2*($btf*bbf**3)-($bd-2*btf)*((btw)**3))/12)]

set Jcol1 [expr ((2*($cbf1*ctf1**3)+($cd1)*((ctw1)**3))/3)]
set Jcol2 [expr ((2*($cbf2*ctf2**3)+($cd2)*((ctw2)**3))/3)]
set Jcol3 [expr ((2*($cbf3*ctf3**3)+($cd3)*((ctw3)**3))/3)]
set Jcol4 [expr ((2*($cbf4*ctf4**3)+($cd4)*((ctw4)**3))/3)]
set Jbeam [expr ((2*($bbf*btf**3)+($bd)*((btw)**3))/3)]

# define sections
#columns
WSection $secTagC1 $matID_BC $cd1 $cbf1 $ctf1 $ctw1 8 2 8 4
WSection $secTagC2 $matID_BC $cd2 $cbf2 $ctf2 $ctw2 8 2 8 4
WSection $secTagC3 $matID_BC $cd3 $cbf3 $ctf3 $ctw3 8 2 8 4
WSection $secTagC4 $matID_BC $cd4 $cbf4 $ctf4 $ctw4 8 2 8 4

# Edge columns
WSection_rotated $secTagC1side $matID_BC $cd1 $cbf1 $ctf1 $ctw1 8 2 8 4
WSection_rotated $secTagC2side $matID_BC $cd2 $cbf2 $ctf2 $ctw2 8 2 8 4
WSection_rotated $secTagC3side $matID_BC $cd3 $cbf3 $ctf3 $ctw3 8 2 8 4
WSection_rotated $secTagC4side $matID_BC $cd4 $cbf4 $ctf4 $ctw4 8 2 8 4

# beam
WSection $secTagB $matID_Beam $bd $bbf $btf $btw 8 2 8 4

#braces
RHSsection $secTagBr1 $matID_fatBrace 140. 140. 12.5 5 4 5 2
RHSsection $secTagBr2 $matID_fatBrace 200. 120. 10. 5 4 5 2

```

```

RHSsection $secTagBr3 $matID_fatBrace 140. 140. 8.8 5 4 5 2
RHSsection $secTagBr4 $matID_fatBrace 140. 140. 6.3 5 4 5 2
RHSsection $secTagBr5 $matID_fatBrace 120. 120. 4. 5 4 5 2
RHSsection $secTagBr6 $matID_Brace 200. 120. 5. 5 4 5 2
RHSsection $secTagBr7 $matID_Brace 120. 120. 5. 5 4 5 2
RHSsection $secTagBr8 $matID_Brace 150. 100. 3. 5 4 5 2

puts "Materials and Sections Done"

#####
#####
#           Define Geometric Transformation
#####
#####
#           set transfTag_C 1
#           set transfTag_Brace 2
#           set transfTag_B 3

# rigid links of columns; columns
geomTransf PDelta $transfTag_C

# braces, gusset plates
geomTransf Corotational $transfTag_Brace

# beams
geomTransf PDelta $transfTag_B

#####
#define dx,dy, dz
#####
set dx [expr $WBay-$brace_wp_x-$brace_wp_x]
set dy [expr $HStory-$brace_wp_y-$brace_wp_y]
set dz 0
set DXZ [expr $Leff/cos($alpha)-$dy]

#####
#####
#           Define Elements
#####
#####

set tol 1.e-8
set maxIter 10

# define columns of a braced frame:
#-----

# Define Beam-Column Elements
set np 3; # number of Gauss integration points for nonlinear curvature distribution--
np=2 for linear distribution ok
# columns
#numbering lab0 implies column element; a is start level; b is pier no.
#

set N0col 1000

#need to build bottom columns one by one untill a brace identifier is created

        element nonlinearBeamColumn 1110 112 211 $np $secTagC1 $transfTag_C; #
columns
        element nonlinearBeamColumn 1120 102 221 $np $secTagC1 $transfTag_C; #
columns
        element nonlinearBeamColumn 1140 103 231 $np $secTagC1 $transfTag_C; #
columns
        element nonlinearBeamColumn 1150 142 241 $np $secTagC1 $transfTag_C;

#columns run from start_level-gridline-2 to end_level-gridline-1
set N0col 1000; # column element numbers
for {set level 2} {$level <= $NStory} {incr level 1} {
    for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
        set elemID [expr $N0col + $level*100 + $pier*10]
        set nodeI [expr $level*100 + $pier*10+2]
        set nodeJ [expr ($level+1)*100 + $pier*10+1]
        #define column section
        if { $level==2} {
            if { $pier==1 || $pier==4 } {
                set secTag $secTagC1
            }
        }
    }
}

```

```

        }
        if { $pier==2 || $pier==3 } {
            set secTag $secTagC1
        }
    }
    if { $level==3 || $level==4 } {
        if { $pier==1 || $pier==4 } {
            set secTag $secTagC2
        }
        if { $pier==2 || $pier==3 } {
            set secTag $secTagC2
        }
    }
    if { $level==5 || $level==7 } {
        if { $pier==1 || $pier==4 } {
            set secTag $secTagC3
        }
        if { $pier==2 || $pier==3 } {
            set secTag $secTagC3
        }
    }
    if { $level==8 || $level==9 } {
        set secTag $secTagC4
    }
    element nonlinearBeamColumn $elemID $nodeI $nodeJ $np $secTag $transfTag_C; #
columns
    }
}

```

#define rigid links at bottom of columns - run from level-0-gridline to level-gridline-2
#numbered 1-level-gridline-1

```

    set Jrigid [expr $Jcol1*10]
    set G [expr $Es/2.0/(1+0.3)]
set level 0
set Arigid [expr $acol1*10.]; # area of the column multiplied by 10
set IrigidZ [expr $Icol1*10]; # Iz of the column multiplied by 10
set IrigidY [expr $Iycol1*10]; # Iy of the column multiplied by 10
set Erigid [expr 210000*100]

```

#define rigid links in bottom storey
set type "beam"

```

element elasticBeamColumn 1112 101 112 $Arigid $Erigid $IrigidZ
$transfTag_C;
element elasticBeamColumn 1152 104 142 $Arigid $Erigid $IrigidZ
$transfTag_C;

```

```

#rigidLink $type 101 112
#rigidLink $type 105 152

```

#Rigid links are sized on the basis that the columns run 2 storeys each

```

for {set level 2} {$level <= $NStory} {incr level 1} {
    for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
        set elemID [expr $N0col + $level*100 + $pier*10+2]
        set nodeI [expr $level*100 + $pier]
        set nodeJ [expr ($level)*100 + $pier*10+2]
        if { $level==2 } {
            set Arigid [expr $acol1*10.]
            set IrigidY [expr $Icol1*10]
        }
        if { $level==3 || $level==4 } {
            set Arigid [expr $acol2*10.]
            set IrigidY [expr $Icol2*10]
        }
        if { $level==5 || $level==6 } {
            set Arigid [expr $acol3*10.]
            set IrigidY [expr $Icol3*10]
        }
        if { $level==7 || $level==8 } {
            set Arigid [expr $acol4*10.]
            set IrigidY [expr $Icol4*10]
        }
    }
}

```



```

        element elasticBeamColumn    $elemID        $nodeI        $nodeJ $Arigid $Erigid
$IrigidZ    $transfTag_C;          # columns
        #rigidLink $type $nodeI $nodeJ
    }
}

#define rigid links at top of columns - run from level-gridline-1 to level-0-gridline
#numbered 1-level-gridline-1

set level 0
set Arigid [expr $acol1*20.]; # area of the column multiplied by 10
set IrigidZ [expr $icol1*20]; # Iz of the column multiplied by 10
set IrigidY [expr $iycol1*20]; # Iy of the column multiplied by 10
for {set level 2} {$level <= $NStory+1} {incr level 1} {
    for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
        set elemID [expr $N0col + $level*100 + $pier*10+1]
        set nodeI [expr $level*100 + $pier*10+1]
        set nodeJ [expr ($level)*100 + $pier]
        if { $level==2 } {
            set Arigid [expr $acol1*10.]
            set IrigidY [expr $icol1*10]
        }
        if { $level==3 || $level==4 } {
            set Arigid [expr $acol2*10.]
            set IrigidY [expr $icol2*10]
        }
        if { $level==5 || $level ==6 } {
            set Arigid [expr $acol3*10.]
            set IrigidY [expr $icol3*10]
        }
        if { $level==7 || $level ==8 } {
            set Arigid [expr $acol4*10.]
            set IrigidY [expr $icol4*10]
        }
        element elasticBeamColumn    $elemID        $nodeI        $nodeJ $Arigid $Erigid
$IrigidZ    $transfTag_C;          # columns
        #rigidLink $type $nodeJ $nodeI
    }
}

#define beams
#2-level-bay-0
#run from level-pier-6 to level-pier-4

set N0beam 2000;    # beam element numbers
set M0 0
for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
    for {set bay 1} {$bay <= $NBay} {incr bay 1} {
        set elemID [expr $N0beam + $level*100 + $bay*10]
        set nodeI [expr $level*100 + $bay*10+6]
        set nodeJ [expr $level*100 + ($bay+1)*10+4]
        element nonlinearBeamColumn $elemID $nodeI $nodeJ $np $secTagB $transfTag_B;    #
    }
}

#Define rigid links in beam
#have to do this manually for each bay because of differences in braced and non braced
#2-level-pier-1 at left of beam
#2-level-pier-2 at right of beam
set N0beam 2000;    # beam element numbers
set M0 0
set IrigidY [expr $Iybeam*10]
set IrigidZ [expr $Ibeam*10]
set Jrigid [expr $Jbeam*10]
set Arigid [expr $abeam*10]
for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
    #pier1
    set elemID [expr $N0beam + $level*100 +10+1]
    set nodeI [expr $level*100+1]
    set nodeJ [expr $level*100 + 16]

```

```

        element elasticBeamColumn   $elemID      $nodeI      $nodeJ      $Arigid $Erigid
$IrigidZ      $transfTag_B;
        #rigidLink $type $nodeI $nodeJ

        #pier2
        set elemID [expr $N0beam + $level*100 +20+2]
        set nodeI [expr $level*100+2]
        set nodeJ [expr $level*100 + 24]
        element elasticBeamColumn   $elemID      $nodeI      $nodeJ      $Arigid $Erigid
$IrigidZ      $transfTag_B;
        #rigidLink $type $nodeI $nodeJ

        set elemID [expr $N0beam + $level*100 +20+1]
        set nodeI [expr $level*100+2]
        set nodeJ [expr $level*100 + 27]
        element elasticBeamColumn   $elemID      $nodeI      $nodeJ      $Arigid $Erigid
$IrigidZ      $transfTag_B;
        #rigidLink $type $nodeI $nodeJ

        #pier3
        set elemID [expr $N0beam + $level*100 +30+2]
        set nodeI [expr $level*100+3]
        set nodeJ [expr $level*100 + 35]
        element elasticBeamColumn   $elemID      $nodeI      $nodeJ      $Arigid $Erigid
$IrigidZ      $transfTag_B;
        #rigidLink $type $nodeI $nodeJ

        set elemID [expr $N0beam + $level*100 +30+1]
        set nodeI [expr $level*100+3]
        set nodeJ [expr $level*100 + 36]
        element elasticBeamColumn   $elemID      $nodeI      $nodeJ      $Arigid $Erigid
$IrigidZ      $transfTag_B;
        #rigidLink $type $nodeI $nodeJ

        #pier4
        set elemID [expr $N0beam + $level*100 +40+2]
        set nodeI [expr $level*100+4]
        set nodeJ [expr $level*100 + 44]
        element elasticBeamColumn   $elemID      $nodeI      $nodeJ      $Arigid $Erigid
$IrigidZ      $transfTag_B;
        #rigidLink $type $nodeI $nodeJ
}

```

#define Braces

```

# define braces:
#-----
# eleID convention: "3acd", 3 = brace, a = location of the brace
# "a" convention: 1 = left brace; 2 = right brace;
# cd = element numbering for the brace (can be between 1 and 99)
# command arguments:      $eleID $iNode $jNode $numIntgrPts $secTag $transfTag
set NIP 3
set bay 1
set shift 3000
for {set level 1} {$level <=$NStory} {incr level 1} {

    if { $level==1 } {
        set secTagBr $secTagBr1
    }
    if { $level==2 } {
        set secTagBr $secTagBr2
    }
    if { $level==3 } {
        set secTagBr $secTagBr3
    }
    if { $level==4 } {
        set secTagBr $secTagBr4
    }
    if { $level==5 } {
        set secTagBr $secTagBr5
    }
    if { $level==6 } {
        set secTagBr $secTagBr6
    }
    if { $level==7 } {

```

```

        set secTagBr $secTagBr7
    }
    if { $level==8 } {
        set secTagBr $secTagBr8
    }

    for { set i 1 } { $i <= [expr $noEle] } { incr i } {
        set nodeID [expr $shift+($level*100)+$i]
        set Inode [expr ($level*1000)+($bay*100)+$i]
        set Jnode [expr ($level*1000)+($bay*100)+$i+1]
        element nonlinearBeamColumn $nodeID $Inode $Jnode $NIP $secTagBr
$transfTag_Brace -iter $maxIter $tol
    }
}

set bay 3
set shift 4000
for {set level 1} {$level <=$NStory} {incr level 1} {

    if { $level==1 } {
        set secTagBr $secTagBr1
    }
    if { $level==2 } {
        set secTagBr $secTagBr2
    }
    if { $level==3 } {
        set secTagBr $secTagBr3
    }
    if { $level==4 } {
        set secTagBr $secTagBr4
    }
    if { $level==5 } {
        set secTagBr $secTagBr5
    }
    if { $level==6 } {
        set secTagBr $secTagBr6
    }
    if { $level==7 } {
        set secTagBr $secTagBr7
    }
    if { $level==8 } {
        set secTagBr $secTagBr8
    }

    for { set i 1 } { $i <= [expr $noEle] } { incr i } {
        set nodeID [expr $shift+($level*100)+$i]
        set Inode [expr ($level*1000)+($bay*100)+$i]
        set Jnode [expr ($level*1000)+($bay*100)+$i+1]
        element nonlinearBeamColumn $nodeID $Inode $Jnode $NIP $secTagBr
$transfTag_Brace -iter $maxIter $tol
    }
}

#Define Rigid Links in braces
#bay 1
#go from level_pier_1 to level_pier_8, level+1_pier_9 to level+1_pier_2
#numbered 3_level_2_1 and 3_level_2_2

    set bay 1
    set shift 3000
    set Arigid [expr 4160.*10.]; # area of the brace multiplied by 10 #T67 =854, T103 =
674, T54 =661 , T32= 901
    set Irigid [expr 368000.*10]; # I of the brace multiplied by 10, T103 180000, T32
898000, T54 362000, T67 368000

    for {set level 1} {$level <=$NStory} {incr level 1} {
        set elementID [expr $shift+($level*100)+21]
        set Inode [expr ($level*100)+1]
        set Jnode [expr ($level*100)+18]
        element elasticBeamColumn $elementID $Inode $Jnode $Arigid $Erigid
$Irigid $transfTag_Brace
        #rigidLink $type $nodeI $nodeJ

        set elementID [expr $shift+($level*100)+22]
        set Inode [expr (($level+1)*100)+2]

```

```

        set Jnode [expr (($level+1)*100)+20+9]
        element elasticBeamColumn $elementID $Inode $Jnode $Arigid $Erigid
$Irigid $transfTag_Brace
        #rigidLink $type $nodeI $nodeJ
    }

#bay 3
#go from level_pier_1 to level_pier_8, level+1_pier_9 to level+1_pier_2
#numbered 4_level_2_1 and 4_level_2_2

    set bay 1
    set shift 4000
    set Arigid [expr 10060.*10.]; # area of the brace multiplied by 10 #T67 =854, T103 =
674, T54 =661 , T32= 901
    set Irigid [expr 368000.*10]; # I of the brace multiplied by 10, T103 180000, T32
898000, T54 362000, T67 368000

    for {set level 1} {$level <= $NStory} {incr level 1} {
        set elementID [expr $shift+($level*100)+21]
        set Inode [expr ($level*100)+4]
        set Jnode [expr ($level*100)+40+8]
        element elasticBeamColumn $elementID $Inode $Jnode $Arigid $Erigid
$Irigid $transfTag_Brace
        #rigidLink $type $nodeI $nodeJ

        set elementID [expr $shift+($level*100)+22]
        set Inode [expr (($level+1)*100)+3]
        set Jnode [expr (($level+1)*100)+30+9]
        element elasticBeamColumn $elementID $Inode $Jnode $Arigid $Erigid
$Irigid $transfTag_Brace
        #rigidLink $type $nodeI $nodeJ
    }

#Define Gusset Plate Springs

    for {set level 1} {$level <= $NStory} {incr level 1} {
        set element [expr 3000+($level*100)+31]
        set Inode [expr ($level*100)+18]
        set Jnode [expr (($level)*1000)+101]
        element zeroLength $element $Inode $Jnode -mat
$matID_gusset -dir 6 ;
        set element [expr 3000+((($level+1)*100)+32]
        set Inode [expr (($level+1)*100)+29]
        set Jnode [expr (($level)*1000)+(101+$noEle)]
        element zeroLength $element $Inode $Jnode -mat
$matID_gusset -dir 6 ;
    }

    for {set level 1} {$level <= $NStory} {incr level 1} {
        set element [expr 4000+($level*100)+31]
        set Inode [expr ($level*100)+48]
        set Jnode [expr (($level)*1000)+301]
        element zeroLength $element $Inode $Jnode -mat
$matID_gusset -dir 6 ;

        set element [expr 4000+((($level+1)*100)+32]
        set Inode [expr (($level+1)*100)+39]
        set Jnode [expr (($level)*1000)+(301+$noEle)]
        element zeroLength $element $Inode $Jnode -mat
$matID_gusset -dir 6 ;
    }

#Define Shear Tab Connections

    set Epin 3420489224; #Calculated Based on Liu & Astaeh(2004)

    uniaxialMaterial Elastic $matID_sheartab $Epin

    for {set level 2} {$level <= $NStory} {incr level 1} {
        set element [expr ($level*100)+23]
        set Inode [expr ($level*100)+26]
        set Jnode [expr ($level*100)+27]
        element zeroLength $element $Inode $Jnode -mat $matID_sheartab -dir

```

```

        set element [expr ($level*100)+33]
        set Inode [expr ($level*100)+34]
        set Jnode [expr ($level*100)+35]
        element zeroLength $element $Inode $Jnode -mat $matID_sheartab -dir
6    }

    # add rigid truss elemnts to assure that the beam nodes move horizontally together
    # Represents floor diaphragm
    set ArigidT 1000000.0; # define area of truss section (make much larger than A of
frame elements)
    # command: element truss $eleID $iNode $jNode $A $materialID

    for {set level 2} {$level <=[expr $NStory+1]} {incr level 1} {
        for {set bay 1} {$bay <= $NBay} {incr bay 1} {
            set elemID [expr 5000+ $level*100 +$bay*10+1]
            set nodeI [expr $level*100 + $bay]
            set nodeJ [expr $level*100 + ($bay+1)]
            element truss $elemID $nodeI $nodeJ $ArigidT $matID_RT; # beams
            #equalDOF $nodeI $nodeJ 1
        }
    }

# define p-delta columns, rigid links and zero-stiffness springs
#define extra nodes
node 105 $Pier5 $Floor1
node 205 $Pier5 $Floor2
node 305 $Pier5 $Floor3
node 405 $Pier5 $Floor4
node 505 $Pier5 $Floor5
node 605 $Pier5 $Floor6

node 251 $Pier5 $Floor2 ; #Aritficial Below 206
node 252 $Pier5 $Floor2 ; #Aritficial Above 206
node 351 $Pier5 $Floor3 ; #Aritficial Below 306
node 352 $Pier5 $Floor3 ; #Aritficial Above 306
node 451 $Pier5 $Floor4 ; #Aritficial Below 406
node 452 $Pier5 $Floor4 ; #Aritficial Above 406
node 551 $Pier5 $Floor5 ; #Aritficial Below 506
node 552 $Pier5 $Floor5 ; #Aritficial Above 506
node 651 $Pier5 $Floor6 ; #Aritficial Below 606

set TrussMatID 800; # define a material ID
set Arigid [expr $acoll*1000]; # define area of truss section (make much larger
than A of frame elements)
set IrigidZ [expr $Icoll*1000];
set IrigidY [expr $Iycll*1000];
set Jrigid [expr $Jcoll*1000];

uniaxialMaterial Elastic $TrussMatID $Es; # define truss material
# rigid links
# command: element truss $eleID $iNode $jNode $A $materialID
# eleID convention: 8xy0, 8 = rigid horizontal link, x = floor #, y = bay #
element truss 8250 204 205 $Arigid $TrussMatID; # Floor 2
element truss 8350 304 305 $Arigid $TrussMatID; # Floor 3
element truss 8450 404 405 $Arigid $TrussMatID; # Floor 4
element truss 8550 504 505 $Arigid $TrussMatID; # Floor 5
element truss 8650 604 605 $Arigid $TrussMatID; # Floor 6

# p-delta columns
# eleID convention: 9xy0, 9 = p-delta columns, x = Storey #, y = pier #
#elasticBeamColumn beamId iNode jNode A E G Jx Iy Iz
transTag
element elasticBeamColumn 9160 105 251 $Arigid $Es $IrigidZ $transfTag_C; #
Story 1
element elasticBeamColumn 9260 252 351 $Arigid $Es $IrigidZ $transfTag_C; #
Story 2
element elasticBeamColumn 9360 352 451 $Arigid $Es $IrigidZ $transfTag_C; #
Story 3
element elasticBeamColumn 9460 452 551 $Arigid $Es $IrigidZ $transfTag_C; #
Story 4

```

```

        element elasticBeamColumn 9560 552 651 $Arigid $Es $IrigidZ $transfTag_C; #
Story 5

# p-delta EDof
equalDOF 205 251 1 2
equalDOF 205 252 1 2
equalDOF 305 351 1 2
equalDOF 305 352 1 2
equalDOF 405 451 1 2
equalDOF 405 452 1 2
equalDOF 505 551 1 2
equalDOF 505 552 1 2
equalDOF 605 651 1 2

fix 105 1 1 0

# p-delta Springs
# ID 9-floor-01 below, ID 9-floor-02 below
set k_free 100
set matID_pin 22
uniaxialMaterial Elastic $matID_pin $k_free
element zeroLength 9101 251 205 -mat $matID_pin -dir 6
element zeroLength 9102 252 205 -mat $matID_pin -dir 6
element zeroLength 9201 351 305 -mat $matID_pin -dir 6
element zeroLength 9202 352 305 -mat $matID_pin -dir 6
element zeroLength 9301 451 405 -mat $matID_pin -dir 6
element zeroLength 9302 452 405 -mat $matID_pin -dir 6
element zeroLength 9401 551 505 -mat $matID_pin -dir 6
element zeroLength 9402 552 505 -mat $matID_pin -dir 6
element zeroLength 9501 651 605 -mat $matID_pin -dir 6

puts "model built"

#####
# Display the model
#####

#recorder display "Model 3d View" 100 100 500 500 -wipe
#prp 0 0 50
#vup 0 1 0
#vpn -2 -1 1
#display 1 2 10

#recorder display "Model Front View" 10 10 500 500 -wipe
#prp 0 0 50
#vup 0 1 0
#vpn 0 0 1
#display 1 2 2

#####
# Define GRAVITY LOADS
#####
if {$analysisType == "pushover" || $analysisType == "dynamic"} {

set UDL 0.0 ;#n/mm, same as KN/m

# define GRAVITY -----
# GRAVITY LOADS # define gravity load applied to beams and columns -- eleLoad applies
loads in local coordinate axis
pattern Plain 101 Linear {

    for {set level 2} {$level <= [expr $NStory+1]} {incr level 1} {
        for {set bay 1} {$bay <= $NBay} {incr bay 1} {
            set elemID [expr 2000 + $level*100 + $bay*10]
            eleLoad -ele $elemID -type -beamUniform -$UDL 0 0; # BEAMS
        }
    }
}

set floorLoad [expr $testmass*-9810]
set roofLoad [expr $roofmass*-9810]

```

```

set edgeColLoad [expr $floorLoad*0.25]
set centreColLoad [expr $floorLoad*0.5]
set PDeltaColLoad [expr $floorLoad*3]

set RoofedgeColLoad [expr $roofLoad*0.25]
set RoofcentreColLoad [expr $roofLoad*0.5]
set RoofPDeltaColLoad [expr $roofLoad*3]

pattern Plain 102 Constant {

    # point loads on leaning column nodes
    # command: load node Fx Fy Mz
    load 201 0.0 $edgeColLoad 0.0
    load 301 0.0 $edgeColLoad 0.0
    load 401 0.0 $edgeColLoad 0.0
    load 501 0.0 $edgeColLoad 0.0
    load 601 0.0 $RoofedgeColLoad 0.0

    load 202 0.0 $centreColLoad 0.0
    load 302 0.0 $centreColLoad 0.0
    load 402 0.0 $centreColLoad 0.0
    load 502 0.0 $centreColLoad 0.0
    load 602 0.0 $RoofcentreColLoad 0.0

    load 203 0.0 $centreColLoad 0.0
    load 303 0.0 $centreColLoad 0.0
    load 403 0.0 $centreColLoad 0.0
    load 503 0.0 $centreColLoad 0.0
    load 603 0.0 $RoofcentreColLoad 0.0

    load 204 0.0 $edgeColLoad 0.0
    load 304 0.0 $edgeColLoad 0.0
    load 404 0.0 $edgeColLoad 0.0
    load 504 0.0 $edgeColLoad 0.0
    load 604 0.0 $RoofedgeColLoad 0.0

    load 205 0.0 $PDeltaColLoad 0.0
    load 305 0.0 $PDeltaColLoad 0.0
    load 405 0.0 $PDeltaColLoad 0.0
    load 505 0.0 $PDeltaColLoad 0.0
    load 605 0.0 $RoofPDeltaColLoad 0.0

}

# Gravity-analysis: load-controlled static analysis
set Tol 1.0e-6; # convergence tolerance for test
constraints Plain; # how it handles boundary conditions
#constraints Penalty 1.0e15 1.0e15;
numberer RCM; # renumber dof's to minimize band-width
(optimize)
system BandGeneral; # how to store and solve the system of
equations in the analysis (large model: try UmfPack)
test NormDispIncr $Tol 10; # determine if convergence has been achieved
at the end of an iteration step
algorithm Newton ; # use Newton's solution algorithm: updates
tangent stiffness at every iteration
set NstepGravity 10; # apply gravity in 10 steps
set DGravity [expr 1.0/$NstepGravity]; # load increment
integrator LoadControl $DGravity; # determine the next time step for an analysis
analysis Static; # define type of analysis static or transient
analyze $NstepGravity; # apply gravity

# maintain constant gravity loads and reset time to zero
loadConst -time 0.0
puts "Model Built"
}

#####
# Prelim Eigenvalue Analysis
#####
if {$analysisType == "pushover" || $analysisType == "dynamic"} {
#number of modes
set numModes 3

# perform eigen analysis

```

```

#-----
set lambda [eigen fullGenLapack $numModes];

# calculate frequencies and periods of the structure
#-----

set pi 3.141593

# calculate frequencies and periods of the structure
#-----
set omega {}
set f {}
set T {}
set pi 3.141593

foreach lam $lambda {
    lappend omega [expr sqrt($lam)]
    lappend f [expr sqrt($lam)/(2*$pi)]
    lappend T [expr (2*$pi)/sqrt($lam)]
}
puts "periods are $T"
set T1 [lindex $T 0];
set T2 [lindex $T 1];
set T3 [lindex $T 2];
puts "T1 is $T1"
puts "T2 is $T2"
set g 9810;
set pi 3.141593
}

#####
#           Static Lateral Analysis           #
#####
if {$analysisType == "static_lateral"} {
    set m_total [expr $testmass*4.5]
    set m_roof_total [expr $roofmass*4.5]

    puts "M Floor = $m_total"
    puts "M_Roof = $m_roof_total"

    set denominator [expr
$ m_roof_total*5*$HStory+$m_total*4*$HStory+$m_total*3*$HStory+$m_total*2*$HStory+$m_total*
1*$HStory]
    set ModelBaseShear 1528193
    pattern Plain 200 Linear {
        load 201 [expr ($ModelBaseShear*$m_total*1*$HStory)/$denominator] 0.0 0.0;
        load 301 [expr ($ModelBaseShear*$m_total*2*$HStory)/$denominator] 0.0 0.0;
        load 401 [expr ($ModelBaseShear*$m_total*3*$HStory)/$denominator] 0.0 0.0;
        load 501 [expr ($ModelBaseShear*$m_total*4*$HStory)/$denominator] 0.0 0.0;
        load 601 [expr ($ModelBaseShear*$m_roof_total*5*$HStory)/$denominator] 0.0
0.0 ;
    }

    puts "Level 2 Load = [expr ($ModelBaseShear*$m_total*1*$HStory)/$denominator]"
    puts "Level 3 Load = [expr ($ModelBaseShear*$m_total*2*$HStory)/$denominator]"
    puts "Level 4 Load = [expr ($ModelBaseShear*$m_total*3*$HStory)/$denominator]"
    puts "Level 5 Load = [expr ($ModelBaseShear*$m_total*4*$HStory)/$denominator]"
    puts "Level 6 Load = [expr
($ModelBaseShear*$m_roof_total*5*$HStory)/$denominator]"

    # recorde disp at node 3 and reactions at the bottom nodes
    recorder Node -file "node3D.out" -time -node 201 301 401 501 601 -dof 1 disp
    recorder Node -file "reactions_all.out" -time -node 101 102 103 104 -dof 1
reaction
    recorder Node -file "reactions.out" -time -node 101 -dof 1 reaction

    set Tol 1.0e-6;                                # convergence tolerance for test
    #constraints Plain;                             # how it handles boundary conditions
    constraints Penalty 1.0e15 1.0e15;
    numberer RCM;                                    # renumber dof's to minimize band-width
(optimization)
    system BandGeneral;                              # how to store and solve the system of
equations in the analysis (large model: try UmfPack)
    test NormDispIncr $Tol 10;                      # determine if convergence has been
achieved at the end of an iteration step

```



```

        algorithm Newton ;                                # use Newton's solution algorithm: updates
tangent stiffness at every iteration
        set NstepLateral 10;                             # apply gravity in 10 steps
        set DLTateral [expr 1.0/$NstepLateral];          # load increment
        integrator LoadControl $DLTateral;               # determine the next time step for an
analysis
        analysis Static;                                  # define type of analysis static or
transient
        analyze $NstepLateral;                            # apply gravity

        puts "Level1disp: [nodeDisp 201 1]"
        puts "Level2disp: [nodeDisp 301 1]"
        puts "Level3disp: [nodeDisp 401 1]"
        puts "Level4disp: [nodeDisp 501 1]"
        puts "Level5disp: [nodeDisp 601 1]"
        puts "[nodeDisp 201 1] [nodeDisp 301 1] [nodeDisp 401 1] [nodeDisp 501 1]
[nodeDisp 601 1]"
    }

#####
#               Pushover Analysis                           #
#####
    if {$analysisType == "pushover"} {
        puts "Carrying out Eigenvalue Analysis"
        #number of modes
        set numModes 2
        set lambda [eigen fullGenLapack $numModes];

        # calculate frequencies and periods of the structure
        #-----
        set omega {}
        set f {}
        set T {}
        set pi 3.141593

        foreach lam $lambda {
            lappend omega [expr sqrt($lam)]
            lappend f [expr sqrt($lam)/(2*$pi)]
            lappend T [expr (2*$pi)/sqrt($lam)]
            puts $T
        }
        puts "periods are $T"

        #Run a one step gravity load with no loading to record eigne vectors
        #-----
        integrator LoadControl 0 1 0 0
        test EnergyIncr 1.0e-10 100 0
        algorithm Newton
        numberer RCM
        constraints Transformation
        system ProfileSPD

        analysis Static
        set ok [analyze 1]
        if {$ok != 0} {
            puts "Modal Analysis Failed"
        }

        #get eigenvectors
        #-----
        set f11 [nodeEigenvector 201 1 1]
        set f21 [nodeEigenvector 301 1 1]
        set f31 [nodeEigenvector 401 1 1]
        set f41 [nodeEigenvector 501 1 1]
        set f51 [nodeEigenvector 601 1 1]

        puts "eigenvector 1: [list [expr {$f11/$f51}] [expr {$f21/$f51}] [expr
{$f31/$f51}] [expr {$f41/$f51}] [expr {$f51/$f51}]]"

        puts "Running Pushover..."
        # assign lateral loads and create load pattern: use ASCE 7-10 distribution
        set ampFactor 100000000
        pattern Plain 200 Linear {
            load 201 [expr $ampFactor*$f11/$f51] 0.0 0.0;
            load 301 [expr $ampFactor*$f21/$f51] 0.0 0.0;
            load 401 [expr $ampFactor*$f31/$f51] 0.0 0.0;

```

```

        load 501 [expr $ampFactor*$f41/$f51] 0.0 0.0;
        load 601 [expr $ampFactor*$f51/$f51] 0.0 0.0 ;
        load 202 [expr $ampFactor*$f11/$f51] 0.0 0.0;
        load 302 [expr $ampFactor*$f21/$f51] 0.0 0.0;
        load 402 [expr $ampFactor*$f31/$f51] 0.0 0.0;
        load 502 [expr $ampFactor*$f41/$f51] 0.0 0.0;
        load 602 [expr $ampFactor*$f51/$f51] 0.0 0.0 ;
        load 203 [expr $ampFactor*$f11/$f51] 0.0 0.0;
        load 303 [expr $ampFactor*$f21/$f51] 0.0 0.0;
        load 403 [expr $ampFactor*$f31/$f51] 0.0 0.0;
        load 503 [expr $ampFactor*$f41/$f51] 0.0 0.0;
        load 603 [expr $ampFactor*$f51/$f51] 0.0 0.0 ;
        load 204 [expr $ampFactor*$f11/$f51] 0.0 0.0;
        load 304 [expr $ampFactor*$f21/$f51] 0.0 0.0;
        load 404 [expr $ampFactor*$f31/$f51] 0.0 0.0;
        load 504 [expr $ampFactor*$f41/$f51] 0.0 0.0;
        load 604 [expr $ampFactor*$f51/$f51] 0.0 0.0 ;
    }

    # recorde disp at node 3 and reactions at the bottom nodes
    recorder Node -file "node3D.out" -time -node 201 301 401 501 601 -dof 1 disp
    recorder Node -file "reactions_all.out" -time -node 101 102 103 104 -dof 1
reaction
    recorder Node -file "reactions.out" -time -node 101 -dof 1 reaction

    # displacement parameters
    set IDctrlNode 601; # node where disp is read for disp control
    set IDctrlDOF 1; # degree of freedom read for disp control
(1 = x displacement)
    set Dmax [expr 0.04*$HStory]; # maximum displacement of pushover
    set Dincr [expr 0.05]; # was dmax/1000 displacement increment

    # pushover analysis commands
    constraints Transformation;
    numberer RCM;
    system BandGeneral;
    #test EnergyIncr 1.0e-5 10
    test NormUnbalance 1.0e-5 10;
    algorithm NewtonLineSearch;
    integrator DisplacementControl $IDctrlNode $IDctrlDOF $Dincr;
    analysis Static;
    set ok 0
    set currentDisp 0.0
    while {$ok == 0 && $currentDisp < $Dmax} {
        set ok [analyze 1]
        if {$ok != 0} {
            test NormDispIncr 1.0e-3 1000 1
            algorithm Newton -initial
            set ok [analyze 1]
            test NormDispIncr 1.0e-6 10
            algorithm Newton
        }
        set currentDisp [nodeDisp $IDctrlNode 1]
    }

    puts "node501disp: [nodeDisp 601 1], oppositeenddisp: [nodeDisp 604 1]"
    puts "TargetDisp: $Dmax"
}

#####
# Transient Analysis #
#####
if {$analysisType == "dynamic"} {
    puts "Running Transient analysis..."

    # Create a recorder to monitor nodal displacements
    recorder Node -file SCBFdispT32.out -time -node 201 301 401 501 601 -dof 1 disp
    recorder Node -file Flooraccel.out -time -node 201 301 401 501 601 -dof 1 accel
    recorder Element -file "Damage3.out" -time -ele 3501 4501 section 7 fiber 15
damage
    source Dynamic_Oak_5by3baymk.EQ.tcl
}

#####

```

```

#                               Eigenvalue Analysis
#####

if {$analysisType == "eigen"} {
    puts "Carrying out Eigenvalue Analysis"
    #number of modes
    set numModes 3

    #set up recorders
    # record drift histories
        # drift recorder command: recorder Drift -file $filename -iNode $NodeI_ID -
jNode $NodeJ_ID -dof $dof -perpDirn $Record.drift.perpendicular.to.this.direction
        #recorder Node -file "mode1.out" -node 201 -dof 1 2 "eigen1";
        #recorder Node -file "mode2.out" -node 201 -dof 1 2 "eigen2";

    # perform eigen analysis
    #-----
    set lambda [eigen fullGenLapack $numModes];

    # calculate frequencies and periods of the structure
    #-----
    set omega {}
    set f {}
    set T {}
    set pi 3.141593

    foreach lam $lambda {
        lappend omega [expr sqrt($lam)]
        lappend f [expr sqrt($lam)/(2*$pi)]
        lappend T [expr (2*$pi)/sqrt($lam)]
        puts $T
    }
    puts "periods are $T"

    #write the output file consisting of periods
    #-----
    #set period "modes/Periods.txt"
    #set Periods [open $period "w"]
    #foreach t $T{
    #    puts $Periods "t"
    #}
    #close $Periods

    #Run a one step gravity load with no loading to record eigne vectors
    #-----
    integrator LoadControl 0 1 0 0
    test EnergyIncr 1.0e-10 100 0
    algorithm Newton
    numberer RCM
    constraints Transformation
    system ProfileSPD

    analysis Static
    set ok [analyze 1]
    if {$ok != 0} {
        puts "Modal Analysis Failed"
    }

    #get eigenvectors
    #-----
    set f11 [nodeEigenvector 201 1 1]
    set f21 [nodeEigenvector 301 1 1]
    set f31 [nodeEigenvector 401 1 1]
    set f41 [nodeEigenvector 501 1 1]
    set f51 [nodeEigenvector 601 1 1]

    set f12 [nodeEigenvector 201 2 1]
    set f22 [nodeEigenvector 301 2 1]
    set f32 [nodeEigenvector 401 2 1]
    set f42 [nodeEigenvector 501 2 1]
    set f52 [nodeEigenvector 601 2 1]

    puts [nodeEigenvector 601 2 1]

    set f13 [nodeEigenvector 201 3 1]
    set f23 [nodeEigenvector 301 3 1]

```

```

set f33 [nodeEigenvector 401 3 1]
set f43 [nodeEigenvector 501 3 1]
set f53 [nodeEigenvector 601 3 1]

puts "eigenvector 1: [list [expr {$f11/$f51}] [expr {$f21/$f51}] [expr {$f31/$f51}] [expr
{$f41/$f51}] [expr {$f51/$f51}]]"
puts "eigenvector 2: [list [expr {$f12}] [expr {$f22}] [expr {$f32}] [expr {$f42}] [expr
{$f52}]]"
puts "eigenvector 2: [list [expr {$f13}] [expr {$f23}] [expr {$f33}] [expr {$f43}] [expr
{$f53}]]"

#set T1 [expr $T];
set g 9810;
set pi 3.141593

# #-----
recorder display "Mode 1" 10 10 500 500 -wipe
prp 0 0 50
vup 0 1 0
vpn 0 0 1
display -1 2 50

#prp 0 0 50
#vup 0 1 0
#vpn 0 0 1
#display 2 2 1
}

```

D.1 5 Storey CBF; $q = 4$; Oakland Site

```

# -----
# Dynamic Earthquake Analysis
# Created by: Vesna Terzic, UC Berkeley, 2011
# Edited by: John Hickey TCD
# execute this file after you have built the model, and after you apply gravity
# -----
set pi 3.141593
format
# puts "in dynamic"
# Uniform Earthquake ground motion (uniform acceleration input at all support nodes)
set GMdirection 1; # ground-motion direction
####set GMfileH "SanFernando2" ; # ground-motion filenames:
horizontal

#####
# Define & Apply Damping
#####
# RAYLEIGH damping parameters
#  $C = \alpha M + \beta K_{curr} K_{current} + \beta K_{comm} K_{lastCommit} + \beta K_{init} K_{initial}$ 

#set xDamp 0.05; # damping ratio
set omeg1 [expr (2.*$pi)/$T1]
set omeg2 [expr (2.*$pi)/$T2]

# puts "w1 = $omeg1"
# puts "w2 = $omeg2"

puts $xDamp
#set betaKcomm [expr 2.*$xDamp*$T1/(2.*$pi)]; #see 'Unintended Consequences of
Modeling Damping in Structures' by Charney (2008), equation 6, for derivation
#puts "a1 no a0 = $betaKcomm"

set a1 [expr (2*($xDamp-$xDamp*($omeg1/$omeg2))/($omeg2-
($omeg1*$omeg1/$omeg2)))]
set a0 [expr (($xDamp-$omeg1*$a1*0.5)/(1/(2*$omeg1)))]
#puts "a0 = $a0"
#puts "a1 = $a1"
set betaKcomm $a1;
#assign tangent stiffness proportional damping to columns, beams, braces and gusset
plate of a braced frame
set lastColumn [expr 1000+((NStory)*100)+5]

```

```

region 1 -eleRange 1110 $lastColumn -rayleigh 0 0. 0. $betaKcomm; #columns
set lastBeam [expr 2000+((NStory+1)*100)+4]
region 2 -eleRange 2210 $lastBeam -rayleigh 0 0. 0. $betaKcomm; #beams

set lastBrace [expr 4000+(NStory*100)+$noEle]
region 3 -eleRange 3101 $lastBrace -rayleigh 0 0. 0. $betaKcomm; #braces

#assign mass proportional damping
region 4 -node 201 202 203 204 301 302 303 304 401 402 403 404 501 502 503 504 601 602
603 604 -rayleigh $a0 0.0 0.0 0.0;

# if { $GP_Model == "GP" } {
#     region 4 -eleRange 411 422 -rayleigh 0. 0. 0. $betaKcomm; #gusset plates
# }

#####
#####
# Perform Dynamic Ground-Motion Analysis for Horizontal and Vertical Component of Ground
Motion
#####
#####

# Uniform EXCITATION: acceleration input
set IDloadTag 400; # for uniform support excitation
set inFileH $GMfileH.at2
#set inFileV $GMfileV.acc
set outFileH $GMfileH
#set outFileV $GMfileV
ground-motion file
#puts "out of ReadSMD"
set g 9810;
set dt $time_step
set GMfatt [expr $g*$GMfact]; # data in input file is in g Units --
ACCELERATION TH
puts "Horizontal File: $outFileH"
puts "Vertical File: $outFileV"

timeSeries Path 10 -dt $dt -filePath $outFileH -factor $GMfatt; # horizontal
time series information
timeSeries Path 11 -dt $dt -filePath $outFileV -factor $GMfatt; # vertical
time series information
pattern UniformExcitation $IDloadTag $GMDirection -accel 10; # create
Uniform excitation for horizontal GM
pattern UniformExcitation [expr $IDloadTag+1] 2 -accel 11; # create
Uniform excitation for vertical GM

recorder Node -file AbsFlooraccel.out -timeSeries 10 -time -node 201 301 401 501 601 -
dof 1 accel
# define analysis objects and performe analysis
#set nPt
#set tFinal [expr $dt*$nPt]; # maximum duration of ground-motion analysis
constraints Transformation
numberer RCM
system BandGeneral
test NormDispIncr 1.0e-6 100
algorithm KrylovNewton; #Newton
integrator Newmark 0.5 0.25
analysis Transient

#new version
#constraints Transformation
#numberer RCM
#system BandGeneral; #may work better with UmfPack
#test EnergyIncr 1.0e-6 100 0
#algorithm ModifiedNewton
#integrator Newmark 0.5 0.25
#analysis Transient

set DtAnalysis_problem [expr $deltaT/100]
set ok 0.0
set currentTime 0.0
while {$ok == 0 && $currentTime < $tFinal} {
    #set TIME_start [clock clicks -milliseconds]
    set ok [analyze 1 $deltaT]
    #set TIME_taken [expr [clock clicks -milliseconds] - $TIME_start]

    if {$ok != 0 } {

```

```

    puts "Trying Krylov Newton with tolerance 10e-4..."
    test NormDispIncr 1e-4 2000
    algorithm KrylovNewton
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {
            puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
            puts "Running Transient Analysis... "
        }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0} {
    puts "Trying Krylov Newton with tolerance 10e-3..."
    test NormDispIncr 1e-3 2000
    algorithm KrylovNewton
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {
            puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
            puts "Running Transient Analysis... "
        }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0} {
    puts "Trying Krylov Newton with tolerance 10e-2..."
    test NormDispIncr 1e-2 2000
    algorithm KrylovNewton
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {
            puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
            puts "Running Transient Analysis... "
        }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0} {
    puts "Trying Newton..."
    test NormDispIncr 1e-6 2000
    algorithm Newton
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {
            puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
            puts "Running Transient Analysis... "
        }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0} {
    puts "Trying Newton..."
    test NormDispIncr 1e-3 2000
    algorithm Newton
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {
            puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
            puts "Running Transient Analysis... "
        }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0} {
    puts "Trying Newton initial with relaxed tolerance..."
    test NormDispIncr 1e-3 2000
    algorithm Newton -initial
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {
            puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
            puts "Running Transient Analysis... "
        }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0} {
    puts "Trying Newton initial with relaxed tolerance..."
    test NormDispIncr 1.0e-3 200 1
    algorithm NewtonLineSearch 0.8
    set ok [analyze 1 $deltaT]
        if {$ok == 0} {

```

```

        puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
        puts "Running Transient Analysis... "
    }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0 } {
    puts "Trying Newton initial then current with relaxed tolerance..."
    test NormDispIncr 1.0e-3 2000
    algorithm Newton -initialThenCurrent
    set ok [analyze 1 $deltaT]
    if {$ok == 0} {
        puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
        puts "Running Transient Analysis... "
    }
    test NormDispIncr 1.0e-6 100
    algorithm KrylovNewton
}
if {$ok != 0 } {
    puts "Trying Broyden .."
    algorithm Broyden 8
    set ok [analyze 100 $deltaT]
    if {$ok == 0} {
        puts "Worked - Back to KrylovNewton with tolerance 10e-6 "
        puts "Running Transient Analysis... "
    }
    algorithm KrylovNewton
}

set currentTime [getTime]
}

puts "Ground Motion Done. End Time: [getTime]. tFinal: $tFinal

```

Appendix E Connection Design Calculations

Connections:

1. Brace welded to gusset plate (Same connection top & bottom)
2. Upper Gusset plate welded to
 - a. 'Beam Plate'
 - b. 'Column Plate'
3. 'Upper Beam Plate' bolted to beam flange
4. 'Upper Column Plate' bolted to column flange
5. Lower Gusset plate welded to 'Plate'
6. 'Plate' bolted frame support

Brace - Gusset Weld			
Weld Length	l_{total}	65 mm	
Weld Throat Thickness	a	8 mm	
Weld Effective Length	l_{eff}	49 mm	$L_{weld} - 2a$
No. of Welds		4	
Brace Force	E_d	251.55 kN	
Design Weld Force per Unit Length	$F_{W,Ed}$	1.283418 kN/mm	$BraceForce / (l_{eff} \times No. \text{ of Welds})$
Ultimate Material Strength	f_u	360 N/mm ²	
Reduction Factor	β_w	0.8	EN 1993-1-8 Table 1.4
Weld Resistance per Unit Area	$f_{vw,d}$	207.8461 N/mm ²	$f_u / \sqrt{3} / (\beta_w \gamma_{M2})$
Weld Resistance per unit Length	$F_{W,Rd}$	1.662769 kN/mm	$a \times f_{vw,d}$
Total Joint Resistance	$F_{joint, Rd}$	325.9027 kN	$F_{W,Rd} \times l_{eff} \times No. \text{ Welds}$

Upper Gusset - Beam Plate Weld			
Weld Length	l_{total}	191 mm	
Weld Throat Thickness	a	6 mm	
Weld Effective Length	l_{eff}	179 mm	$L_{weld} - 2a$
No. of Welds		2 Welds	
Brace Axial Force	E_d	251.55 kN	
Brace Angle	α	0.945968	Radians
Beam Weld Design Force		204.0231 kN	$\sin \alpha \times \text{Brace Axial Force}$
Design Force per Unit Length	$F_{W,Ed}$	0.569897 kN/mm	$Beam \text{ Weld Force} / (l_{eff} \times No. \text{ of Welds})$
Ultimate Material Strength	f_u	360 N/mm ²	
Reduction Factor	β_w	0.8	EN 1993-1-8 Table 1.4
Weld Resistance per Unit Area	$f_{vw,d}$	207.8461 N/mm ²	$f_u / \sqrt{3} / (\beta_w \gamma_{M2})$
Weld Resistance per unit Length	$F_{W,Rd}$	1.247077 kN/mm	$a \times f_{vw,d}$
Total Joint Resistance	$F_{joint, Rd}$	446.4534 kN	$F_{W,Rd} \times l_{eff} \times No. \text{ Welds}$

Upper Gusset - Column Plate Weld			
Weld Length	l_{total}	208	mm
Weld Throat Thickness	a	6	mm
Weld Effective Length	l_{eff}	196	mm
No. of Welds		2	Welds
Brace Axial Force	E_d	251.55	kN
Brace Angle	α	0.945968	Radians
Column Weld Design Force		147.1461	kN
			$\cos \alpha \times \text{Brace Axial Force}$
Design Force per Unit Length	$F_{W,Ed}$	0.375373	kN/mm
			Column Weld Force/($l_{eff} \times \text{No. of Welds}$)
Ultimate Material Strength	f_u	360	N/mm ²
Reduction Factor	β_w	0.8	EN 1993-1-8 Table 1.4
Weld Resistance per Unit Area	$f_{vw,d}$	207.8461	N/mm ²
			$f_u/\sqrt{3}/(\beta_w \gamma_{M2})$
Weld Resistance per unit Length	$F_{W,Rd}$	1.247077	kN/mm
			$a \times f_{vw,d}$
Total Joint Resistance	$F_{joint,Rd}$	488.854	kN
			$F_{W,Rd} \times l_{eff} \times \text{No. Welds}$

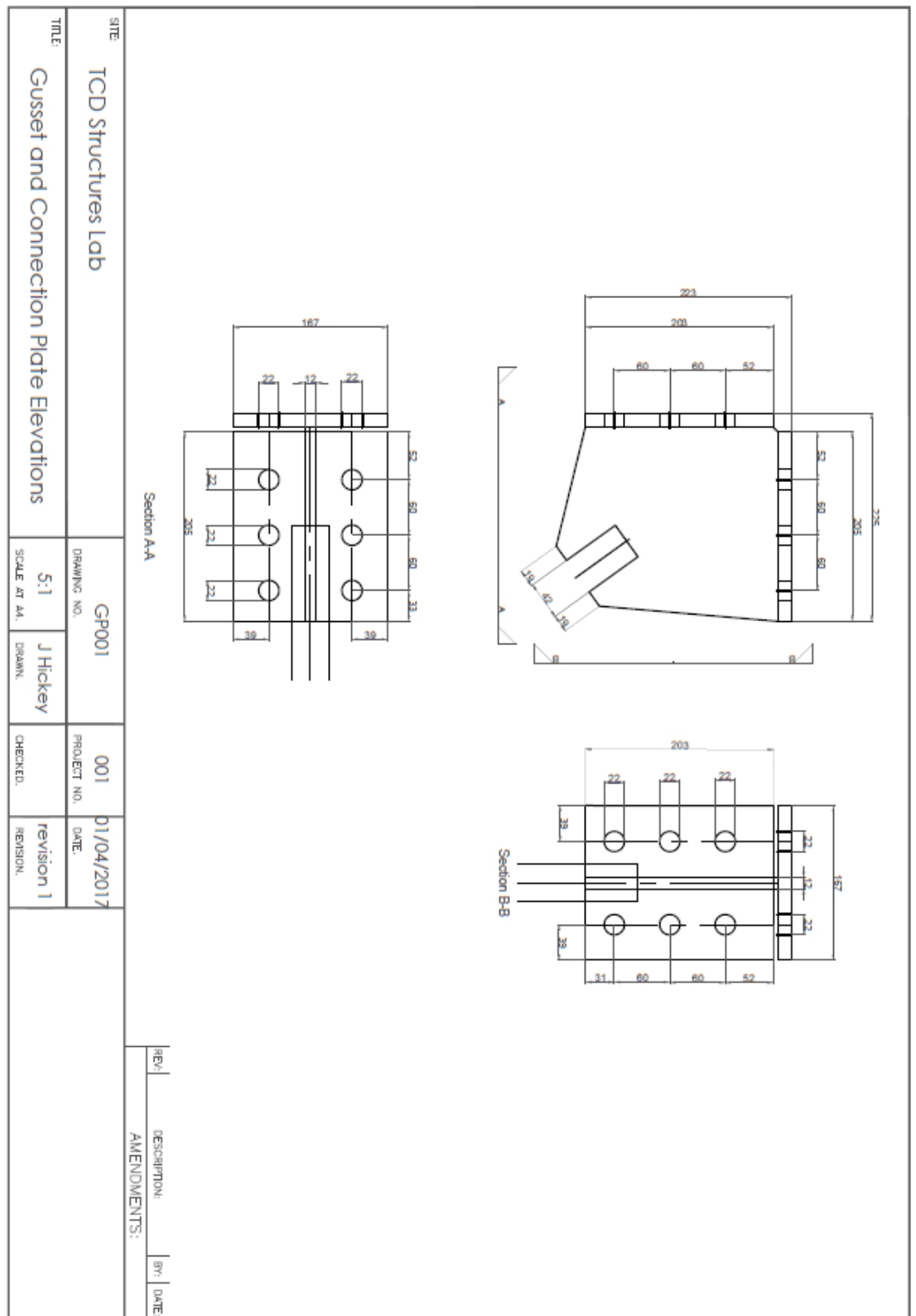
Bolted Connection - Upper Beam Plate to Beam Flange			
Shear			
No. of Bolts		6	
Design Action		204.0231	kN
			$\sin \alpha \times \text{Brace Axial Force}$
Design Action per Bolt	$F_{V,Ed}$	34.00385	kN
			Design Action/ No. of Bolts
Bolt Grade		8.8	
	a_v	0.6	EN 1993-1-8 Table 3.4
Bolt		M20	
Tensile Stress Area	A_s	244.79	mm ²
Bolt Ultimate Tensile Strength	f_{ub}	800	N/mm ²
			EN 1993-1-8 Table 3.1
Shear Resistance per Bolt	$F_{V,Rd}$	93.99936	kN
			$a_v A_s f_{ub}/\gamma_{M2}$
Joint		563.9962	kN
			$F_{V,Rd} \times \text{No. Bolts}$
Bearing - Bolt Plate			
Design Action per Bolt	$F_{V,Ed}$	34.00385	kN
			Design Action/ No. of Bolts
Ultimate Bolt Strength	f_{ub}	800	N/mm ²
Ultimate Plate Strength	f_u	360	N/mm ²
	f_{ub}/f_u	2.222222	
			Minimum EN 1993-1-8 Table 3.3
	e_1	31	mm
			26.4 mm; 1.2 d_0
	p_1	60	mm
			48.4 mm; 2.2 d_0
	e_2	39	mm
			26.4 mm; 1.2 d_0
	p_2	90	mm
			52.8 mm; 2.4 d_0
	d_0	22	mm
	$a_{d,end}$	0.469697	
			$e_1/3d_0$
	$a_{d,inner}$	0.659091	
			$p_1/3d_0 - 0.25$
	$a_{d,critical}$	0.469697	
			Smallest of $a_{d,end}$; $a_{d,inner}$
	$k_{1,end}$	2.5	
			smallest of 2.5; 1.4 $p_2/3d_0 - 1.7$; 2.8 $e_2/3d_0 - 1.7$
	$k_{1,inner}$	2.5	
			smallest of 2.5; 1.4 $p_2/3d_0 - 1.7$
	a_b	0.469697	
			Smallest of $a_{d,i}$; 1; f_{ub}/f_b
Bolt Diameter	d	20	mm
Plate Thickness	t	15	mm
Bearing Resistance	$F_{b,Rd}$	101.4545	kN
			$a_v k_1 f_u dt/\gamma_{M2}$
Joint		608.7273	

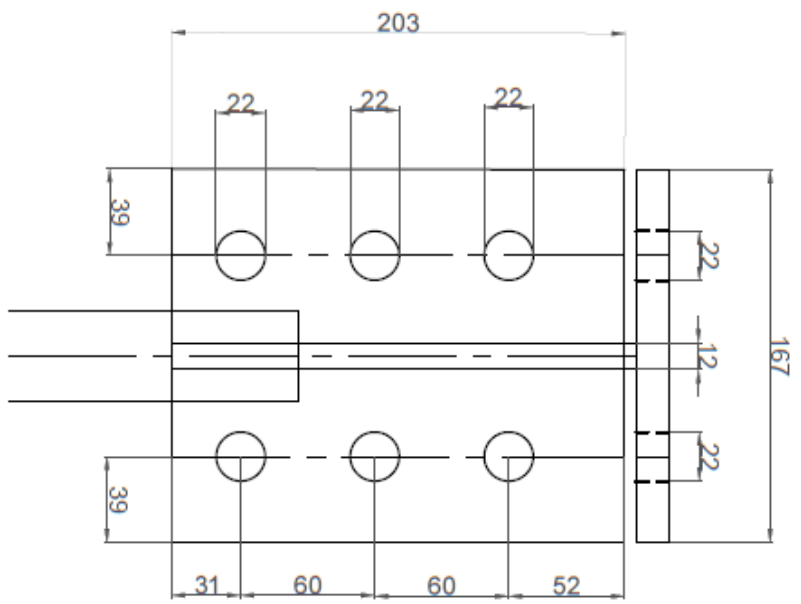
Bolted Connection - Upper Column Plate to Column Flange				
Shear				
No. of Bolts		6		
Design Action		147.1461	kN	$\cos\alpha \times \text{Brace Axial Force}$
Design Action per Bolt	$F_{V,Ed}$	24.52435	kN	Design Action/ No. of Bolts
Bolt Grade		8.8		
	α_v	0.6		EN 1993-1-8 Table 3.4
Bolt		M20		
Tensile Stress Area	A_s	244.79	mm ²	
Bolt Ultimate Tensile Strength	f_{ub}	800	N/mm ²	EN 1993-1-8 Table 3.1
Shear Resistance per Bolt	$F_{V,Rd}$	93.99936	kN	$\alpha_v A_s f_{ub} / \gamma_{M2}$
Joint		563.9962	kN	$F_{V,Rd} \times \text{No. Bolts}$
Bearing - Bolt Plate				
Design Action per Bolt	$F_{V,Ed}$	24.52435	kN	Design Action/ No. of Bolts
Ultimate Bolt Strength	f_{ub}	800	N/mm ²	
Ultimate Plate Strength	f_u	360	N/mm ²	
	f_{ub}/f_u	2.222222		Minimum EN 1993-1-8 Table 3.3
	e_1	31	mm	26.4 mm; 1.2 d_0
	p_1	60	mm	48.4 mm; 2.2 d_0
	e_2	38	mm	26.4 mm; 1.2 d_0
	p_2	90	mm	52.8 mm; 2.4 d_0
	d_0	22	mm	
	$a_{d,end}$	0.469697		$e_1/3d_0$
	$a_{d,inner}$	0.659091		$p_1/3d_0 - 0.25$
	$a_{d,critical}$	0.469697		Smallest of $a_{d,end}$; $a_{d,inner}$
	$k_{1,end}$	2.5		smallest of 2.5; 1.4 $p_2/3d_0 - 1.7$; 2.8 $e_2/3d_0 - 1.7$
	$k_{1,inner}$	2.5		smallest of 2.5; 1.4 $p_2/3d_0 - 1.7$
	a_b	0.469697		Smallest of $a_{d,i}$; 1; f_{ub}/f_b
Bolt Diameter	d	20	mm	
Plate Thickness	t	15	mm	
Bearing Resistance	$F_{b,Rd}$	101.4545	kN	$\alpha_v k_1 f_u dt / \gamma_{M2}$
Joint		608.7273		

Lower Gusset - Plate Weld				
Weld Length	l_{total}	191	mm	
Weld Throat Thickness	a	6	mm	
Weld Effective Length	l_{eff}	179	mm	$L_{weld} - 2a$
No. of Welds		2	Welds	
Brace Axial Force	E_d	251.55	kN	
Weld Design Force		251.55	kN	Brace Axial Force - Conservative Assumption
Design Force per Unit Length	$F_{W,Ed}$	0.569897	kN/mm	Weld Force/($l_{eff} \times \text{No. of Welds}$)
Ultimate Material Strength	f_u	360	N/mm ²	
Reduction Factor	β_w	0.8		EN 1993-1-8 Table 1.4
Weld Resistance per Unit Area	$f_{vw,d}$	207.8461	N/mm ²	$f_u / \sqrt{3} / (\beta_w \gamma_{M2})$
Weld Resistance per unit Length	$F_{W,Rd}$	1.247077	kN/mm	$a \times f_{vw,d}$
Total Joint Resistance	$F_{joint,Rd}$	446.4534	kN	$F_{W,Rd} \times l_{eff} \times \text{No. Welds}$

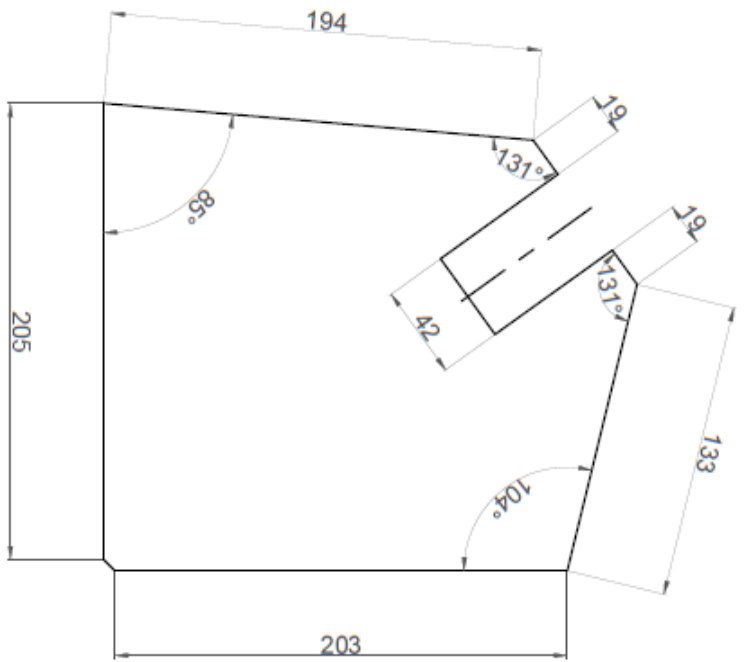
Bolted Connection - Lower Plate to Frame Support				
Shear				
Brace Axial Force	E_d	251.55	0	
Shear Design Force		251.55	0	Brace Axial Force - Conservative Assumption - Entire brace axial force transferred as shear
No. of Bolts		6		
Design Shear Force per Bolt	$F_{v,Ed}$	41.925		Design Tension Force/ Number of Bolts
Bolt Grade		8.8		
	α_v	0.6		
Bolt		M20		
Tensile Stress Area	A_s	244.79		
Bolt Ultimate Tensile Strength	f_{ub}	800		
Shear Resistance per Bolt	$F_{v,Rd}$	93.99936		
Tension				
Brace Axial Force	E_d	251.55	kN	
Tension Design Force		251.55	kN	Brace Axial Force - Conservative Assumption - Entire brace axial force transferred as tension
No. of Bolts		6		
Design Tension Force per Bolt	$F_{t,Ed}$	41.925		Design Tension Force/ Number of Bolts
k_2		0.9		EN 1993-1-8 Table 3.4
Bolt Ultimate Tensile Strength	f_{ub}	800	N/mm ²	EN 1993-1-8 Table 3.1
Tensile Stress Area	A_s	244.79	mm ²	
Tensile Resistance per Bolt	$F_{t,Rd}$	140.999	kN	$k_2 A_s f_{ub} / \gamma_{M2}$
Combined Tension & Shear				
Design Shear Force per Bolt	$F_{v,Ed}$	41.925		
Shear Resistance per Bolt	$F_{v,Rd}$	93.99936		
Design Tension Force per Bolt	$F_{t,Ed}$	41.925		
Tensile Resistance per Bolt	$F_{t,Rd}$	140.999		
Combined Tension & Shear Check		0.658401		$F_{v,Ed}/F_{v,Rd} + F_{t,Ed}/1.4F_{t,Rd} < 1$

Appendix F Drawings of Experimental Substructure

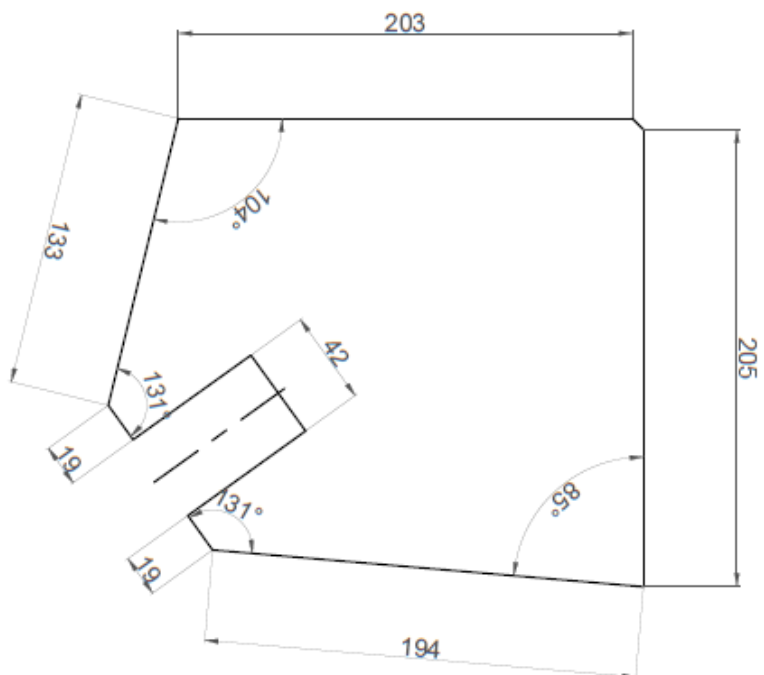




SITE:	TCD Structures Lab		DRAWING NO.		001	PROJECT NO.	01/04/2017	DATE	AMENDMENTS:		
			GP003								
TITLE:	Column to Gusset Plate Connection		SCALE AT A4.	2.5:1	J Hickey	CHECKED.	revision 1	REVISION.			

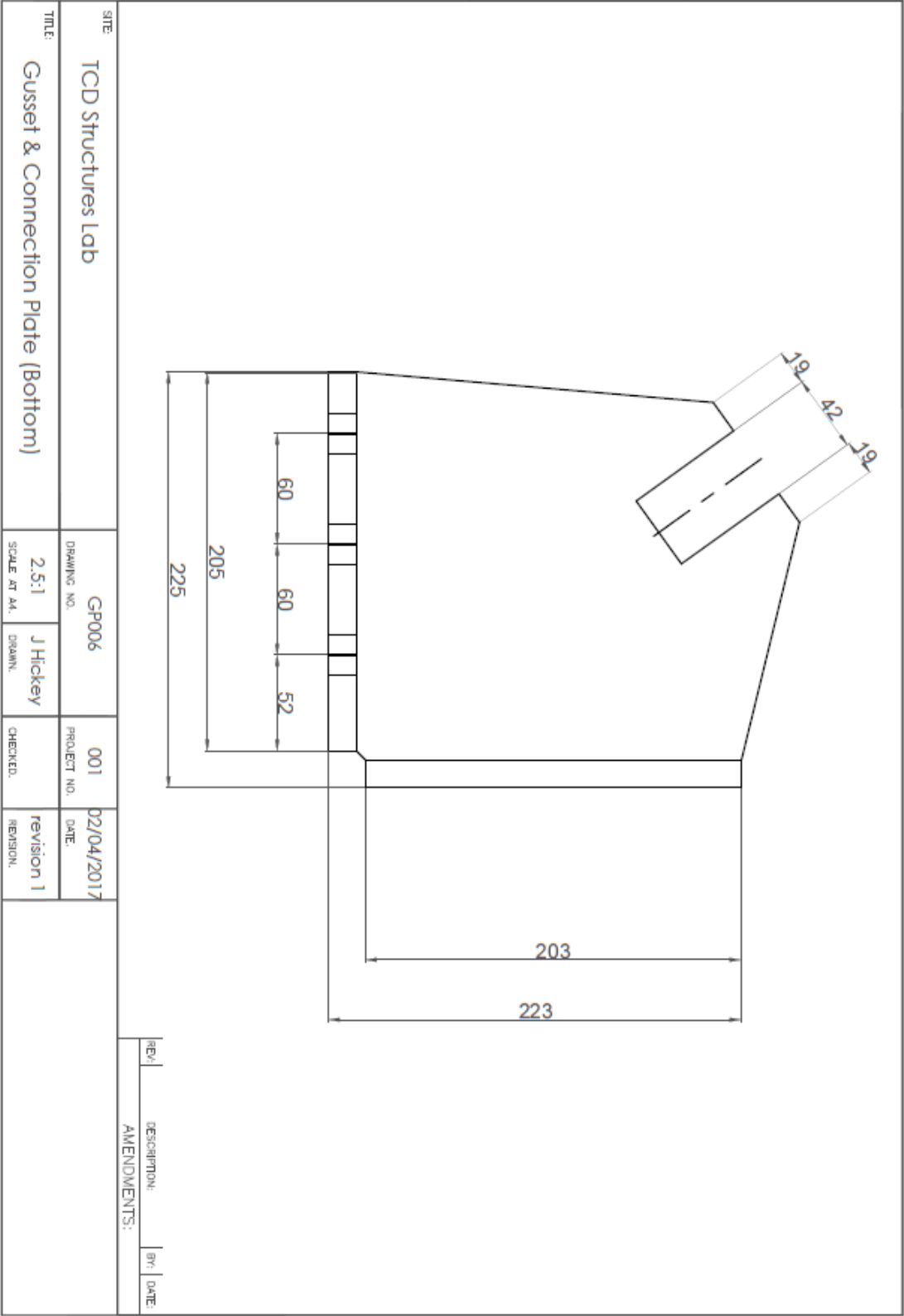


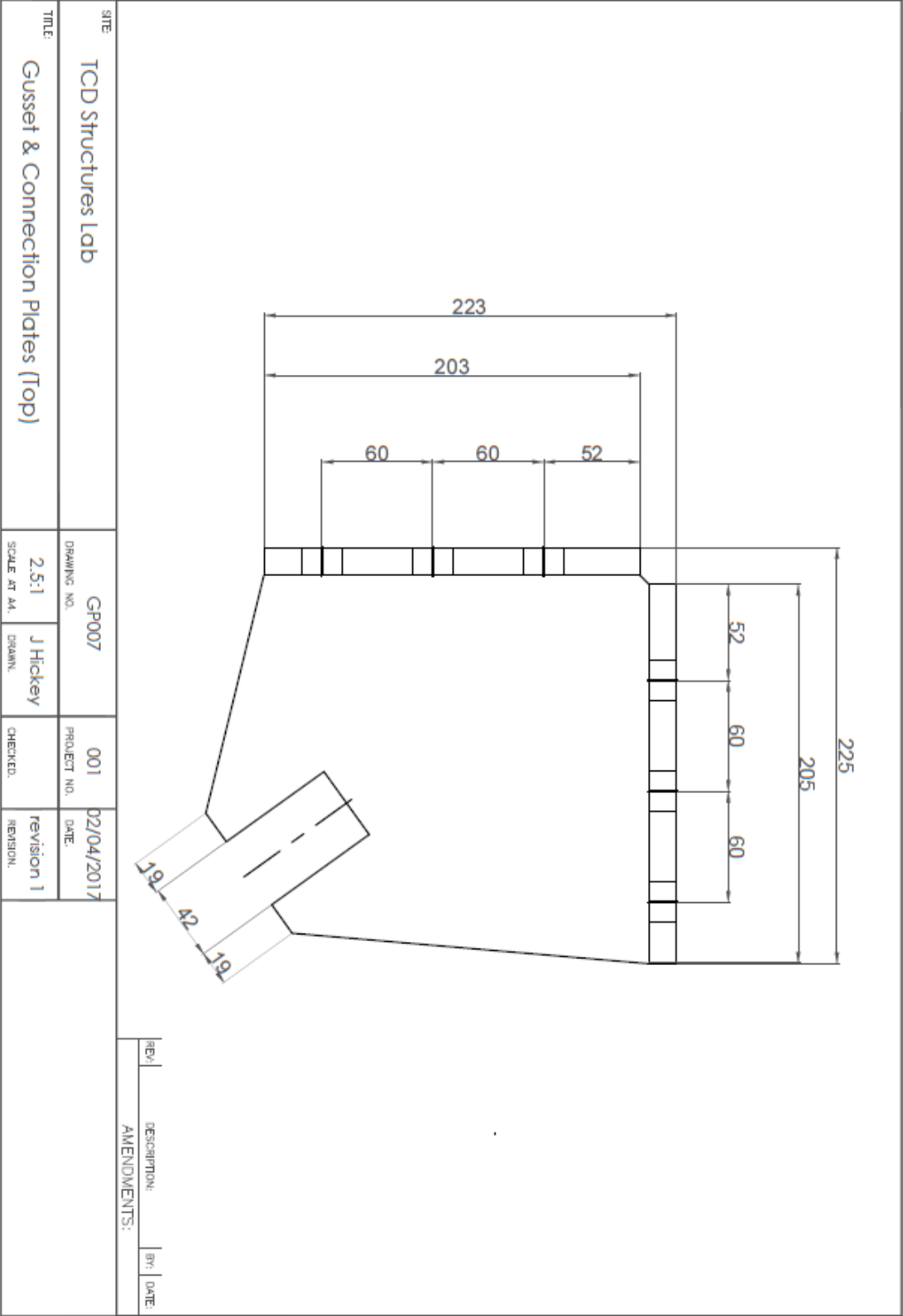
				REV#			DESCRIPTION		BY#		DATE	
AMENDMENTS:												
SITE		DRAWING NO.		PROJECT NO.		DATE						
TCD Structures Lab		GP004		001		02/04/2017						
TITLE:		SCALE AT A4		DRAWN		CHECKED		REVISION				
Gusset Plate (Bottom)		2.5:1		J Hickey				revision 1				

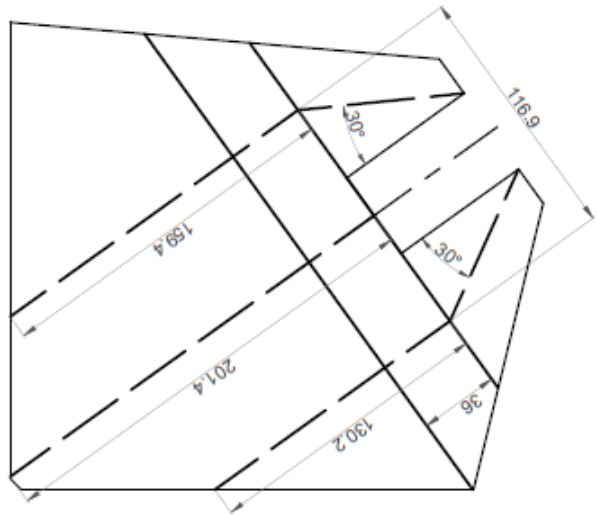


SITE		DRAWING NO.		PROJECT NO.		DATE	
TCD Structures Lab		GP005		001		02/04/2017	
TITLE		SCALE		DRAWN		CHECKED	
Gusset Plate (Top)		2.5:1		J Hickey		revision 1	
		AT A1.				REVISION.	

REV.	DESCRIPTION	BY	DATE
AMENDMENTS:			







SITE		DRAWING NO.		PROJECT NO.	DATE	REV.		DESCRIPTION	BY	DATE
TCD Structures Lab		GP008		001	03/04/2017	AMENDMENTS:				
TITLE:		SCALE	DRAWN	CHECKED	REVISION					
Gusset Plate Elevation - Design Dimensions		25:1 AT A4	J Hickey		revision 1					

Appendix G Model Updating OpenSees Code

Analysis code to Update Elastic Modulus (E); Yield Strength (F_y) using 2 numerical counterparts for each. Applies Updated values to an updated substructure; see Figure 10-18

```
#####
#Node & Element Numbering Convention
#####
## Model Created in Separate File
# For control substructure:
# Nodes 1, 2 base of columns
# Nodes 3, 4 top of columns, actuator connected to Node 3
# Node 5 base of brace

# For numerical counterpart n:
# n0000 + (Node number in Control Substructure)

# For Updated Substructure:
# 70000 + (Node number in Control Substructure)
#####

#Identify Parameters to be Updated
parameter 1 element 7000135 material #matID_NonLinear_Up E
parameter 2 element 7000136 material #matID_NonLinear_Up E

parameter 3 element 7000135 material #matID_NonLinear_Up sigmaY
parameter 4 element 7000136 material #matID_NonLinear_Up sigmaY

timeSeries Constant 3
set outFileID [open OptimizedData.txt w]
set has_yielded 0

#####
#Identify Update Parameters
#####
parameter 1 element 7000135 material #matID_NonLinear_Up E #Updated substructure brace E
parameter 2 element 7000136 material #matID_NonLinear_Up E #Updated substructure brace E

parameter 3 element 7000135 material #matID_NonLinear_Up sigmaY #Updated substructure
brace Fy
parameter 4 element 7000136 material #matID_NonLinear_Up sigmaY #Updated substructure
brace Fy

timeSeries Constant 3
set outFileID [open OptimizedData.txt w]
set has_yielded 0
#####
## Start Timer
#####
set start [clock microseconds]

set stored_sums {0}
set stored_sums_f {0}
set stored_sums_b {0}

puts "line 1"
for {set i 1} {$i < 101} {incr i} {
    lappend stored_sums 0
    lappend stored_sums_f 0
    lappend stored_sums_b 0
}

# Step 1 & 2: perform the analysis and update test structure
#set tTot [time {
for {set i 1} {$i < $duration} {incr i} {
    puts "Step $i"
    set t [time {analyze 1 [expr $dt/1.0]}]
```

```

# Obtain calculated displacment
set exp_deform [nodeDisp 3 1];          #Actuator Node in control substructure
#puts "exp_deform $exp_deform"
#####
#Step 3: Apply calculated displacement to numerical counterparts
#####
pattern Plain [expr 300+$i] 3 {
    #sp $nodeTag $DOFtag $DOFvalue
    sp 10003 1 $exp_deform              #Actuator Node in numerical counterpart 1
    sp 20003 1 $exp_deform              #Actuator Node in numerical counterpart 2
    sp 30003 1 $exp_deform              #Actuator Node in numerical counterpart 3
    sp 40003 1 $exp_deform              #Actuator Node in numerical counterpart 4
    sp 50003 1 $exp_deform              #Actuator Node in numerical counterpart 5
    sp 60003 1 $exp_deform              #Actuator Node in numerical counterpart 6
}
#####

#Step 2 & 3 Get Restoring Forces
#Control Substructure
if { $i==1 } {
    set b [nodeReaction 1]
    set c [nodeReaction 5]
    set d [nodeReaction 2]
    set b_d1 [lindex $b 0]
    set c_d1 [lindex $c 0]
    set d_d1 [lindex $d 0]
    set exp_force [expr $b_d1+$c_d1+$d_d1];
}

# Counterpart 1
set b [nodeReaction 10001]
set c [nodeReaction 10005]
set d [nodeReaction 10002]
set b_d1 [lindex $b 0]
set c_d1 [lindex $c 0]
set d_d1 [lindex $d 0]
set m1_force [expr $b_d1+$c_d1+$d_d1];

# Counterpart 2
set b [nodeReaction 20001]
set c [nodeReaction 20005]
set d [nodeReaction 20002]
set b_d1 [lindex $b 0]
set c_d1 [lindex $c 0]
set d_d1 [lindex $d 0]
set m2_force [expr $b_d1+$c_d1+$d_d1];

#####
## Step 4: Optimize Elastic Modulus E
#####

#Optimize
#####
#Idiot Optimization - Calculate weighting factors
set minimised_objective_fun 10000000000 ;#A big number
#set stored_sums $stored_sums_blank
for {set j 0} {$j < 101} {incr j} {

    #puts "j = $j"
    set w1 [expr $j*0.01]
    set w2 [expr 1-$w1]

    #puts "w1 = $w1, w2 = $w2"

    set previous_sum [lindex $stored_sums $j]
    #puts "Previous Sum = $previous_sum"
    set new_sum [expr $previous_sum + ($exp_force-$w1*$m1_force-$w2*$m2_force)**2]
    #puts "New Sum = $new_sum"
    set stored_sums [lreplace $stored_sums $j $j $new_sum]

    set sum_of_interest [lindex $stored_sums $j]
    #puts "new value in stored_sums = $sum_of_interest"
}

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set obj_fun_value [expr $sum_of_interest**0.5]
#puts "Objective function value = $obj_fun_value"

if { $obj_fun_value < $minimised_objective_fun } {
    set minimised_objective_fun $obj_fun_value
    set w1_final $w1
    set w2_final $w2
}
}

#####
## Step 4: Optimize yield strength Fy
#####
#Get Forces
if { $i==1 } {
    set b [nodeReaction 1]
    set c [nodeReaction 5]
    set d [nodeReaction 2]
    set b_d1 [lindex $b 0]
    set c_d1 [lindex $c 0]
    set d_d1 [lindex $d 0]
    set exp_force [expr $b_d1+$c_d1+$d_d1];
}
# # Counterpart 3
set b [nodeReaction 30001]
set c [nodeReaction 30005]
set d [nodeReaction 30002]
set b_d1 [lindex $b 0]
set c_d1 [lindex $c 0]
set d_d1 [lindex $d 0]
set m3_force [expr $b_d1+$c_d1+$d_d1];

# Counterpart 4
set b [nodeReaction 40001]
set c [nodeReaction 40005]
set d [nodeReaction 40002]
set b_d1 [lindex $b 0]
set c_d1 [lindex $c 0]
set d_d1 [lindex $d 0]
set m4_force [expr $b_d1+$c_d1+$d_d1];

#Optimize
#####
if {$has_yielded == 1} {
    #Simplistic Optimization - Calculate weighting factors
    set minimised_objective_fun 10000000000 ;#A big number
    #set stored_sums $stored_sums_blank
    for {set j 0} {$j < 101} {incr j} {

        #puts "j = $j"
        set w1 [expr $j*0.01]
        set w2 [expr 1-$w1]

        #puts "w1 = $w1, w2 = $w2"

        set previous_sum [lindex $stored_sums_f $j]
        #puts "Previous Sum = $previous_sum"
        set new_sum [expr $previous_sum + ($exp_force-$w1*$m3_force-
$w2*$m4_force)**2]
        #puts "New Sum = $new_sum"
        set stored_sums_f [lreplace $stored_sums_f $j $j $new_sum]

        set sum_of_interest [lindex $stored_sums_f $j]
        #puts "new value in stored_sums = $sum_of_interest"
        set obj_fun_value [expr $sum_of_interest**0.5]
        #puts "Objective function value = $obj_fun_value"

        if { $obj_fun_value < $minimised_objective_fun } {
            set minimised_objective_fun $obj_fun_value
            set w1_f_final $w1
            set w2_f_final $w2
        }
    }
}

```

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    }
}

#####
puts -nonewline $outFileID $exp_force;
puts -nonewline $outFileID " $m1_force";
puts -nonewline $outFileID " $m2_force";

puts -nonewline $outFileID " $w1_final";
puts -nonewline $outFileID " $w2_final";

#Step 5 & 6: Identify if parameter should be updated & Calculate value of updated
parameters
#If control substructure has not yielded update E, otherwise don't update
if { $has_yielded == 0 } {
    set E_Updated [expr $Emodel1E*$w1_final+ $Emodel2E*$w2_final]
}
puts -nonewline $outFileID " $E_Updated";

#If control substructure has yielded update Fy, otherwise don't update
if { $has_yielded == 1 } {
    set fy_Updated [expr $fy_model1f*$w1_f_final+ $fy_model2f*$w2_f_final]
} else {
    set w1_f_final 0.5
    set w2_f_final 0.5
    set fy_Updated [expr $fy_model1f*$w1_f_final+ $fy_model2f*$w2_f_final]
}
#Print updated values to file
puts -nonewline $outFileID " $w1_f_final";
puts -nonewline $outFileID " $w2_f_final";
set fy_Updated_print [expr $fy_Updated*1000]
puts -nonewline $outFileID " $fy_Updated_print";

puts $outFileID " $has_yielded"

# Restoring force for control substructure: needs to be measured at the end of the
# analysis step to synchronise with numerical counterparts
if { $i!=1 } {
    set b [nodeReaction 1]
    set c [nodeReaction 5]
    set d [nodeReaction 2]
    set b_d1 [lindex $b 0]
    set c_d1 [lindex $c 0]
    set d_d1 [lindex $d 0]
    set exp_force [expr $b_d1+$c_d1+$d_d1];
}
#####
## Check Yield
#####
set stiffness [expr $exp_force/$exp_deform]
puts -nonewline $outFileID_k "$stiffness"
set stiffness_sum [expr $stiffness_sum+$stiffness]
puts -nonewline $outFileID_k " $stiffness_sum"
set ave_stiffness [expr $stiffness_sum/$i]
puts $outFileID_k " $ave_stiffness"
if { $i == 100 } {
    set linear_stiffness $ave_stiffness
    puts "Estimated Linear Stiffness = $linear_stiffness"
    after 3000
}
if { $i > 100 && $has_yielded == 0 } {
    set offset 50
    set F_Plus [expr $exp_deform*$linear_stiffness+$offset]
    set F_Minus [expr $exp_deform*$linear_stiffness-$offset]
    if { $exp_force > $F_Plus || $exp_force < $F_Minus } {
        set has_yielded 1
        puts "Yield Point = $i"
        after 3000
    }
}
#puts $outFileID_k " $yield_check"

```

```

        #Step 7: Update Parameters
        updateParameter 1 $E_Updated
        updateParameter 2 $E_Updated
        updateParameter 3 $fy_Updated
        updateParameter 4 $fy_Updated
    }
    #Step 8: Analysis step finished; return to start of loop for next step

#####
##End Timer
set end [clock microseconds]
puts "Analysis Completed, step: $i"
puts "Time Taken  = [expr {($end - $start)}] microseconds"

# close the output file
close $outFileID

puts "Analysis Finished"

```