

Seismic Behaviour and Design of Steel Frames Incorporating Tubular Members

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I confirm that this submission is my own work. In it, I give references and citations whenever I refer to, describe or quote from the published, or unpublished, work of others.

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”Vulgar and inactive minds confound familiarity with knowledge, and conceive themselves informed of the whole nature of things when they are shown their form or told their use; but the Speculatist, who is not content with superficial views, harasses himself with fruitless curiosity, and still as he enquires more perceives only that he knows less.”

Samuel Johnson, *The Idler*.

”The fear of the LORD is the beginning of knowledge”

Proverbs 1:7

To Joaquin and Kari,
for giving me roots and wings.

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Abstract

This thesis deals with the seismic behaviour of steel frames with particular focus on structures that employ tubular members as either columns or bracing elements. It addresses a number of design and assessment issues at the local (connection), frame, and overall (system interaction) levels. At the connection level, two experimental investigations on: (i) blind-bolted and angle connections, and (ii) combined channel/angle connections, are presented. The main behavioural patterns and the effects of key design parameters on the connection performance are examined. Refined mechanical models able to estimate the response of these connecting details are developed. These mechanical models are subsequently employed to perform parametric studies based on which simplified design-oriented expressions for the estimation of stiffness, strength and ductility are suggested. The susceptibility to low-cycle fatigue within critical connection components and the predictions of available fatigue damage models are also assessed.

At the frame level, an evaluation of the inelastic demands on moment-resisting, partially-restrained and concentrically-braced steel structures is performed and equivalent linear models for the estimation of peak deformations are proposed. Particular attention is given to the influence of a number of scalar ground-motion frequency content parameters on the estimation of peak displacements. Additionally, simplified models based on rigid-plastic dynamics, and implemented within response history analysis, are proposed. It is shown that such rigid-plastic models can predict global deformations with reasonable accuracy.

At the system interaction level, a comparative assessment of the peak response of one-way, two-way and mixed framing configurations under bi-directional earthquake loading is studied by means of idealized 3D simplifications and refined 2D models. This enables a detailed quantification of the contribution of gravity frames to the reduction of seismic risk and highlights the benefits of proper secondary frame design in mitigating the probabilities of dynamic instability. Finally, the findings of the thesis are summarized and future research areas are identified.

Contents

Abstract	xi
Contents	xiii
List of Figures	xix
List of Tables	xxv
Notations	xxvii
I Introduction	1
1 Preamble	3
1.1 Background	3
1.2 Motivation and objectives	4
1.3 Outline of the thesis	4
2 Literature Review	9
2.1 Introduction	9
2.2 Open beam-to-tubular column connections	9
2.3 Seismic design and assessment of steel frames	13
2.4 Behaviour of framing configurations	17
2.5 Summary	18
II Connection Behaviour	21
3 Experimental Response of Blind-bolted Angle Connections	23
3.1 Introduction	23

CONTENTS

3.2	Testing arrangement and specimen details	24
3.2.1	Experimental set-up	24
3.2.2	Beam-to-column connection specimens	25
3.2.3	Characteristics of Hollo-bolts	27
3.2.3.1	General	27
3.2.3.2	Direct tension tests	28
3.3	Experimental results and observations	30
3.3.1	Monotonic tests	31
3.3.1.1	Deformation patterns	31
3.3.1.2	Gauge distance	34
3.3.1.3	Hollo-bolt Grade	34
3.3.1.4	Column thickness	34
3.3.1.5	Web angles	35
3.3.1.6	Beam width	36
3.3.2	Cyclic behaviour	37
3.3.2.1	Hysteretic response	37
3.3.2.2	Ductility and energy dissipation	38
3.4	Concluding Remarks	38
4	Experimental Response of Combined Channel/Angle Connections	43
4.1	Introduction	43
4.2	Experimental set-up and details	44
4.2.1	Testing arrangement	44
4.2.2	Connection specimens	44
4.3	Main results and observations	46
4.3.1	Monotonic tests	49
4.3.1.1	Deformation patterns	49
4.3.1.2	Gauge distance	49
4.3.1.3	Reverse channel thickness	51
4.3.1.4	Web angles	53
4.3.2	Cyclic behaviour	55
4.3.2.1	Hysteretic response	55
4.3.2.2	Ductility and energy dissipation	56
4.3.2.3	Fatigue Damage	58
4.4	Concluding Remarks	60

5	Mechanical Models for Angle Connections	63
5.1	Introduction	63
5.2	Model development	64
5.2.1	General	64
5.2.2	Prying action	66
5.2.3	Column face component	68
5.2.4	Hollo-bolt response	69
5.2.5	Beam flange stiffness	70
5.2.6	Standard bolts	70
5.2.7	Tensile behaviour of horizontal leg (beam side)	71
5.2.8	Bolt slippage	71
5.2.9	Force-displacement solution procedure	71
5.2.10	Model adaptation to combined channel/angle connections	74
5.2.11	Hysteretic rules and overall joint response	77
5.3	Comparison with test results	78
5.4	Parametric assessment of Hollo-bolted angle connections	80
5.4.1	Angle thickness and Hollo-bolt grade	83
5.4.2	Column face flexibility	85
5.4.3	Gauge distance	86
5.5	Design considerations for Hollo-bolted angle connections	87
5.5.1	Initial stiffness	88
5.5.2	Failure mode	88
5.5.3	Bolt-row capacity	89
5.5.4	Overall moment-rotation response	91
5.6	Design considerations for reverse channel connections	94
5.6.1	Connection initial stiffness	94
5.6.2	Connection capacity	96
5.7	Validation of fatigue damage models	97
5.8	Concluding remarks	101
III	Frame Behaviour	103
6	Evaluation of Inelastic Seismic Demands	105
6.1	Introduction	105
6.2	Structural systems and earthquake ground-motions	106
6.2.1	PR Pinching Model	106

CONTENTS

6.2.2	CB Pinching Model	108
6.2.3	Ground-motions considered	108
6.2.4	Scope of parametric analysis	110
6.3	Assessment of inelastic demands	112
6.3.1	Inelastic displacement demands	112
6.3.2	Effect of level of inelastic behaviour	115
6.3.3	Effect of soil conditions	117
6.4	Influence of scalar frequency content parameters	119
6.4.1	Frequency content parameters investigated	119
6.4.2	Results of statistical study	121
6.5	Proposed equivalent linear models	124
6.5.1	Non-iterative equivalent linearization models	124
6.5.2	Verification and application	126
6.6	Concluding remarks	130
7	Rigid-Plastic Models for Seismic Design and Assessment	133
7.1	Introduction	133
7.2	Moment resisting (MR) frames	135
7.2.1	Rigid-plastic idealization for SDOF systems	135
7.2.2	Response prediction of MDOF systems	138
7.3	Centrally-braced (CB) frames	140
7.3.1	Rigid-plastic idealization for CB frames	140
7.3.2	Response prediction of CB SDOF systems	143
7.3.3	Response prediction of multi-storey CB systems	144
7.4	Dual frames	147
7.4.1	Rigid-plastic idealization of dual systems	147
7.4.2	Response prediction of dual SDOF systems	148
7.4.3	Response prediction of dual multi-storey frames	149
7.5	Rigid-plastic design procedures	152
7.5.1	Relationship between strength and structural response	152
7.5.2	Rigid-plastic design of steel buildings	153
7.6	Concluding remarks	158
IV	Building Systems	161
8	Seismic Response of Steel Framed Buildings	163

8.1	Introduction	163
8.2	Steel framing systems incorporating tubular members	164
8.3	Seismic drift demands in building configurations	166
8.3.1	3D frame idealizations	166
8.3.2	Building response under static loads	168
8.3.3	Building response under dynamic loads	171
8.4	Contribution of secondary systems to the seismic performance of CB buildings in near-collapse stages	174
8.4.1	Generalities and design approach	174
8.4.2	Evaluation of design scenarios	177
8.4.3	Case study	179
8.5	Concluding remarks	181
V	Closure	185
9	Conclusions	187
9.1	Summary of main findings	187
9.2	Design and assessment contributions	190
9.3	Limitations and future work	191
	Appendixes	197
A	Matlab Implementation of the Mechanical Model	197
B	Supplemental Results on Seismic Demands in Steel Structures	201
C	Estimation of Fatigue Damage through Rigid-plastic Models	209
	References	213

List of Figures

3.1	Experimental set-up.	24
3.2	Details of connection configurations.	25
3.3	Loading protocol considered in the cyclic tests [40].	27
3.4	Lindapter Hollo-bolt before (left) and after (right) installation [98]. . . .	28
3.5	Set-up for direct tension tests on Hollo-bolts.	29
3.6	Hollo-bolt load-axial displacement relationships.	30
3.7	Deformation and fracture of Grade 8.8 Hollo-bolt after failure.	30
3.8	Deformation patterns observed at the end of tests.	32
3.9	Example of strain measurements.	33
3.10	Moment-rotation relationships for top and seat angle connections. . . .	35
3.11	Moment-rotation relationships for top, seat and web angle connections. .	36
3.12	Column face deformation in the compression zone of Specimen B10-G8.8- d40-M.	37
3.13	Moment-rotation hysteresis for top and seat angle connections.	39
3.14	Moment-rotation hysteresis for top, seat and web angle connections. . .	40
3.15	Fracture in top angle due to low-cycle fatigue in Specimen A10-G10.9- d65-Y.	40
3.16	Cumulative energy dissipation.	40
4.1	Details of connection configurations.	45
4.2	Main deformation patterns observed in monotonic tests.	50
4.3	Moment-rotation relationships for monotonic tests.	50
4.4	Influence of gauge distance.	52
4.5	Influence of channel thickness.	54
4.6	Deformation at the end of test in Specimen W10-A8-d40-M.	55
4.7	Influence of web angles.	55
4.8	Hysteretic moment-rotation relationships.	57

LIST OF FIGURES

4.9	Cumulative energy dissipation.	58
4.10	Fatigue damage.	59
5.1	Mechanical model for angle connections.	65
5.2	Value of factor f_{pry} in Equation 5.3 as a function of the Hollo-bolt relative stiffness.	67
5.3	Axial response of M16 Hollo-bolts.	69
5.4	Schematic representation of the decision tree used for the mechanical model. Phase I.	73
5.5	Schematic representation of the decision tree used for the mechanical model. Phase II.	74
5.6	Force-deformation characterization of reverse channel component.	76
5.7	Hysteretic load-displacement response of bolt-row under cyclic loading.	77
5.8	Comparison of experimental and analytical response of Hollo-bolted angle connections under monotonic loading.	79
5.8	Comparison of experimental and analytical response of Hollo-bolted angle connections under monotonic loading (Cont.).	80
5.9	Comparison of experimental and analytical response of combined channel/angle connections under monotonic loading.	81
5.10	Comparison of experimental and analytical response of Hollo-bolted angle connections under cyclic loading.	82
5.11	Comparison of experimental and analytical response of combined channel/angle connections under cyclic loading.	83
5.12	Influence of angle thickness for Grade 10.9 Hollo-bolts.	84
5.13	Influence of angle thickness for Grade 8.8 Hollo-bolts.	84
5.14	Influence of column face slenderness (b_c/t_c) for Grade 10.9 Hollo-bolts.	85
5.15	Influence of column face slenderness (b_c/t_c) for Grade 8.8 Hollo-bolts.	86
5.16	Influence of gauge distance d for Grade 10.9 bolts.	87
5.17	Influence of gauge distance d for Grade 8.8 bolts.	87
5.18	Total bolt-row displacement at which blind-bolt yield would be expected as a function of ht_f/d'^2 for M16 Grade 10.9 Hollo-bolts.	90
5.19	Relationship between F_{pi}/F_{pg} (as defined by Equation 5.22) and b_c/t_c for different d'/t_f ratios and M16 Grade 10.9 Hollo-bolts.	91
5.20	Relationship between strain hardening stiffness (k_{pi}) and initial stiffness (k_i) as a function of the column face slenderness (b_c/t_c).	91
5.21	Design-oriented simplified mechanical model of angle connection.	92

LIST OF FIGURES

5.22	Comparison of experimental results and predictions of the more simplified design-oriented component model for monotonic tests on Hollow-bolted angle connections.	95
5.23	Comparison of experimental and predicted moment-rotation relationships for monotonic tests on combined channel/angle connections. . . .	97
5.24	Fatigue damage analysis for Specimen A10-G10.9-d65-Y.	99
5.25	Fatigue damage analysis for Specimen T10-A8-d40-Y.	99
5.26	Fatigue damage analysis for Specimen T6.3-A8-d40-Y.	100
5.27	Fatigue damage analysis for Specimen T10-A8-d65-Y.	100
6.1	PR behaviour and modelling.	107
6.2	CB behaviour and modelling.	108
6.3	Median normalized acceleration response spectra for different soil classes. . . .	110
6.4	Comparison of hysteretic models of PR and CB systems.	112
6.5	Inelastic displacement demands for bi-linear systems.	113
6.6	Mean inelastic displacement ratios of PR systems normalized by mean displacement ratios of bi-linear systems.	115
6.7	Mean inelastic displacement ratios for CB systems normalized by mean ratios of bi-linear systems.	116
6.8	Effect of lateral strength ratio on the inelastic displacement ratios of PR systems.	117
6.9	Effect of lateral strength ratio on the inelastic displacement ratios of CB systems.	117
6.10	Effect of soil conditions on displacement demands of PR systems.	118
6.11	Effect of soil conditions on displacement demands of CB systems.	120
6.12	Coefficient of variation of inelastic displacement ratios of bi-linear systems for different normalized periods. $R = 5$	123
6.13	Coefficient of variation of inelastic displacement ratios of PR systems for different normalized periods, $R = 5$ and different Pinching Factors P	123
6.14	Coefficient of variation of inelastic displacement ratios of CB systems for different normalized periods, $R = 5$ and different normalized slenderness $\bar{\lambda}$	123
6.15	Median ratio of approximate to exact (δ_{ap}/δ_{ex}) peak inelastic displacements for PR models.	127
6.16	COV of approximate to exact (δ_{ap}/δ_{ex}) peak inelastic displacements for PR models.	128

LIST OF FIGURES

6.17	Median ratio of approximate to exact peak inelastic displacements for CB models.	128
6.18	COV of approximate to exact (δ_{ap}/δ_{ex}) peak inelastic displacements for CB models.	128
6.19	Application of the equivalent linearization on the displacement estimation of a structure.	129
7.1	Rigid-plastic oscillator subjected to ground-motion excitation.	136
7.2	Integration algorithm for rigid-plastic MR frames.	137
7.3	Rigid-plastic analysis of a MDOF MR frame.	139
7.4	Rigid-plastic analysis of a SDOF CB frame.	141
7.5	Integration algorithm for rigid-plastic concentrically-braced frames. . . .	142
7.6	Ratio of peak drift predictions (rigid-plastic to non-linear model) as a function of slenderness and PGA/a_y for the Beverly Hills Station 1994 Northridge Earthquake record.	144
7.7	Plan layout and elevation of the four-storey CB frame under study. . . .	145
7.8	Rigid-plastic analysis of a four-storey CB frame.	146
7.9	Rigid-plastic analysis of dual SDOF structures.	149
7.10	Effect of brace slenderness and ratio PGA/a_y in the accuracy of rigid-plastic predictions.	150
7.11	Rigid-plastic analysis of a four-storey dual frame.	151
7.12	Rigid-plastic relationship between lateral strength and peak displacement for a dual system with flexural strength of 20% of the total capacity when subjected to the 1994 Northridge earthquake Sylmar Station record.	153
7.13	Plan layout and elevation of the six-storey steel frame with fixed bases used for design.	154
7.14	Drift at effective height vs. base shear relationship for dual rigid-plastic oscillators with flexural resistance of 20% of the total building capacity. Thin lines represent relationships for individual records. The median values and median ± 1 standard deviation are highlighted. The standard deviation corresponding to 3.5% design drift is $\pm 0.8\%$	155
7.15	Drift at equivalent height of the 6-storey steel braced building with fixed columns when subjected to records corresponding to a 2% probability of exceedance in Los Angeles area.	156
7.16	Seismic design framework with de-coupled assignment of stiffness and capacity based on elastic and rigid-plastic models.	158

LIST OF FIGURES

8.1	Framing systems.	165
8.2	Comparison of histories of first storey drift in complete frame and simplified models by Tagawa et al. [146].	166
8.3	Planar simplification to a fishbone model of a one-way building.	167
8.4	Simplified 3D models.	168
8.5	Pushover and drift concentration curves for the three-storey one-way framing system with MR frames.	169
8.6	Pushover and drift concentration curves for the three-storey one-way framing system with CB frames.	170
8.7	Pushover and drift concentration curves for the three-storey two-way and mixed framing systems.	171
8.8	Ductility versus strength demand curves. One-way framing system with MR frames.	172
8.9	IDA curve and COV for one-way framing systems with MR frames. T=0.4 seconds.	173
8.10	Ductility versus strength demand curves. One-way framing system with CB frames.	174
8.11	IDA curve and COV for one-way framing systems with CB frame. T=0.2 seconds.	174
8.12	Ductility versus strength demand curves. Two-way and mixed framing systems.	175
8.13	IDA curve and COV for two-way and mixed framing systems. T=0.4 seconds.	175
8.14	Force-displacement relationships for the overall structure (left), primary (middle) and secondary (right) systems normalized to yield of primary system.	176
8.15	SDOF model of primary CB and secondary PR-MR for collapse parametric studies.	178
8.16	Median collapse capacities for hybrid CB + PR-MR SDOF systems. . .	178
8.17	One-way CB case study building for collapse stage verification.	179
8.18	Collapse probability assessment of case study building.	181
8.19	Collapse mechanisms.	182
8.20	One-way CB case study building for collapse stage verification.	182
A.1	Iterative algorithm for component assemblage.	199
A.2	Graphical user interface for the mechanical model.	200

LIST OF FIGURES

A.3	GUI display of generated moment-rotation response.	200
B.1	R- μ -T relationships.	202
B.2	Fatigue damage index of PR systems.	204
B.3	Park and Ang damage index of bi-linear and PR systems.	205
C.1	Prediction of local strains.	210
C.2	Comparison of strain demand history for the CB oscillator with 40x2.5 SHS when subjected to the El Centro record scaled so as to obtain a ratio PGA/a_y of 2.5.	212

List of Tables

3.1	Summary of test programme.	26
3.2	Summary of results for monotonic tests.	31
3.3	Summary of results for cyclic tests.	31
4.1	Summary of test programme.	47
4.2	Material properties.	48
4.3	Summary of results for monotonic tests.	48
4.4	Summary of results for cyclic tests.	48
5.1	Hollo-bolt axial force-displacement relationships.	70
6.1	Summary of earthquake ground-motions used in this study.	109
7.1	Characteristics of Los Angeles ground-motions [142] with a 2% probability of exceedance in 50 years, used for the design example.	155
8.1	Characteristics of the simplified 3D models.	169
A.1	Stiffness at different deformation stages.	198
B.1	Acceleration time series used in Chapter 6.	206
B.1	Acceleration time series used in Chapter 6 (Cont.).	207

Notation

A_b	standard bolt cross sectional area
a_r	relative acceleration
a_y	acceleration at yield
a'	effective distance between the point of application of B_b and the root of the angle
B_b	axial force in the standard bolt connecting the angle to the beam
B_c	axial force in the Hollo-bolt connecting the angle to the column
B_{RC}	reverse channel yield strength
B_{yc}	column face yield strength
b'	effective distance between the points of application of B_b and Q_b
C_R	inelastic displacement ratio
C_t	coefficient for the calculation of k_{cf}
c_f	friction coefficient
c'	effective distance between the points of application of B_c and Q_c
DCF'	drift concentration factor
D_H	Hollo-bolt hole diameter
D_T	accumulated fatigue damage index
d_{bh}	bolt head diameter
d_h	bolt hole diameter
d_{Hb}	Hollo-bolt diameter
d_r	relative displacement
d'	effective distance between the point of application of B_c and the root of the angle
E	Young's modulus
F_i	bolt-row force
F_{pg}	axial force at the formation of plastic mechanism in the leg of the angle connected to the column (vertical leg)
F_{pi}	bolt-row plastic axial force
F_{total}	total external axial force applied to the connection
F_y, F_{yp}, F_{yn}	overall, maximum positive and maximum negative structural yield strengths
F_{sp}, F_{sn}	positive and negative structural pinching strengths
f_k	factor for the calculation of B_y
f_{pry}	factor accounting for the change in the plastic hinge location in the leg of the angle component
$GM S_a$	geometric mean spectral acceleration
H	summation of beam height and bottom angle thickness
h_P	primary system post-elastic stiffness coefficient
h_T	total building post-elastic stiffness coefficient

NOTATION

I	moment of inertia of the angle cross section ($pt^3/12$)
K_P	primary system stiffness
K_S	secondary system stiffness
K_T	total building stiffness
k_{bf}	beam flange stiffness
k_{cf}	column face tension stiffness
k_{comp}	column face compression stiffness
k_{eq}	equivalent stiffness of web and top angle rows
k_g	angle stiffness parameter used to relate f_{pry} and k_{Hb}
k_{Hb}	Hollo-bolt stiffness
k_i	bolt-row stiffness
k_{pi}	bolt-row strain hardening stiffness
k_{RC}	reverse channel stiffness
k_{sb}	standard bolt axial stiffness
k_{slip}	stiffness related to bolt-slippage
k_T	top angle bolt-row stiffness
k_t	tensile stiffness of angle leg connected to the beam (horizontal leg)
L	reverse channel effective width
L_{HB}	horizontal spacing between Hollo-bolts in the bolt-row
L_b	effective length of standard bolt mobilized in tension
M_{pb}	beam plastic capacity
M_{total}	total external moment applied to the connection
M_y	overall connection plastic moment capacity
m	mass
m_b	lever arm measured between the beam web and the point of application of B_b
m_{pla}	flexural plastic capacity of the angle leg
m_{plc}	plastic moment of column face per unit length
m_y	flexural strength of the angle leg at yielding
N	number of bolts in the bolt-row
N_f	number of cycles to fatigue failure
P	pinching factor
PGA	peak ground acceleration
P_f	friction slip resistance
p	angle effective width
p_{pf}	pre-stressing force due to bolt tightening
Q_b	prying force at the leg of the angle connected to the beam (horizontal leg)
Q_c	prying force at the leg of the angle connected to the column (vertical leg)
R	strength ratio
R_m	reverse channel yield mechanism radius
r	angle root radius
S_a	spectral acceleration
S_θ	connection rotational stiffness
T	period
T_{aver}	ground-motion average spectral period
T_{eq}	equivalent period
T_g	ground-motion predominant period
T_g	ground-motion characteristic period
T_m	ground-motion mean period
T_o	ground-motion smoothed spectral predominant period

t_b	beam flange thickness
t_c	column face thickness
t_f	angle thickness
t_s	time
t_s, t_1, t_2	Modified Richard Abbott shape parameters
v	bolt-row displacement
v_{pi}	bolt-row displacement at the attainment of the plastic capacity in the top angle
v_r	relative velocity
y_{eq}	equivalent lever arm of web and top angle rows
y_i	distance between bolt-row and reference datum
y_T	distance between top angle bolt-row and the point of rotation of the joint
α, α_1	regression parameters for the calculation of the bolt-row capacity
δ	deformation (displacement or rotation)
δ_m	maximum deformation
Φ	shape vector
Γ	base shear coefficient
γ	factor used in the calculation of B_y
$\bar{\lambda}$	normalized slenderness
ν	Poisson's ratio
Θ	stability coefficient
$\Theta_{s,max}$	maximum storey drift along the height of the building
$\Theta_{s,y}$	yield storey drift
θ	overall joint rotation
θ_{jpi}	effective plastic equiamplitude rotation
θ_{pi}	overall joint rotation at the attainment of the plastic capacity in the top angle
σ_y	material yield strength
μ	ductility
ξ_{eq}	equivalent damping

Part I

Introduction

Chapter 1

Preamble

1.1 Background

Over the last few decades, Earthquake Engineering has taken significant strides towards a full understanding of both the seismic demand and the structural response of commonly used structures. Indeed, since the first ground acceleration recordings became available, seismic design has evolved from no consideration of earthquake action whatsoever in the early 30's to the design and assessment at several performance levels in today's practice. Capacity and ductility considerations initially studied in the 50's and 60's [8] remain today as the philosophical basis for seismic design and have been established in various forms into current seismic design provisions [28, 56]. As the importance of economic, resilience and sustainability issues is being increasingly recognised, several attempts are also being made in order to incorporate reliability considerations, life-time costs, loss estimations and downtime costs into the design process [99].

Field inspection and evaluation of buildings performance during recent earthquakes have significantly increased our understanding of structural behaviour and fuelled the progress of earthquake resistant design and construction practices. The unexpected levels of damage observed in earthquakes during the 90's motivated several investigation initiatives aimed at producing guidelines for assessing and improving the performance of moment resisting steel frames mainly incorporating open sections [54]. In fact, the widespread damage observed in welded connections during the Northridge earthquake prompted engineers to seek new and more ductile and economical connection alternatives for moment resisting frames and increased the interest in other steel framing

1. PREAMBLE

configurations such as concentrically-braced frames. To this end, structural Hollow Sections offer a number of advantages over their open section counterparts in terms of both structural efficiency and architectural appeal. In particular, the torsional stiffness of tubular sections and their favourable strength to weight ratio make them an ideal choice as column and bracing members in steel frames. Nevertheless, the difficulties associated with connection detailing and the limited research available in terms of the overall frame seismic behaviour have often resulted in underexploitation of these merits.

1.2 Motivation and objectives

As noted in the previous section, there is a need to evaluate the seismic response of steel framed structures incorporating tubular members. In this context, reliable analytical tools are indispensable to predict with reasonable accuracy the structural response well into the non-linear range typically attained by structures subjected to severe ground-motion. Yet, these tools need to be simple enough to facilitate their adoption in practice thus contributing towards the final goal of seismic risk reduction. At the same time, seismic design recommendations need to be improved in order to favour a reliable structural performance at each of the local (i.e. connection), frame and overall (system interaction) levels.

Motivated by the above-mentioned factors, this thesis aims to: *contribute to enhancing the fundamental understanding of the seismic behaviour of steel framed structures incorporating tubular members, and to develop reliable analysis and design methods with a view to improving seismic design guidance from a local, global and system interaction perspective.*

In light of the above, a number of issues related to the seismic behaviour of steel frames incorporating tubular members are examined in this thesis; and experimental, analytical and numerical methods are applied as outlined in the following section.

1.3 Outline of the thesis

This thesis deals with the seismic behaviour of steel frames with particular focus on structures that employ tubular members as either columns or bracing elements. It

addresses a number of design and assessment issues at the local connection level (considered in Part II of this thesis), frame level (dealt with in Part III), and overall system interaction level (examined in Part IV). The thesis is divided into five parts comprising nine chapters. It aims to achieve the objectives of Section 1.2 through a combination of experimental, analytical and numerical approaches. Part I is the introduction to the thesis, it includes this chapter and Chapter 2 which reviews previous research and sets the context for the contributions made in subsequent chapters.

Part II focuses on the response of practical and cost-effective semi-rigid bolted connection alternatives between open beams and tubular members, on which design guidance is lacking in comparison with fully-rigid configurations. Chapter 3 describes an experimental investigation into the behaviour of blind-bolted angle connections subjected to monotonic and cyclic loading. Apart from direct tension tests on *Hollo-bolts*, seventeen tests with different connection configurations are presented. The specimens tested include *top and seat* as well as *top, seat and web angle* connections with varying dimensions, column sizes, beam sections and Hollo-bolt classes. Based on the experimental results, the influence of the main connection parameters on key response characteristics is examined.

Chapter 4 describes another experimental investigation into the behaviour of open beam-to-tubular column connections incorporating channel/angle components. Ten monotonic and cyclic tests on specimens with different sizes, channel dimensions and geometric arrangements, are described including *top and seat* as well as *top, seat and web angle* details. Based on the experimental results, the main patterns of behaviour are discussed and the salient response characteristics such as stiffness, strength and energy dissipation are examined. Particular emphasis is given to the assessment of key detailing parameters such as channel thickness, angle gauge distance and the presence of web angles.

Chapter 5 deals with the analytical prediction of the monotonic and cyclic response of angle connections with blind-bolts and reverse channel components. A mechanical model, based on the component approach is proposed and a detailed description of the model assumptions, component characterisation, overall considerations and model validation is presented. The proposed mechanical model is then used to perform a parametric investigation into the key factors influencing the behaviour of Hollo-bolted connections. It is shown that the blind-bolt grade, angle thickness, column face slen-

1. PREAMBLE

derness and gauge distance have a significant influence on the connection behaviour. Based on these findings, simplified approaches for the estimation of the initial stiffness and moment capacity of blind-bolted connections are suggested. Failure mode considerations and simple expressions for ensuring the desired yield mechanism are also discussed. Besides, simplified mechanical models for determining the stiffness and capacity of combined channel/angle connections are proposed. This chapter also assesses the predictions of available fatigue damage models.

The Third Part of the thesis focuses on the behaviour of steel framed structures employing tubular members as columns and/or braces, including those incorporating the connection details examined in previous chapters. Part III starts with Chapter 6 that deals with the estimation of peak inelastic displacements in steel frames under constant relative strength scenarios. Mean inelastic deformation demands in bi-linear systems (simulating moment resisting frames) are considered as the basis for comparative purposes. Steel frames incorporating PR connections similar to those examined in the first part of the thesis are also studied through single-degree-of-freedom (SDOF) pinching models. Additionally, fibre-based models for concentrically-braced (CB) SDOF frames are also employed to assess the influence of different force-displacement relationships on peak inelastic displacement ratios. The studies presented in Chapter 6 illustrate that the ratio between the overall yield strength and the strength during pinching intervals is the main factor governing the inelastic deformations of PR models and leading to significant differences when compared against predictions based on bi-linear structures, especially in the short-period range. It is also shown that the response of CB systems can differ significantly from other pinching models when subjected to low or moderate levels of seismic demand, highlighting the necessity of employing dedicated models for studying the response of CB structures. Particular attention is also given to the influence of a number of scalar parameters that characterize the frequency content of the ground-motion on the estimated peak displacement ratios. The relative merits of using the average spectral period T_{aver} , mean period T_m , predominant period T_g , characteristic period T_c and smoothed spectral predominant period T_o of the earthquake ground-motion, are assessed. It is demonstrated that the predominant period, defined as the period at which the input energy is maximum throughout the period range, is the most suitable frequency content indicator for reducing the variability in displacement estimations. Finally, non-iterative equivalent linearization expressions based on the secant period and equivalent damping ratios are presented and verified for the prediction of peak deformation demands in steel structures.

Chapter 7, on the other hand, aims to demonstrate the applicability of response history analyses based on rigid-plastic models for the seismic assessment and design of steel buildings. The rigid-plastic force-deformation relationship as applied in steel moment resisting (MR) frames is re-examined and new rigid-plastic models are developed for CB frames and dual structural systems consisting of MR coupled with CB frames. It is demonstrated that such rigid-plastic models are able to predict global seismic demands with reasonable accuracy. It is also shown that the direct relationship that exists between peak displacement and plastic capacity of rigid-plastic oscillators can be used to define the level of seismic demand for a given performance target.

While Chapters 6 and 7 primarily deal with the fundamental behaviour and analytical assessment of steel frames, Chapter 8 (which forms Part IV of this thesis) is concerned with the influence of different framing layouts on the seismic performance of complete structural building systems. It examines the peak response of one-way, two-way and mixed framing configurations incorporating MR and CB lateral resisting systems under bi-directional earthquake loading. Extensive non-linear response history analyses over a number of idealized 3D structures are performed under a suite of earthquake records and particular attention is given to the influence of gravity frames on the overall building performance. Chapter 8 illustrates that secondary systems, which are normally designed to carry gravity loads only, can play a significant role in reducing both seismic losses and the probability of dynamic instability of the overall structure if the primary system consists of CB frames, whereas no significant improvement is observed for buildings with primary MR frames. It is also shown that by modifying the two-way configuration through the introduction of variable degrees of moment release at selected beam-to-column connections (e.g. by introducing connection details similar to those studied in Chapters 3 to 5), significant enhancements in the seismic response can be obtained.

In the final chapter, a summary of the main conclusions is drawn alongside recommendations for future research on the topics addressed in this thesis.

Chapter 2

Literature Review

2.1 Introduction

This chapter presents a brief literature survey of previous work which is of direct relevance to the current study. The main aim of this review is to set up the context for the contributions made in subsequent chapters. It would be unrealistic to provide a full review of all design and assessment issues pertaining to steel frames using tubular columns and/or braces. Instead, a research framework is proposed which organizes the topic under consideration according to three pre-defined levels of structural discretization: (i) connection level, (ii) frame level and, (iii) overall system interaction level. This broad, yet structured, approach is intentional. It is believed that a wide-reaching evaluation of design and assessment issues would help to recognise the key research needs and to provide an appropriate framework for identifying the proposed contributions. Furthermore, this broad approach aims to integrate different aspects of building design/assessment and to show the interdependence and complementarity of the different levels of structural analysis/design. In the following, previous research is reviewed and divided, as is this thesis, into three main sections: open beam-to-tubular column connections, seismic design and assessment of frames and behaviour of overall building systems.

2.2 Open beam-to-tubular column connections

Although connections between open beams and tubular columns have been studied by many investigators, to date most of the attention on their performance has fo-

2. LITERATURE REVIEW

cused on fully-rigid fully-welded connections [20, 82, 83, 87] for which suitable design guidance is also available [88]. Similarly, Eurocode 3 Part 1.8 [27] proposes rules for determining the resistance of fully welded tubular joints but lacks information on their semi-rigid/partial-strength counterparts.

Due to the practical and economical merits offered by bolted angle connections, the behaviour of open beam-to-open column joints have been extensively studied both experimentally and analytically [60, 85, 104, 138, 139]. Previous research [41, 46, 92] has demonstrated that, when properly designed, bolted semi-rigid connections are often able to provide similar or even more favourable performance, particularly under seismic loading, than their fully-rigid counterparts. The use of semi-rigid bolted forms also eliminates the potential problems associated with weld fracture in fully-rigid configurations.

In comparison with open columns, semi-rigid connections to tubular columns have received relatively less attention. White and Fang [155] performed tests on five different connection configurations including fin plates, T-stubs, through plates and angles, welded to the face of tubular columns. Significant variation of stiffness was observed and the influence of column width was highlighted. Dawe and Grondin [31] performed ten tests on seat angle connections shop-welded to the column face and site-bolted to the beam, based on which eight failure modes were identified. Maquoi et al. [106] evaluated the use of threaded studs welded to tubular columns and found the column face deformation to be the main contributor to the performance of the connection owing to the relatively thin sections employed. More recently, Neves [117] investigated the same connection configuration but with thicker column walls. It was found that the welding zone in the threaded studs was of critical importance and that the connection ductility was substantially reduced due to fracture of the stud.

The cost and inspection/maintenance implications associated with welding have motivated the development of other connection alternatives. Several investigations have been carried out in order to explore the possibility of using thermal drilling techniques such as the *flowdrill* process in tubular connections both at the local [9, 11] and joint [58, 59] levels. France et al. [59] performed tests on full and partial depth end-plates flowdrilled to tubular columns and bolted to open beams. In general, the thermal drilling technique was reported to be adequate for column thickness between 5 and 12.5 mm whereas, for thicker plates, conventional drilling techniques were suggested.

2. LITERATURE REVIEW

Nevertheless, practical issues seem to render the application of flowdrilling cumbersome and hamper its use more widely.

To provide more practical alternatives for blind-bolting in tubular members, a number of bolts with sleeves designed to expand inside the tube have been developed and tested [9, 12, 38, 72, 80, 81, 123]. Korol et al. [81] performed tests on five end-plate bolted beam to column connection assemblies using *Blind Oversized Mechanical Bolts (BOM)* and *High Strength blind-bolts (HSBB)* developed by Huck International Inc [72]. It was shown that the behaviour of open beam-to-tubular column connections using HSBB was similar to that using conventional bolts whereas a connection employing BOM bolts did not achieve equivalent levels of strength.

A simpler blind-bolt design is that proposed by Lindapter International [98] through the development of the *Hollo-bolt*. Despite its wide availability and ease of practical use, experimental studies on connections incorporating Hollo-bolts are very limited [13, 57]. Based on monotonic tests on three end-plate joints, France [57] compared the behaviour of blind-bolt and flowdrill bolts, and suggested that flowdrill systems can provide relatively higher stiffness and capacity. Barnett et al. [14, 15] performed a review of different blind-bolting options and carried out an experimental study on blind-bolted T-stubs and connections using Hollo-bolts. To improve the clamping mechanism, a modified blind-bolt, referred to as the Reverse Mechanism Hollo-Bolt (RMHB) was proposed. At present, information on the behaviour of blind-bolted connections is limited, particularly for angle connections, and there are no reported studies on their cyclic response.

Owing to its versatility and ease of use, the *Reverse Channel Component* is emerging as a detail with a potential for wider adoption in practice, offering a cost-effective and practical alternative for joining open beams to tubular columns. Reverse channel configurations incorporate a channel section which is shop-welded at the legs end to the face of the tubular column. The channel face is then connected on site to the open beam by means of any conventional bolted detail (e.g. end-plates, top, seat and/or web angles, T-stubs, etc.). Despite its potential, there is a dearth of experimental studies on connections incorporating reverse channel components, particularly under cyclic loading. Test results on the fire resistance of four end-plate reverse channel connections have been reported by Ding and Wang [35]. The behaviour was compared with other connection details between open beams and concrete filled tubular columns. It was

2. LITERATURE REVIEW

concluded that reverse channel connections appear to have the best combination of structural performance and construction cost among the different configurations considered. However, there is a need for further assessment and characterization of the behaviour of various forms of reverse channel connections under both monotonic and cyclic loading conditions.

Many studies have been carried out on the analytical modelling of semi-rigid connections incorporating conventional bolts. For example, Agerskov [1] and Yee and Melchers [156] contributed to some of the early studies on the well-established equivalent T-stub component model incorporated in Eurocode 3 [27]. Jaspart [76], Faella et al. [50], Swanson and Leon [145] and Lemonis and Gantes [91] proposed various multi-linear models which are able to produce good estimates of the complete force-displacement relationship for T-stub components. As for angle connections, Kishi and Chen [79] and Kishi et al. [78] developed parametric models for the prediction of moment-rotation curves of top and seat angle connections. De Stefano et al. [33] proposed a mechanical model for the prediction of the inelastic cyclic moment-rotation relationships of double-angle connections. Shen and Astaneh-Asl [139] developed a hysteretic model for top and seat angle specimens which distinguishes between the behaviour of "thin" and "thick" angles. Garlock et al. [60] also performed experimental and analytical studies on bolted angle components with emphasis on their hysteretic behaviour.

In contrast, analytical research on the response prediction of blind-bolted connections to tubular columns is scarce. Ghobarah et al. [63] suggested an analytical model for the evaluation of the initial stiffness and plastic moment capacity of blind-bolted *end-plate* connections between open beams and tubular columns. Silva et al. [141] presented an analytical model for the estimation of the column face stiffness in *end-plate* connections with concrete filled tubular column components; representative expressions were proposed based on finite element models as well as experimental results. However, available models on end-plate connections cannot be directly applied to blind-bolted connections with angles due to the significant influence of contact phenomena as well as the complex interactions between the blind-bolts, column face and angle components as observed clearly in experimental studies [45]. Accordingly, there is a need for a dedicated model that provides a faithful characterisation of the response of blind-bolted angle connections. Such model should also be capable of predicting the response under cyclic loading conditions.

The dearth of research studies on semi-rigid bolted angle connections for tubes is also reflected in the limited design guidance available [88, 157]. Yeomans [157] proposed some criteria for the failure of bolts and column face, whilst CIDCET Design Guide 9 [88] suggests that simple shear connections can be designed with available guidance for open columns although special attention needs to be given to the column face deformation. On the other hand, AISC [2] suggests minimum thickness according to the bolt type for angle connections. In contrast, there is no consideration of blind-bolted connections to tubular columns in the current European standards [27].

2.3 Seismic design and assessment of steel frames

Current earthquake performance-based design and assessment methodologies pay special attention to the reliable determination of structural displacements [5]. Although such seismic demands can be calculated through sophisticated non-linear response history analyses, their application in practical assessment is still hampered by the considerable time, costs and expertise they require. Therefore, there is a need for simplified yet reliable methods for the estimation of structural seismic demands. Moreover, although in many cases structures do not behave as single-degree-of-freedom (SDOF) systems, various studies have shown that equivalent SDOF models can provide the basis for the estimation of global demands in building structures [100, 112, 130]; accordingly, recent recommendations for the evaluation of maximum deformations in buildings are based on such equivalent SDOF representations [29, 56].

Numerous studies have used SDOF models to develop probabilistic estimations of peak inelastic displacements under several suites of ground-motions [17, 23, 114, 134, 135, 136, 152, 154]. Some of these studies have focused on the estimation of strength ratios for systems of known ductility, distinguishing between stiff and soft soils and providing relationships that can be useful in the design of new structures to attain specified target ductility levels [114, 135]. On the other hand, other studies have evaluated peak displacement demands for strength-defined structures and have provided relationships which are useful for the seismic assessment of existing buildings [134, 136]. More recently, Bozorgnia et al. [17] performed a detailed investigation on inelastic deformations in SDOF systems based on predictive equations formulated on the basis of a large database including 3122 records. This study verified the applicability of the "equal displacement rule" originally proposed by Newmark and Hall [119] and Veletsos

2. LITERATURE REVIEW

and Newmark [153], and assessed the influence of several earthquake parameters on the inelastic displacement ratios of elastic-perfectly plastic systems.

Most of the available studies offer statistical results based primarily on elastic-plastic SDOF systems, and those studies that include pinching behaviour incorporate simultaneously severe levels of strength deterioration typical of reinforced concrete structures [65, 116, 136, 143, 152]. Goda et al. [65] concluded that the effects of degradation plus pinching can be significant and must be considered through the use of period-dependent and hysteretic-dependent factors. In the latter study, analyses on three structures (with periods of 0.2, 1.0 and 2.0 seconds) were performed for different values of normalized yield strength and for three levels of coupled pinching-degradation. Such levels of concurrent pinching and deterioration differ notably from pinching hysteresis of typical partially-restrained (PR) or concentrically-braced (CB) systems where no significant deterioration in strength is evident up to considerably high levels of deformation demand [44, 45, 101]. One of the few studies that differentiate between the influence of degradation and pinching characteristics is that by Song and Pincheira [143]. They observed that the influence of monotonic and cyclic strength as well as stiffness degradation and pinching is important only for structures built on soft soils with virtually no effect observed for ground-motions recorded on rock or stiff soils. However, pinching loops were constrained to be origin-centred and only 12 records were used. Therefore, there is a clear need for a full characterization of the influence of different pinching ratios within the range typically observed in PR structures, which commonly do not exhibit simultaneous severe strength degradation [45, 101]. Similarly, the tension elongation and compression buckling of bracing members in CB structures induce several levels of pinching behaviour which are typically unaccompanied by major strength deterioration and can also induce characteristic dynamic effects which warrant specific consideration [44].

The influence of frequency content of the ground-motion has long been recognized as a crucial parameter for the accurate estimation of seismic demands [119, 153]. Although response or Fourier spectra fully characterize the frequency content of a ground-motion and are always illustrative during the design and assessment processes, the use of a single scalar parameter representative of the record frequency content can offer some practical advantages. For example, Chopra and Chintanapakdee [22] introduced the ratio of structural period (T) over the characteristic period of the ground-motion (T_c) to better characterize the difference in inelastic deformation ratios between near-fault

and far-fault ground-motions. Vidic et al. [154] proposed simplified expressions for the estimation of strength reduction factors utilizing a ground-motion dependent period referred to as T_1 (calculated as an approximation of T_c). Miranda [112, 113, 135, 136] recommended the use of the predominant period (T_g) in order to improve estimations of peak deformation demands on structures built on soft soils sites. Shimazaki and Sosen [140], Uan and Maarouf [148], Tiwari and Gupta [147] and Chakraborti and Gupta [21] also used the predominant period (T_g) to better characterize seismic deformation demands and studied the influence of various earthquake and site parameters on the displacement estimations. More recently, Kumar et al. [86] also emphasized the improved accuracy in the peak displacement estimation obtained if a frequency content indicator is included in the evaluation process, and proposed the use of the mean period (T_m) based on the detailed study of Rathje et al. [131]. To this end, Rathje et al. [131] developed empirical relationships for four frequency content parameters and encouraged the use of T_m as a robust representation of a strong ground-motion, particularly its long period frequency content, in light of the relation of T_m with the Fourier Amplitude Spectrum. Yet, a comparison of the relative merits of the use of one scalar frequency content parameter over others in the context of the prediction of seismic inelastic demands has not been carried out, particularly for PR and CB structures.

In general, simplified methodologies for the estimation of peak structural deformations have largely followed two approaches: (i) a seismic coefficient approach (where the response of a non-linear structure is obtained on the basis of the empirically modified response of an elastic systems with the same lateral stiffness and viscous damping [53]), or (ii) an equivalent linearization approach (in which the inelastic behaviour is accounted for by considering the response of an equivalent linear system with lower equivalent stiffness and higher viscous damping [4]). Owing to its versatility, the equivalent linearization approach also forms the basis of seismic assessment procedures in current European provisions [28, 29]. Furthermore, several equivalent linearization models have been proposed which can be grouped according to the definition of equivalent parameters employed. A first set of studies define the equivalent period as the period related to the secant stiffness at peak inelastic displacement and obtain the equivalent damping from energy balance relationships [68] or statistical analyses [39]. A second set of studies derive period and damping pairs through various mathematical optimization procedures [69, 89]. In most cases the proposed equivalent damping and period expressions are derived as functions of target ductility levels which are unknown when assessing the response of existing structures. Only Lin and Lin [97] and

2. LITERATURE REVIEW

Goda and Atkinson [64] propose equivalent linear systems with parameters defined in terms of strength ratios suitable for the evaluation of existing structures. Goda and Atkinson [64] present prediction equations of equivalent linear models based on optimal periods for non-degrading, degrading and degrading plus pinching structures. Lin and Lin [97] suggest equivalent linear models based on the secant period for bi-linear systems with varying strain hardening stiffness. However, equivalent linearized models for the estimation of inelastic structural response of PR and CB structures are lacking. Besides, the use of the secant period as the equivalent period in equivalent SDOF models should be considered, as it leads to maximum inelastic displacement and acceleration occurring at the intersection of the capacity and demand diagrams for a given equivalent damping, hence providing engineers with a direct graphical comparison tool.

Despite its common use in studying the post-elastic behaviour under static loads, rigid-plastic models have not been widely employed in dynamic problems mainly due to the lack of suitable computational procedures. Another perceived problem is related to the inability of the method to deal with resonance effects, although it should be noted that resonant response decays rapidly as soon as non-linear deformations start to occur. One of the first studies dealing with rigid-plastic approximations in the earthquake engineering field was the work by Newmark [118] who devised a method to estimate earthquake induced displacements on slope stability problems where relative displacements start to accumulate whenever the earthquake induced acceleration exceeds certain resistance thresholds. In fact, this viscous-like idealisation has become an important field of research in soil mechanics [48] and in the analysis of viscous dampers subjected to earthquake loading [150]. A few years later, Augusti [6] proposed general formulae for the solution of the dynamic response of framed structures based on rigid-plastic oscillators subjected to simple pulse and periodic loading. Strain hardening and strength degradation effects were partially considered in his study. Paglietti and Porcu [124], by means of the same classic rigid-plastic model, calculated the maximum displacement response of several SDOF systems to seismic loading. Their study also introduced the idea of the so called *rigid-plastic spectrum* defined as the relationship between a structural response parameter (such as peak structural displacement) and the plastic capacity of the rigid-plastic model. More recently, and based on such relationship, Domingues-Costa et al. [37] proposed an earthquake resistant design procedure for reinforced-concrete moment resisting frames modifying the classical rigid-plastic model to take into account pinching effects typical of reinforced-concrete structures. This study highlighted the relative merits of rigid-plastic approaches, which led to renewed

interest in exploiting its advantages, particularly in terms of simplified seismic design and assessment procedures.

2.4 Behaviour of framing configurations

Two main framing configurations have been traditionally used in Japan and Europe (as well as in North America). Japanese engineers often adopt a two-way framing layout in which the structure consists of 3D beam-column assemblages designed to resist the simultaneous action of seismic and gravity loads [146]. On the other hand, it is common practice in current design procedures within European and American seismic codes to differentiate between primary and secondary resisting systems, and to concentrate efforts on providing adequate lateral strength and ductility to the primary system. Nevertheless, secondary/gravity frames (which do not provide a significant contribution in terms of initial base shear/stiffness) typically exhibit large elastic deformations; hence if properly designed, they can significantly enhance the post/elastic response of the overall structure. Previous researchers [77, 127] have recognized such potential benefits in the course of examining residual displacements in buckling-restrained braced frames. Other researchers [3] have highlighted the significant contribution made by partially-restrained connections to the earthquake performance of hybrid systems incorporating viscoelastic bracings.

The overall behaviour of building structures has been extensively investigated, mainly in relation to the torsional response of asymmetric plan buildings [24, 34, 62, 66, 125]. Similarly, assessment procedures and simplified equivalent 3D models have been developed [25, 52, 96]. Also, the effects of earthquake intensity and frequency content on the response of plan asymmetric buildings has been studied [107, 126]. Moreover, the importance of considering bi-directional loading has been highlighted [30, 49, 90]. However, a comparative assessment of the behaviour of different framing configurations has not been effectively carried out.

Liao et al. [93] developed a 3D model of a three-storey moment resisting frame building with pre and post-Northridge connections to evaluate the effect of connection fracture, gravity frame contribution, panel zone flexibilities and column deformation on the building performance. As, expected, this study indicated that the building with pre-Northridge connections has a much higher probability of failure at all performance

2. LITERATURE REVIEW

levels. The same authors [94] also evaluated the response of nine three-storey and twelve-storey one-way moment resisting frame buildings with different plan layouts. It was found that the number of primary frames has a significant impact on the structural drift demands and that buildings with a small number of primary frames may not meet target performance criteria. Tagawa et al. [146] evaluated the seismic performance of one-way and two-way steel moment-frame structures through 3D inelastic dynamic analysis. It was shown that the two-way frame had a smaller mean annual probability of exceedence for drifts of less than 3% whereas larger mean annual probabilities of exceedence were observed for larger inter-storey drifts. Nevertheless, their study focused on buildings with constant steel volumes (rather than a range of comparable periods/stiffnesses) and the contribution of secondary frames was neglected. Besides, only moment resisting (MR) frames were considered and neither CB nor mixed framing systems were studied. More recently, Erduran and Ryan [49] evaluated the response of a three storey braced frame building with varying mass eccentricities under bi-directional loading. Storey drifts from bi-directional excitations were found to be much larger than those resulting from uni-directional earthquake action. Also, simplified analysis procedures seemed to be inadequate to capture storey drift amplification. Buildings with CB frames closer to the mass centre were noted to have the potential for significant drift reductions, although no analysis was performed to confirm this suggestion.

2.5 Summary

This chapter has reviewed previous research with the aim of identifying several research needs at three levels of structural discretization: (i) connection level, (ii) frame behaviour level and, (iii) overall building system level. From the review presented in this chapter, the conclusions listed below can be drawn, which in turn have motivated the corresponding contributions described in subsequent chapters of this thesis.

- There is extensive research available on fully-welded fully-rigid connections between open beams and tubular columns, and this research has often been translated into design guidance. Nevertheless, information and design guidance on more practical and cost-effective connection details is still lacking. Of particular interest (due to their versatility and easy of use) are blind-bolted connections and other semi-rigid connections incorporating channel components. The response of such connections is experimentally investigated in Chapters 3 and 4 while Chapter 5 is devoted to the development of suitable modelling procedures as well as

the calibration and validation of design procedures.

- Although extensive studies have been dedicated to the assessment of the response of steel frames under various earthquake scenarios, there is still a need for a full characterization of the influence of different pinching ratios within the range typically observed in partially-restrained structures such as those incorporating the connection details studied in Chapters 3 to 5. Similarly, tubular braced structures have specific behavioural characteristics that call for a more dedicated study. These issues are addressed in Chapter 6, where an in-depth study of the seismic response of MR, PR and CB steel structures is performed.
- Simplified design and assessment methodologies have been extensively validated for steel moment resisting frames, but equivalent linearized models for the estimation of the inelastic structural response of PR and CB structures are lacking. Besides, the use of the secant period as the equivalent period in equivalent SDOF idealizations has not been considered for this type of steel structures. Accordingly, new equivalent linearization procedures based on secant parameters are developed and validated in Chapter 6.
- The influence of the frequency content of the ground-motion has long been recognized as a crucial parameter for the accurate estimation of seismic demands; yet, a comparison of the relative merits of the use of one scalar frequency content parameter over others in the context of the prediction of seismic inelastic demands has not been carried out. This issue is addressed in Chapter 6, where a comparative assessment of the advantages of a number of scalar frequency content indicators is performed in terms of their influence on the response prediction.
- Despite its relative merits and common use in studying the post-elastic behaviour under static loads, rigid-plastic models have not been widely employed in dynamic problems, mainly due to the lack of suitable computational procedures. Chapter 7 is dedicated to developing such procedures and to the evaluation of the potential benefits of using rigid-plastic approximations in the seismic design and assessment of steel framed buildings.
- Extensive investigations on the seismic performance of complete building systems have been performed mainly in relation to the torsional response of asymmetric plan buildings. However, a comprehensive comparative assessment of the behaviour of different framing layouts over a range of comparable periods/stiffnesses

2. LITERATURE REVIEW

is yet to be carried out. These comparisons are explored in Chapter [8](#) with focus on the contribution of secondary systems to the inelastic response of full building configurations.

Part II

Connection Behaviour

Chapter 3

Experimental Response of Blind-bolted Angle Connections

3.1 Introduction

The literature review presented in the previous chapter (Section 2.2) has highlighted the relative lack of information and design guidance available on practical semi-rigid connection details between open beams and tubular columns. This lack of guidance seems to hinder the wider use of tubular construction hence the architectural and structural advantages of tubular columns remain underexploited. It is the aim of this part of the thesis (which comprises Chapters 3 to 5) to study the behaviour of practical and cost-effective semi-rigid open beam-to-tubular column connection alternatives. To this end, two details with the potential for a wider application in practice were identified in the review of the previous chapter: (i) Holo-bolted angle connections and (ii) combined channel/angle connections. This chapter deals with the experimental response of Holo-bolted angle configurations while the next chapter is concerned with reverse channel alternatives. The wide availability and ease of use (requiring only standard installation equipment) of Holo-bolted connection details is particularly attractive and has therefore motivated the research presented herein.

Subsequent sections of this chapter present and discuss the results of a series of tests performed in order to examine the behaviour of blind-bolted angle connections between open beams and tubular columns. *Top and seat* and *Top, seat and web* angle connections blind-bolted to structural hollow columns by means of Lindapter Holo-bolts [98] are examined. Apart from direct tension tests on blind-bolts, seventeen specimens

3. BLIND-BOLTED ANGLE CONNECTIONS

with three different connection configurations and varying dimensions, column sizes, beam sections and Hollo-bolt classes, were tested under monotonic and cyclic loading conditions. The arrangements used for bolt and connection tests are discussed and a detailed description of the specimens used is provided. The main experimental results are then presented and salient behavioural observations are highlighted. The discussion focuses on issues related to the stiffness, capacity and failure mechanisms, and their implications on the performance and design of this type of connection.

3.2 Testing arrangement and specimen details

3.2.1 Experimental set-up

The arrangement used for testing the beam-to-column connections is shown schematically in Figure 3.1a. In order to accommodate the large rotations expected in the connections a loading mechanism consisting of swivel hinges was constructed as indicated in Section A-A of Figure 3.1a. An hydraulic actuator operating in displacement control was used to apply vertical deformation at the tip of the cantilever open beam. On the other hand, the tubular column was restrained at both ends with the aid of another reaction frame.

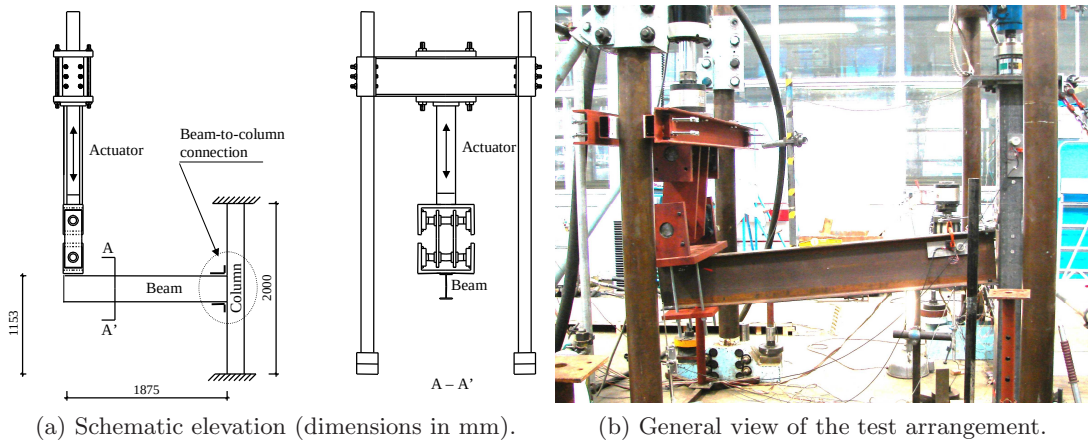


Figure 3.1: Experimental set-up.

Vertical displacements and corresponding forces were recorded by the load cell and displacement transducer incorporated within the actuator. The verticality of the column was monitored through an inclinometer attached to it, while another inclinometer was

3. BLIND-BOLTED ANGLE CONNECTIONS

placed on the beam to confirm the angle of rotation. Displacement transducers installed through small openings in the face of the tubular column opposite to the connection were used to measure the out-of-plane deformation of the face of the column at selected locations. Strain gauges were used to monitor the strains at the beam flanges and at the expected locations of plastic hinges within the angles.

3.2.2 Beam-to-column connection specimens

In total, seventeen open beam to tubular column semi-rigid Holo-bolted angle connections were tested. A summary of the test series is given in Table 3.1, which includes the geometric details of the connection as well as the beam and column sizes. Note that UB, RHS, SHS refer to Universal Beam, Rectangular Hollow Section and Square Hollow Sections, respectively. When RHS were used for columns, the beam was connected to the wider face. Figure 3.2 illustrates the three connection configurations utilised (A, B and C). The reference used for the specimens follows the format $Tt - Gx - dy - R$ where T represents the specimen type (A, B or C with reference to Figure 3.2), t is the column face thickness in mm, x is the Holo-bolt grade, y is the gauge distance between the centre of the Holo-bolt and the beam flange in mm, and R reflects the testing regime (M for monotonic and Y for cyclic).

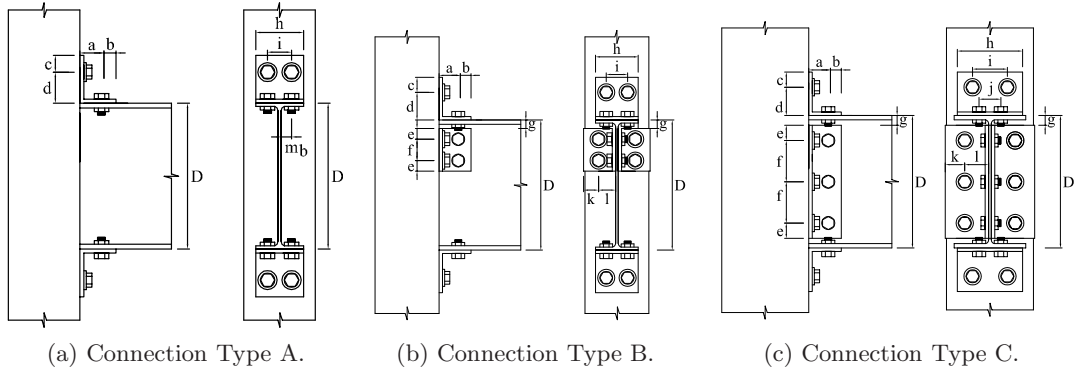


Figure 3.2: Details of connection configurations.

Table 3.1: Summary of test programme.

Reference	Connection configuration	Column section	Beam section	Hollo-bolt grade	Testing regime	Dimensions [mm] (see Figure 3.2)											
						a	b	c	d	e	f	g	h	i	j	k	l
A5.0-G8.8-d65-M	A	RHS 200x100x5	UB 305x102x25	8.8	Monotonic	45	30	35	65	-	-	-	100	45	45	-	-
A6.3-G8.8-d65-M	A	SHS 150x150x6.3	UB 305x102x25	8.8	Monotonic	45	30	35	65	-	-	-	100	45	45	-	-
A6.3-G8.8-d40-M	A	SHS 150x150x6.3	UB 305x102x25	8.8	Monotonic	50	50	35	40	-	-	-	100	45	45	-	-
A10-G8.8-d65-M	A	SHS 150x150x10	UB 305x102x25	8.8	Monotonic	45	30	35	65	-	-	-	100	45	45	-	-
A10-G8.8-d40-M	A	SHS 150x150x10	UB 305x102x25	8.8	Monotonic	50	50	35	40	-	-	-	100	45	45	-	-
A10-G10.9-d65-M	A	SHS 150x150x10	UB 305x102x25	10.9	Monotonic	45	30	35	65	-	-	-	100	45	45	-	-
A10-G10.9-d40-M	A	SHS 150x150x10	UB 305x102x25	10.9	Monotonic	50	50	35	40	-	-	-	100	45	45	-	-
A5.0-G8.8-d65-Y	A	RHS 200x100x5	UB 305x102x25	8.8	Cyclic	45	30	35	65	-	-	-	100	45	45	-	-
A6.3-G8.8-d65-Y	A	SHS 150x150x6.3	UB 305x102x25	8.8	Cyclic	45	30	35	65	-	-	-	100	45	45	-	-
A6.3-G8.8-d40-Y	A	SHS 150x150x6.3	UB 305x102x25	8.8	Cyclic	50	50	35	40	-	-	-	100	45	45	-	-
A10-G8.8-d65-Y	A	SHS 150x150x10	UB 305x102x25	8.8	Cyclic	45	30	35	65	-	-	-	100	45	45	-	-
A10-G10.9-d65-Y	A	SHS 150x150x10	UB 305x102x25	10.9	Cyclic	45	30	35	65	-	-	-	100	45	45	-	-
B10-G8.8-d40-M	B	SHS 150x150x10	UB 305x102x25	8.8	Monotonic	50	50	35	40	27.5	45	20	100	45	45	35	40
C10-G10.9-d65-M	C	SHS 200x200x10	UB 305x165x40	10.9	Monotonic	45	30	35	65	35	95	22.5	150	80	50	45	55
C10-G10.9-d40-M	C	SHS 150x150x10	UB 305x165x40	10.9	Monotonic	50	50	35	40	35	95	22.5	150	80	50	35	40
C10-G10.9-d65-Y	C	SHS 200x200x10	UB 305x165x40	10.9	Cyclic	45	30	35	65	35	95	22.5	150	80	50	45	55
C10-G10.9-d40-Y	C	SHS 150x150x10	UB 305x165x40	10.9	Cyclic	50	50	35	40	35	95	22.5	150	80	50	35	40

3. BLIND-BOLTED ANGLE CONNECTIONS

Grade 8.8 M16 standard bolts were employed to connect the angles to the beam. The grade of steel used for the beam and angle components was S275, whereas S355 was adopted for the columns. The average yield strength values obtained from coupon tests for the beam, angle and column components were 329, 312 and 400 N/mm^2 respectively, while average values of 443, 438 and 502 N/mm^2 were obtained for the ultimate strength of the beam, angle and column, respectively.

All beam-to-column connection tests were conducted under displacement-control conditions. In the ten monotonic tests, the displacement at the tip of the beam was increased gradually up to failure or until the stroke of the actuator was reached at 250 mm. For the cyclic tests, the beam was lowered in order to accommodate deformations of up to ± 125 mm in both directions. The cyclic testing protocol shown in Figure 3.3 was used based on the recommendations provided by ECCS [40], where Δ is the applied displacement and Δ_y is the estimated yield displacement.

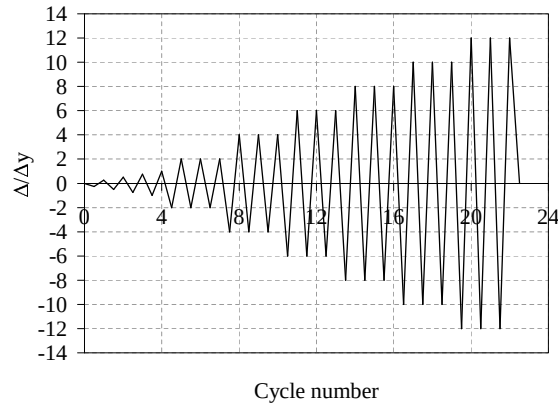


Figure 3.3: Loading protocol considered in the cyclic tests [40].

3.2.3 Characteristics of Hollo-bolts

3.2.3.1 General

The Hollo-bolt system developed by Lindapter [98] is employed in the present study as it represents a widely available configuration and the bolting mechanism does not require any special installation devices. In its typical form, the Hollo-bolt system utilizes Grade 8.8 bolts placed inside a special sleeve that spreads during tightening, hence clamping the connected plates together. A schematic view of the Hollo-bolt mechanism

3. BLIND-BOLTED ANGLE CONNECTIONS

is presented in Figure 3.4. As expected, the hole diameter required to accommodate the sleeve and tolerances is significantly larger than the bolt shank, in comparison with conventional bolts. This clearly has some implications on the arrangement and the number of bolts that can be used within a connection in practice.

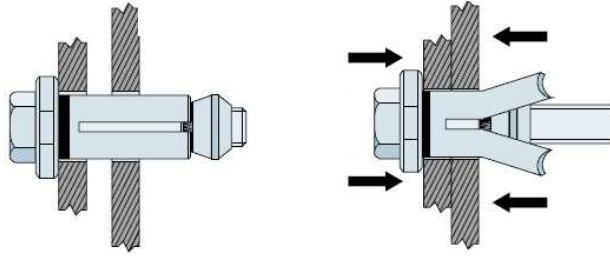


Figure 3.4: Lindapter Holo-bolt before (left) and after (right) installation [98].

The characteristics of typical Grade 8.8 Holo-bolt in direct shear and in direct tension have been examined by previous investigators [12, 15, 80, 123]. As expected, in direct tension, the failure mechanism is related to the thickness of the connected plates. For relatively thin plates, plastic deformations occur in the plates which can lead to pull-out of the bolt at a later stage. Otherwise, the typical failure mechanism is by yield followed by fracture of the fastener legs. Due to the direct relevance of tension behaviour of the bolts on the response of the angle connections considered in this chapter, it was decided to carry out tension tests on the bolts as described below.

3.2.3.2 Direct tension tests

Several tests were performed in order to characterise the tension behaviour of the typical Holo-bolts of Grade 8.8 as well as to explore the effects of higher pre-stressing forces allowed if Grade 10.9 bolts were used instead. Direct tension tests were carried out by employing the testing arrangement depicted in Figure 3.5. As shown in the figure, the Holo-bolt was used to connect two plates that were then pulled apart through an actuator operating in displacement-control. Results obtained for both M16 and M12 bolts are presented herein for information, although only M16 bolts were utilised in the connection tests described later in this chapter. The sizes of the clearance holes were as specified by the manufacturer, namely 21 mm for the M12 bolts and 28 mm for the M16 bolts. A tightening torque of 190 Nm was applied to the Grade 8.8 M16 Holo-bolt while 244 Nm was applied to the Grade 10.9 M16 bolts. For the M12 Holo-bolts,

3. BLIND-BOLTED ANGLE CONNECTIONS

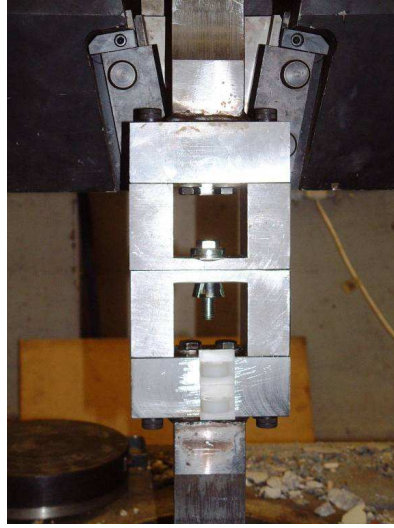


Figure 3.5: Set-up for direct tension tests on Holo-bolts.

torques of 80 Nm and 98 Nm were applied for the Grade 8.8 and 10.9 bolts, respectively.

Figure 3.6a depicts the typical axial load versus displacement response from the tests performed on M16 Holo-bolts. The thickness of the connected plate was at least 10 mm to ensure that the tests represented the bolt response. As shown in the figure, the initial stiffness is only maintained up to a load of about 35 kN, followed by considerable post-yield plateau with increasing deformation, with the ultimate load and deformation reaching nearly 120 kN and 9 mm, respectively. Gradual separation of the connected plates was related mainly to deformation of the sleeve, with the final failure occurring by crushing and fracture of the sleeve legs (see Figure 3.7).

In order to explore the influence of bolt shank grade on the performance, Grade 10.9 specimens were also tested as mentioned before and shown in Figure 3.6a for M16 bolts. Clearly, in this case the elastic stiffness is maintained to a much higher load (about 80 kN) without a notable change in the ultimate strength. However, the failure displacement reduces to about 6mm. Importantly, the ratio between the shank capacity and the elastic limit reduces from about 3.8 for Grade 8.8 bolts to approximately 2.2 in the case of Grade 10.9 bolts; this is directly attributed to the higher initial tightening torque allowed in the 10.9 bolts as noted before. The same trends discussed above, including the influence of bolt grade, are also reflected in tests carried out on the M12 Holo-bolts, as indicated in Figure 3.6b; again, as a result of the higher torques

3. BLIND-BOLTED ANGLE CONNECTIONS

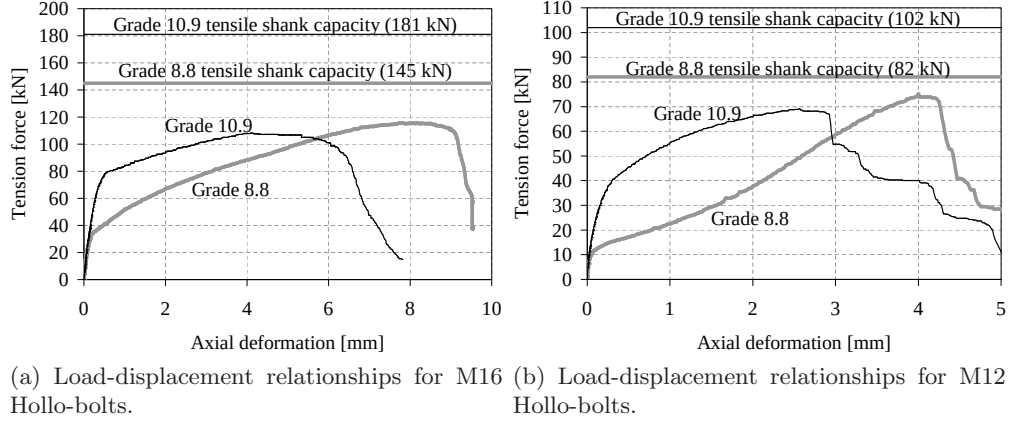


Figure 3.6: Hollo-bolt load-axial displacement relationships.

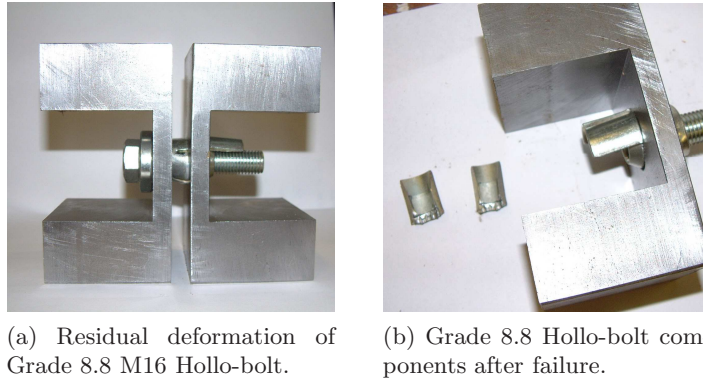


Figure 3.7: Deformation and fracture of Grade 8.8 Hollo-bolt after failure.

applied in the Grade 10.9 bolts, the ratio between the shank capacity and the elastic limit reduces from about 5.6 for Grade 8.8 bolts to roughly 2.5 in the case of Grade 10.9 bolts. The above-described behaviour of Hollo-bolts has a direct influence on the response of angle connections, as discussed in subsequent sections.

3.3 Experimental results and observations

The main response parameters obtained from the tests are summarised in Tables 3.2 and 3.3, and the key results are presented in Figures 3.8 to 3.14. The initial stiffness and moment values at global yield were obtained from a bi-linear idealisation of the moment-rotation relationships. Due to the significant post-yield stiffness, values of moment at a joint rotation of 40 mrad are also given in the tables. On the other hand, for the cyclic tests, the hysteretic energy is defined as the summation of the areas en-

3. BLIND-BOLTED ANGLE CONNECTIONS

Table 3.2: Summary of results for monotonic tests.

Reference	Initial stiffness [kNm/rad]	Moment at yield [kNm]	Moment [kNm] at 40 mrad rotation
A5.0-G8.8-d65-M	455	6.1	7.9
A6.3-G8.8-d65-M	989	9.3	12.9
A6.3-G8.8-d40-M	1414	14.0	18.7
A10-G8.8-d65-M	1448	9.7	15.0
A10-G8.8-d40-M	1403	18.8	22.7
A10-G10.9-d65-M	1642	11.0	16.0
A10-G10.9-d40-M	3205	25.0	31.8
B10-G8.8-d40-M	1650	19.8	36.4
C10-G10.9-d65-M	3243	24.0	38.8
C10-G10.9-d40-M	4169	49.2	58.5

Table 3.3: Summary of results for cyclic tests.

Reference	Initial stiffness [kNm/rad]	Moment at yield [kNm]	Moment [kNm] at 40 mrad rotation	Rate of energy dissipation [kJ/rad]
A5.0-G8.8-d65-Y	457.1	6.4	9.1	3.86
A6.3-G8.8-d65-Y	879.3	10.2	14.8	5.55
A6.3-G8.8-d40-Y	1391.8	13.5	18.5	7.71
A10-G8.8-d65-Y	1596.8	9.9	15.2	7.58
A10-G10.9-d65-Y	1794.1	12.2	18.0	8.14
C10-G10.9-d65-Y	3263.9	23.5	44.2	16.96
C10-G10.9-d40-Y	3598.0	36.7	64.0	20.31

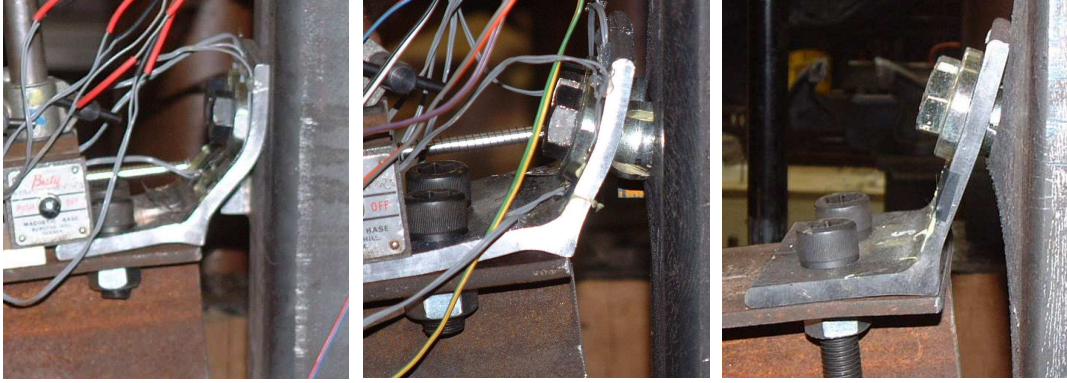
closed by the moment-rotation curves. In addition to the measured initial stiffness and yield moment, Table 3.3 includes the hysteretic energy dissipated per unit rotation. In subsequent sections, the experimental results and observations from the 17 connection tests, summarised in Tables 3.1 to 3.3, are presented. Particular focus is given in the discussions to the influence of the following parameters on the connection behaviour: (1) bolt grade, (2) gauge distance, (3) column thickness, (4) beam width, and (5) presence of web angles.

3.3.1 Monotonic tests

3.3.1.1 Deformation patterns

Before presenting the results and observations from the individual tests, it is perhaps more useful to start by highlighting the key inelastic modes exhibited by the angle/bolt assembly as these provide an overall context within which the findings can be viewed. Three main angle deformation patterns were identified in the present experimental

3. BLIND-BOLTED ANGLE CONNECTIONS



(a) Specimen A10-G10.9-d65-M. (b) Specimen A10-G10.9-d40-M. (c) Specimen A5.0-G8.8-d65-M.

Figure 3.8: Deformation patterns observed at the end of tests.

study, with an example of each of them depicted in Figure 3.8. As expected, the interaction between the blind-bolts and the angle had a direct influence on the inelastic behaviour of the connection. Angles with the longer leg connected to the column through Grade 10.9 Hollo-bolts typically yielded prior to any significant deformation in the bolt (Case (a) in Figure 3.8). When the angle was stiffer (i.e. shorter leg connected to the column face), significant plastic deformation was observed in the Hollo-bolt (Case (b) in Figure 3.8) which in the case of Specimen A10-G10.9-d40-M led to eventual failure of the connection by pulling out of the bolt (but at a rotation of over 100 mrad). Finally, when relatively thin column faces were used, deformations accumulated in the column-face/Hollo-bolt assemblage and the angle yielded near the root radius in the vertical leg only while the horizontal leg remained virtually straight (Case (c) in Figure 3.8). In general, all specimens exhibited high rotation capacities reaching deformations which are well beyond those typically required or implied by building codes, without significant degradation in capacity and or post-yield stiffness. The maximum moment values were several times higher than the yield moment capacities in the connection. Such high post-yield capacities were the result of both material strain hardening coupled with membrane effects in the angle components at large displacements.

The development of plasticity in the top angle can be illustrated through Figure 3.9 in which selected strain measurements from Specimens C10-G10.9-d65-M and C10-G10.9-d40-M, are presented as an example. When a gauge distance of 65 mm between the Hollo-bolt and the beam flange was used, plastic hinges formed at a similar rotation level in the vertical leg and near the corner within the horizontal leg. The zone between the bolts connected to the beam remained elastic. On the other hand, when a stiffer

3. BLIND-BOLTED ANGLE CONNECTIONS

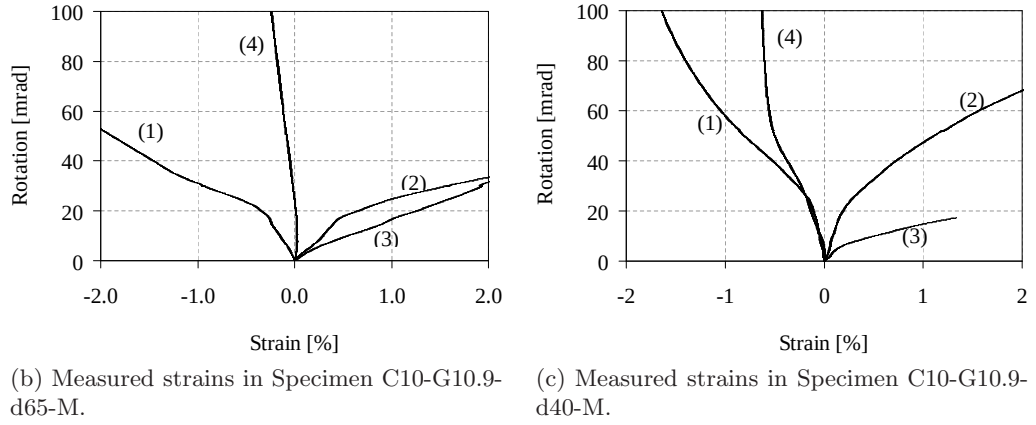
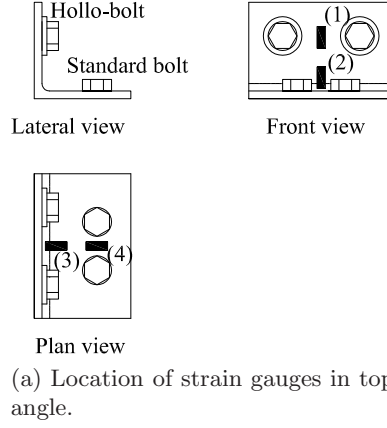


Figure 3.9: Example of strain measurements.

vertical leg was used (gauge distance of 40 mm between the Hollo-bolt and the beam flange), yield strains were first reached near the toe in the horizontal leg. As loading continued, plastic hinges started to form in the vertical leg of the angle until yielding of the Hollo-bolt was reached. Thereafter, significant deformations in the blind-bolts prevented the accumulation of additional strains in the angle near the bolt zone.

In light of the deformation patterns and inelastic modes highlighted above, subsequent sections present the moment-rotation results obtained from the monotonic connection tests. Each sub-section focuses on the influence of specific geometric or material parameters.

3. BLIND-BOLTED ANGLE CONNECTIONS

3.3.1.2 Gauge distance

Figure 3.10a presents the moment-rotation relationships obtained for four top and seat angle connections, namely A10-G10.9-d40-M, A10-G8.8-d40-M, A10-G10.9-d65-M and A10-G8.8-d65-M, whereas Figure 3.10b presents the results of A6.3-G8.8-d40-M, A6.3-G8.8-d65-M and A5.0-G8.8-d65-M. As noted before, the influence of angle stiffness (examined here by varying the angle gauge distance) and Hollo-bolt stiffness (considered here by varying the blind-bolt grade) are strongly interrelated. Reducing the distance between the blind-bolts and the beam flange generally increased the connection stiffness and capacity as indicated in Figures 3.10a and 3.10b. This can be observed by comparing A10-G8.8-d65-M with A10-G8.8-d40-M, A10-G10.9-d65-M with A10-G10.9-d40-M, and A6.3-G8.8-d65-M with A6.3-G8.8-d40-M. A change from 65 mm to 40 mm in the angle gauge distance increased the stiffness by about 25% on average when Grade 8.8 Hollo-bolts were employed whilst the capacity increased approximately 35% on average. When Grade 10.9 bolts were used, the enhancement in the overall connection stiffness was of the order of 70 % (Specimens A10-G10.9-d65-M and A10-G10.9-d40-M in Figure 3.10a).

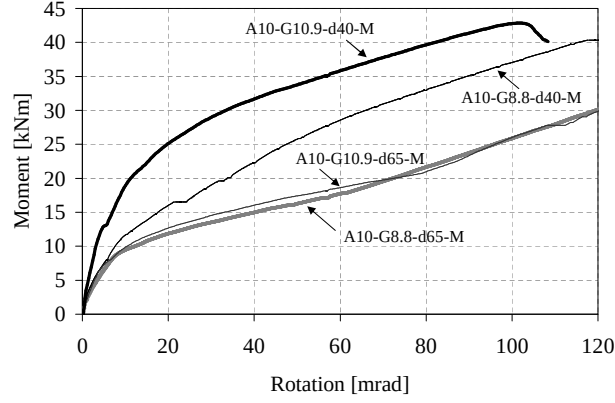
3.3.1.3 Hollo-bolt Grade

As illustrated in Figure 3.10a, the importance of the blind-bolt grade depends on the inelastic deformation pattern achieved. For A10-G8.8-d65-M and A10-G10.9-d65-M, the angle develops significant yielding and the bolt grade does not have a notable influence on the response in this case. On the other hand, for Tests A10-G8.8-d40-M and A10-G10.9-d40-M (also shown in Figure 3.10a), due to the relatively higher stiffness and capacity of the angle in this orientation, the blind-bolts play a direct role in the response. In this case, using higher grade bolts nearly doubles the stiffness and capacity of the connection.

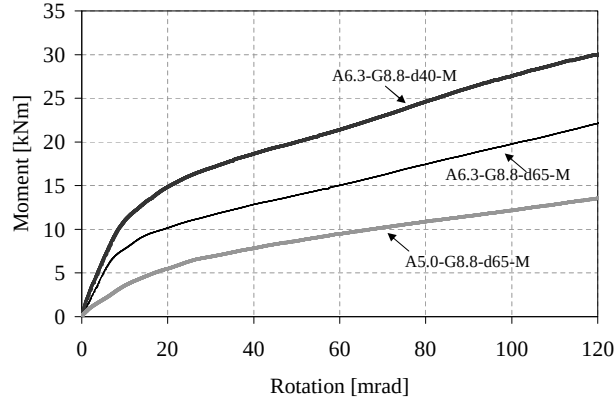
3.3.1.4 Column thickness

As expected, the stiffness of the column face has a direct influence on the response. This can be observed from Figures 3.10a and 3.10b by comparing the curves for A5.0-G8.8-d65-M, A6.3-G8.8-d65-M and A10-G8.8-d65-M in which the only parameter varied is the column face stiffness (through the column size and wall thickness as indicated in Table 3.1). The influence on the connection stiffness and capacity is evident in the

3. BLIND-BOLTED ANGLE CONNECTIONS



(a) Specimens A10-G10.9-d40-M, A10-G8.8-d40-M, A10-G10.9-d65-M and A10-G8.8-d65-M.



(b) Specimens A6.3-G8.8-d40-M, A6.3-G8.8-d65-M and A5.0-G8.8-d65-M.

Figure 3.10: Moment-rotation relationships for top and seat angle connections.

figures as well as in the summary of the results presented in Table 3.2. As expected, the reduction in column wall thickness results in lower overall stiffness and capacity for the connection.

3.3.1.5 Web angles

Figure 3.11 presents the experimental moment-rotation results obtained from the monotonic tests on connections with top, seat and web angles (i.e. Types B and C in Figure 3.2). The inclusion of web angles was found to be effective in increasing the connection capacity and stiffness provided local plasticity of the column face was prevented, otherwise the relative enhancement in stiffness was only marginal. This can be observed by comparing the moment-rotation curves of Specimens B10-G8.8-d40-M

3. BLIND-BOLTED ANGLE CONNECTIONS

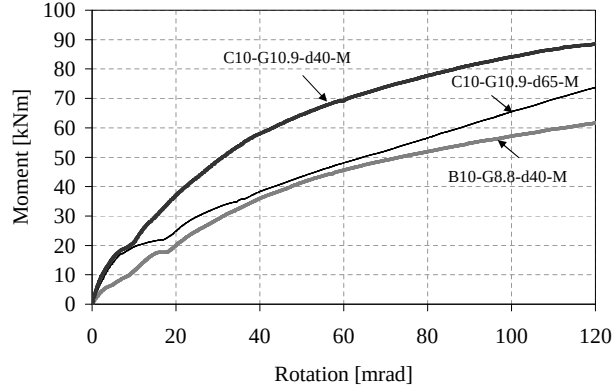


Figure 3.11: Moment-rotation relationships for top, seat and web angle connections.

(Figure 3.11) and A10-G8.8-d40-M (Figure 3.10a). The inclusion of web angles in the upper part of the web in Specimen B10-G8.8-d40-M lead to nearly doubling of the yield moment whilst the stiffness was not significantly enhanced.

The other two specimens with web angles (C10-G10.9-d40-M and C10-G10.9-d65-M, of Type C shown in Figure 3.2) achieved relatively high stiffness and capacity. This is illustrated in Figure 3.11 and Table 3.2. It should be noted that slippage occurred in the standard bolts connecting the angles to the beam flanges at a moment of about 20 kNm, as evident in the plots.

In addition to the contribution of the web angles, another reason for the relatively high initial stiffness obtained in these tests is the use of beams of width larger than the effective width of the column face. The effective width of the column face refers to the flat part of the SHS (i.e without the curved corners). This aspect is discussed further in the following section.

3.3.1.6 Beam width

Apart from the direct influence of the column face geometry on the behaviour on the tension side of the connection, it can also play an important role on the compression side. Unless the beam is at least as wide as the effective column face width, the contact forces in compression can result in local deformations in the column face which in turn contributes to the overall flexibility of the connection. It should be noted however that the effect of the flexibility of the column face in compression is only significant when

3. BLIND-BOLTED ANGLE CONNECTIONS



Figure 3.12: Column face deformation in the compression zone of Specimen B10-G8.8-d40-M.

relatively high forces are able to develop in the connection, even in cases where the column is wider than the connected beam.

An illustration of the localised deformations that can occur in the column face on the compression side is shown in Figure 3.12 for Specimen B10-G8.8-d40-M in which the column was wider than the beam. In general, it is clear that this aspect of behaviour is important and should either be designed out by using a sufficiently wide beam, or by incorporating its effect in the determination of the stiffness and capacity of the connection.

3.3.2 Cyclic behaviour

3.3.2.1 Hysteretic response

The cyclic response of top and seat angle connections is presented in Figure 3.13 while Figure 3.14 shows the moment rotation curves for specimens with top, seat and web angles. It is clear from Figures 3.13 and 3.14 that all specimens exhibited stable hysteretic behaviour with gradual transition between elastic and inelastic response. Furthermore, the moments developed were consistent with those observed in the corresponding monotonic tests in all cases. As shown in Figures 3.13 and 3.14, the reloading branch is different for each cycle but always reaches the same point at which the previous cycle unloaded. On the other hand, the unloading branches are largely parallel to each other regardless of the rotation magnitude. With increasing plastic deformations in the angles and Holo-bolts, the conditions change in subsequent cycles and residual defor-

3. BLIND-BOLTED ANGLE CONNECTIONS

mations are developed. This in turn leads to flattening of the shape of the hysteresis loops during the reversal of loading, due to the reduction in the resistance of the angle. Nevertheless, increase in stiffness occurs when contact with the column is reintroduced at the compression side. The slippage in the standard bolts connected to the beam contributes to this phenomenon. As expected, this pinching behaviour is more evident when higher capacities are reached like in the case of Specimen C10-G10.9-d40-Y in Figure 3.14b. In all cases, rotations exceeding ± 50 mrad were reached without any sign of failure. As shown in Figure 3.15, in only one test (A10-G10.9-d65-Y), fracture due to low-cycle fatigue occurred at the very last set of cycles at maximum amplitude; this low-cycle fatigue fracture appeared at the base of the vertical leg of the top angle.

3.3.2.2 Ductility and energy dissipation

In general, the blind-bolted connections examined in this chapter exhibited high ductility under monotonic and cyclic loading conditions. Depending on angle and bolt properties, some specimens experienced failure by bolt pull-out under monotonic loading (as shown in Figure 3.8b). This occurred in Tests A6.3-G8.8-d40-M, A10-G8.8-d40-M and A10-G10.9-d40-M, but typically at rotation levels which are well beyond those expected under characteristic loading conditions. Nevertheless, this type of behaviour is clearly undesirable. A more favourable mechanism is that involving full plasticity in the vertical and horizontal legs of the angles and/or column face such as that shown in Figure 3.8a.

An assessment of the energy dissipation in the cyclic tests is presented in Figure 3.16. The curves depict the cumulative dissipated energy versus cumulative rotation, whilst the rate of energy dissipated per unit rotation is also given in Table 3.3. As expected, there is a direct relationship between the stiffness/capacity of the connection and the energy dissipation capabilities. Accordingly, the connections incorporating web angles exhibited comparatively high levels of energy dissipation.

3.4 Concluding Remarks

The behaviour of blind-bolted angle connections between tubular columns and open beams was examined in this chapter. An experimental programme comprising seventeen monotonic and cyclic tests was described. The tests enabled a direct assessment

3. BLIND-BOLTED ANGLE CONNECTIONS

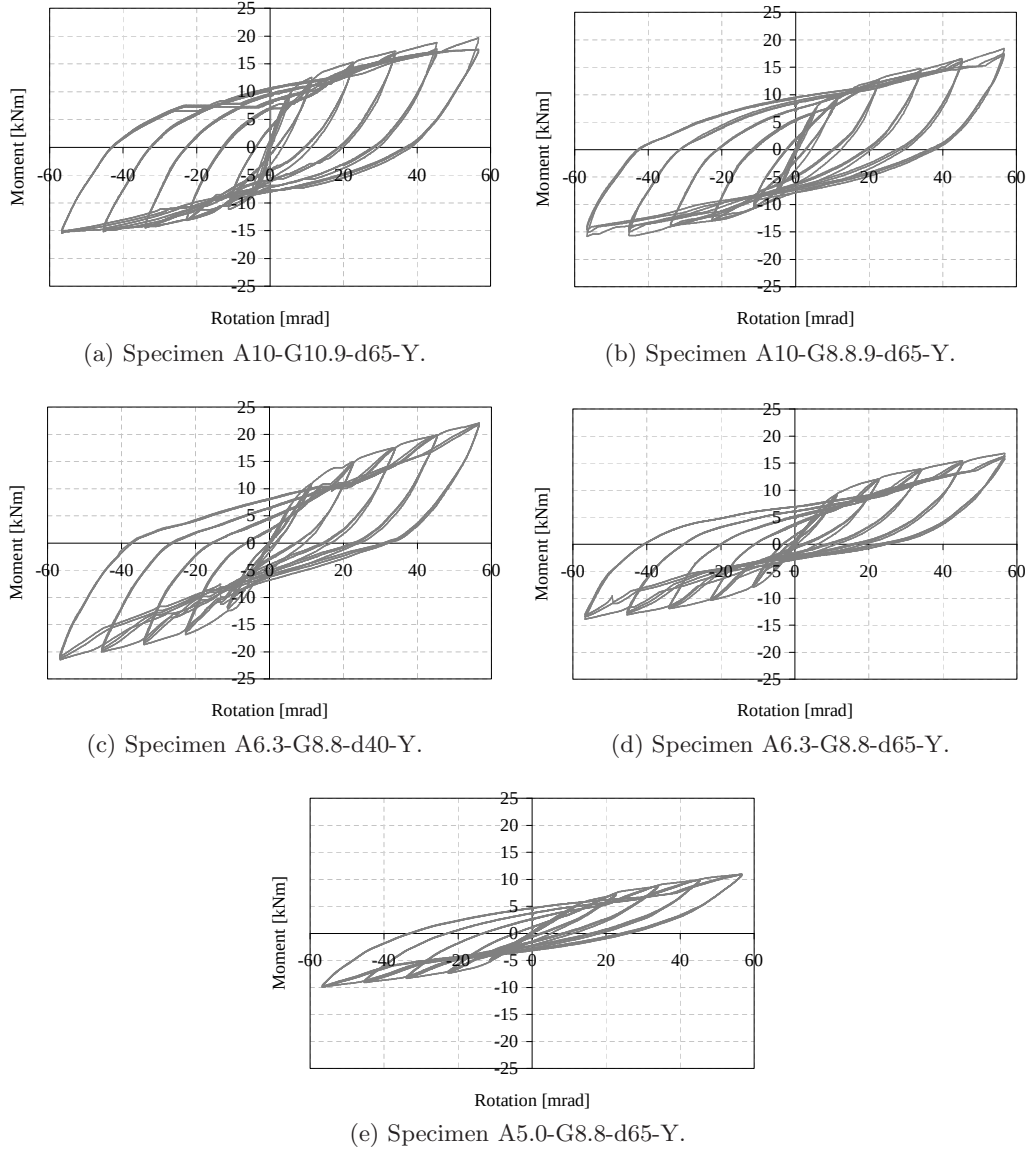


Figure 3.13: Moment-rotation hysteresis for top and seat angle connections.

of the influence of several salient connection properties and geometric parameters on the behaviour. The results also provide necessary information for the validation and calibration of the analytical models described in Chapter 5.

The three main inelastic mechanisms exhibited by this type of connection were identified in the tests. As expected, these mechanisms primarily involve interaction between the angle components and the Hollo-bolt/column face assemblage. Importantly, it was

3. BLIND-BOLTED ANGLE CONNECTIONS

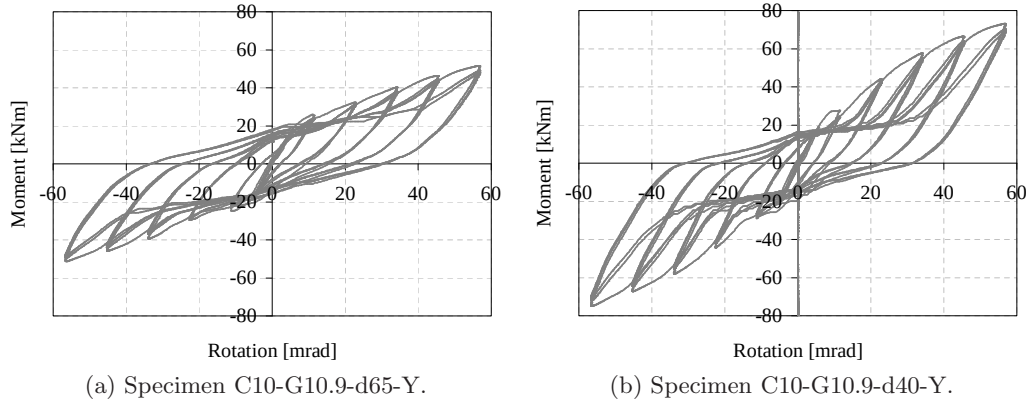


Figure 3.14: Moment-rotation hysteresis for top, seat and web angle connections.

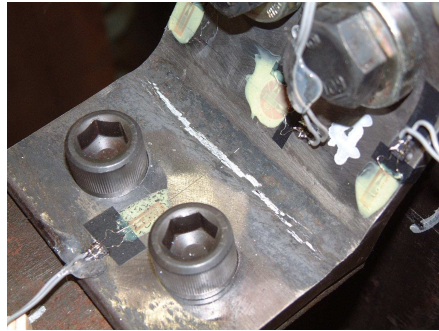


Figure 3.15: Fracture in top angle due to low-cycle fatigue in Specimen A10-G10.9-d65-Y.

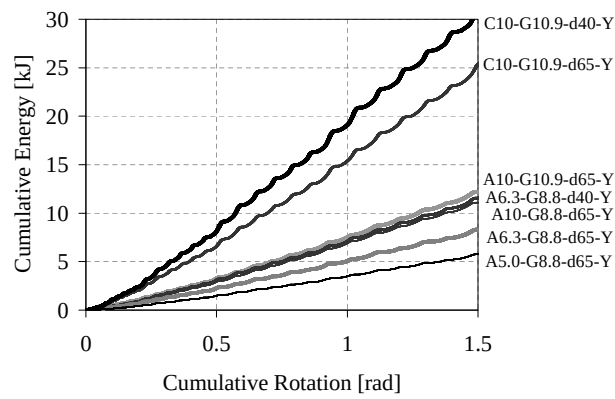


Figure 3.16: Cumulative energy dissipation.

3. BLIND-BOLTED ANGLE CONNECTIONS

shown that the grade of the Holo-bolt, coupled with the gauge distance between the Holo-bolt and the beam flange, have a most notable effect on the performance. As illustrated in direct tension tests on the bolts, the higher torque permitted in Grade 10.9 Holo-bolts results in significant improvement in its axial stiffness and capacity in comparison with Grade 8.8, at the expense of some reduction in local ductility. This is directly reflected in a more favourable performance on the overall connection level, depending on the relative stiffness of other connection components.

The tests showed that the extension induced within the blind-bolts (which is directly related to prying action and component separation), as well as the bending deformations within the column face, should be limited in design in order to satisfy serviceability requirements. The tests also demonstrated the importance of ensuring the presence of an adequate mechanism for transferring the compression forces to the column, in order to achieve satisfactory performance. These forces become more pronounced when web angles are incorporated in the connection due to the enhanced connection capacity.

In terms of ductility, connections with or without web angles provided rotational capacity in excess of 120 mrad under monotonic loading and more than 60 mrad under cyclic loading, which are well beyond those required under typical design scenarios. The connections generally exhibited stable hysteretic response and provided reasonable energy dissipation capabilities, with notable pinching behaviour in some tests. The ductility was typically limited by the stroke of the actuator rather than failure of constituent components. The only exceptions were the monotonic test A10-G10.9-d40-M, which involved blind-bolt failure at about 100 mrad, and the cyclic test A10-G10.9-d65-Y, in which angle fracture occurred in the last cycle at around 60 mrad. Although these limiting phenomena took place at excessive levels of rotation, there is still a need for their consideration within analytical and design studies; these issues are addressed in Chapter 5.

Overall, the experimental findings of this chapter indicate the suitability of Holo-bolted angle connections for secondary or primary frame systems, depending on the specific structural configuration and loading conditions under consideration. To this end, Chapter 8 examines the influence of different structural arrangements as well as the effects of secondary/primary frame interaction for framed structures that employ this type of Holo-bolted semi-rigid partially-restrained angle connections.

Chapter 4

Experimental Response of Combined Channel/Angle Connections

4.1 Introduction

The previous chapter has studied the experimental monotonic and cyclic response of one practical connection alternative between open beams and tubular members, namely, Holo-bolted angle connections. Another alternative for open beam-to-tubular column angle connections is that offered by the use of *Reverse Channel Components*. These configurations incorporate a channel section which is shop-welded at the legs end to the face of the tubular column. The channel face is then connected on site to the open beam by means of any conventional bolted detail (e.g. end-plates, top, seat and/or web angles, T-stubs, etc.). Owing to its versatility and ease of use, the Combined Channel/Angle Connection is emerging as a detail with a potential for wider adoption in practice. Nevertheless, as mentioned in the literature survey of Chapter 2, despite its potential there is a dearth of experimental studies on connections incorporating reverse channel components, particularly under cyclic loading. Ding and Wang [35] reported fire test results on four end-plate reverse channel connections and compared their behaviour with that of other connection details between open beams and concrete filled tubular columns. It was concluded that reverse channel connections usually have the best combination of structural performance and construction cost among the different configurations considered. However, there is a need for further assessment and characterization of the behaviour of various forms of reverse channel connections under both

4. COMBINED CHANNEL/ANGLE CONNECTIONS

monotonic and cyclic loading conditions.

This chapter examines the experimental response of reverse channel connections with bolted angles. It presents and discusses the results of monotonic and cyclic tests on ten *Top and seat* as well as *Top, seat and web angle* connections with different geometric configurations. A detailed description of the testing arrangement and material properties is given, and the main experimental results and salient behavioural observations are discussed. The discussion focuses on issues related to stiffness, capacity and failure mechanisms including fracture due to low-cycle fatigue. The implications of the findings on the performance and design of this type of connection are highlighted.

4.2 Experimental set-up and details

4.2.1 Testing arrangement

The same arrangement used for testing blind-bolted connections and described in the previous chapter (Section 3.2.1), as depicted in Figure 3.1, was also used for testing the reverse channel configurations. Vertical displacements and forces were recorded by the load cell and displacement transducer incorporated within the actuator at the tip of the beam. The verticality of the column was monitored through an inclinometer attached to it, while another inclinometer was placed on the beam to confirm the angle of rotation. Displacement transducers were used to measure the deformation at the face of the reverse channel component and at selected points along the beam length to monitor its deformation. Strain gauges were also used to measure strains at the beam flanges, at the expected locations of plastic hinges within the angles, and in the inner face of the reverse channel component at the midpoint between the top angle bolts.

4.2.2 Connection specimens

In total, ten open beam-to-tubular column semi-rigid reverse channel connections with angles were tested. Previous experimental work with reverse channel connections utilised conventional channel sections and bolted end-plates [35]. In the present study, the reverse channel components were obtained by longitudinal cutting of hot rolled tubular sections. This is believed to enhance the versatility and practicality of this type of connection due to the wider range available in tubular sections, hence offering a broader selection of width-to-thickness ratios than equivalent standard channel sections. After welding the legs of the reverse channel component to the face of the

4. COMBINED CHANNEL/ANGLE CONNECTIONS

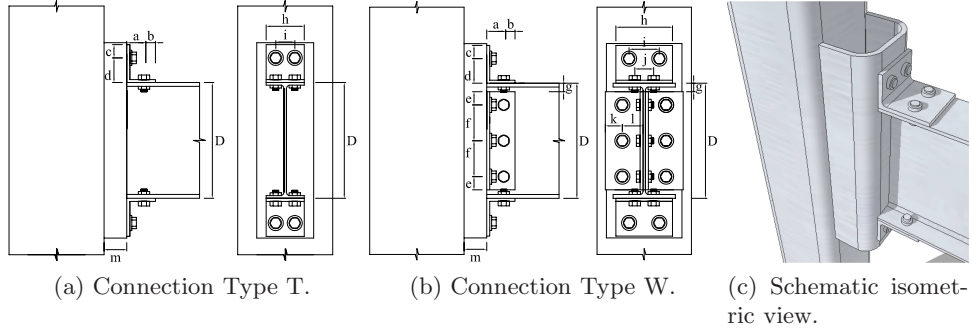


Figure 4.1: Details of connection configurations.

tubular columns, the open beam was connected to it by means of bolted angle cleats.

A summary of the test series is given in Table 4.1, which includes the geometric details of the connection as well as the beam and column sizes. UB and SHS refer to Universal Beam and Square Hollow Sections, respectively. It should be noted that the channel was obtained by cutting the indicated SHS. Figure 4.1 illustrates the connection configurations utilised. The reference used for the specimens follows the format $Dt - Ax - dy - R$ where D represents the specimen detail (for which T stands for top and seat angles and W for top, seat and web angle connections as indicated in Figure 4.1), t is the thickness of the reverse channel in mm, x is the thickness of the angle components in mm, y is the gauge distance measured between the centre of the bolt at the channel component and the beam flange in mm, and R reflects the testing regime (M for monotonic and Y for cyclic). SHS 200x200x10 members of 2 m length were used in all cases as columns.

Grade 10.9 M16 standard bolts were employed in all specimens. Fillet welding with thickness of 10 mm was used throughout the length of the external face of the channel legs, and for a depth extending beyond the height of the top and seat angles. The average material yield strength and ultimate strength for the different components used (beams, columns, reverse channels and angle components) were obtained from at least three coupon tests and are presented in Table 4.2.

All beam-to-column connection tests were conducted under displacement-control conditions. In the monotonic tests, the displacement at the tip of the beam was increased gradually up to failure or until the stroke of the actuator was reached at 250 mm. For the cyclic tests, the beam was lowered in order to accommodate deformations of up to ± 125 mm in both directions. As for blind-bolted connections, the cyclic testing

4. COMBINED CHANNEL/ANGLE CONNECTIONS

protocol shown in Figure 3.3 was also used for reverse channel connections based on the recommendations provided by ECCS [40]. After the maximum displacement was reached, all cyclic test specimens except Specimen T10-A8-d40-Y were subjected to additional displacement reversals at an amplitude of $\pm 12\Delta_y$ up to the point when fatigue fracture was observed in any of the connection components. For Specimen T10-A8-d40-Y, due to experimental constraints, additional displacement cycles at $\pm 10\Delta_y$ amplitude were used instead until fracture occurred.

4.3 Main results and observations

The main response parameters obtained from the tests are summarised in Tables 4.3 and 4.4, and the corresponding deformation patterns and moment-rotation relationships are presented in Figures 4.2 to 4.8. The initial stiffness and moment values at global connection yield were obtained from a bi-linear idealisation of the moment-rotation relationships. Due to the significant post-yield stiffness, values of moment at a joint rotation of 40 mrad are also given in the tables. In addition to the measured initial stiffness and yield moment, Table 4.4 also includes the hysteretic energy dissipated per unit rotation for the cyclic tests. The hysteretic energy is defined as the summation of the areas enclosed by the moment-rotation curves. In subsequent sections, the experimental results and observations from the ten connection tests, described in Tables 4.1 to 4.4, are presented. Particular focus is given in the discussions to the influence of the following parameters on the connection behaviour: (1) gauge distance, (2) channel thickness and (3) presence of web angles.

Table 4.1: Summary of test programme.

Reference	Connection configuration	Channel section from:	Beam section	Angle section	Testing regime	Dimensions [mm] (see Figure 4.1)												
						a	b	c	d	e	f	g	h	i	j	k	l	m
T10-A8-d40-M	T	SHS 150x150x10	UB 305x102x25	L 100x75x8	Monotonic	50	50	35	40	-	-	-	100	45	45	-	-	75
T6.3-A8-d40-M	T	SHS 150x150x6.3	UB 305x102x25	L 100x75x8	Monotonic	50	50	35	40	-	-	-	100	45	45	-	-	75
T10-A8-d65-M	T	SHS 150x150x10	UB 305x102x25	L 100x75x8	Monotonic	45	30	35	65	-	-	-	100	45	45	-	-	75
T6.3-A15-d40-M	T	SHS 150x150x6.3	UB 305x165x40	L 150x80x15	Monotonic	50	50	35	40	-	-	-	150	45	45	-	-	75
W10-A8-d40-M	W	SHS 150x150x10	UB 305x165x40	L 100x75x8	Monotonic	50	50	35	40	35	95	22.5	150	80	50	35	40	60
T10-A8-d40-Y	T	SHS 150x150x10	UB 305x102x25	L 100x75x8	Cyclic	50	50	35	40	-	-	-	100	45	45	-	-	75
T6.3-A8-d40-Y	T	SHS 150x150x6.3	UB 305x102x25	L 100x75x8	Cyclic	50	50	35	40	-	-	-	100	45	45	-	-	75
T10-A8-d65-Y	T	SHS 150x150x10	UB 305x102x25	L 100x75x8	Cyclic	45	30	35	65	-	-	-	100	45	45	-	-	75
T6.3-A15-d40-Y	T	SHS 150x150x6.3	UB 305x165x40	L 150x80x15	Cyclic	50	50	35	40	-	-	-	150	45	45	-	-	75
W10-A8-d40-Y	W	SHS 150x150x10	UB 305x165x40	L 100x75x8	Cyclic	50	50	35	40	35	95	22.5	150	80	50	35	40	60

Note: SHS 200x200x10 was used for columns in all specimens.

4. COMBINED CHANNEL/ANGLE CONNECTIONS

Table 4.2: Material properties.

Component	Average yield strength [N/mm ²]	Average ultimate strength [N/mm ²]
SHS 200x200x10	511	556
SHS 150x150x10	400	502
SHS 150x150x6.3	385	485
UB305x102x25	329	443
L 150x80x15	293	449
L 100x75x8 (A) ¹	312	438
L 100x75x8 (B) ²	252	381

¹ Used in Specimens T10-A8-d40-M, T6.3-A8-d40-M, T10-A8-d65-M, T10-A8-d40-Y, T6.3-A8-d40-Y and T10-A8-d65-Y

² Used in Specimens W10-A8-d4-M and W10-A8-d4-Y

Table 4.3: Summary of results for monotonic tests.

Reference	Initial stiffness [kNm/rad]	Moment at yield [kNm]	Moment [kNm] at 40 mrad rotation
T10-A8-d40-M	3982	22.7	34.0
T6.3-A8-d40-M	1880	16.0	22.2
T10-A8-d65-M	2410	12.1	17.3
T6.3-A15-d40-M	3459	21.1	38.5
W10-A8-d40-M	4928	34.0	57.5

Table 4.4: Summary of results for cyclic tests.

Reference	Initial stiffness [kNm/rad]	Moment at yield [kNm]	Moment [kNm] at 40 mrad rotation	Rate of energy dissipation [kJ/rad]
T10-A8-d40-Y	3623	22.1	32.7	12.8
T6.3-A8-d40-Y	1938	15.5	22.8	10.2
T10-A8-d65-Y	2406	13.0	16.0	8.1
T6.3-A15-d40-Y	3806	23.6	40.3	17.2
W10-A8-d40-Y	5108	34.0	60.0	20.7

4.3.1 Monotonic tests

4.3.1.1 Deformation patterns

The experimental program was designed with the aim of covering a reasonable range of practical configurations as well as to enable validation of the characterisation of individual connection components. This was accomplished by testing details representative of the boundaries in the connection deformation mechanism as well as other details representing intermediate behavioural patterns. To this end, the response of some specimens was expected to be largely determined by the influence of a specific connection component, such as the deformation in the angle component in Specimen T10-A8-d65-M (Figure 4.2a) or the deformation in the reverse channel in Specimen T6.3-A15-d40-M (Figure 4.2c). Other connection details (e.g. Specimen T10-A8-d40-M of Figure 4.2b) exhibited an intermediate deformation pattern with significant interaction between the constituent components. As expected, the interaction between the angle and reverse channel components had a direct influence on the inelastic behaviour of the connection. Angles with the longer leg connected to the channel (gauge distance of 65 mm) typically yielded prior to the development of any significant deformation in the channel, whereas stiffer angles (i.e. gauge distance of 40 mm) tended to concentrate considerable plastic deformation within the channel component. Importantly, regardless of the yield mechanism observed, all connections exhibited high ductility capacities well beyond the common rotation requirements in design codes, without showing significant degradation in stiffness or capacity. The moment-rotation curves of the five monotonic tests are given in Figure 4.3. The deformation patterns shown in Figure 4.2 are discussed in more detail in subsequent sections with respect to the influence of key geometric parameters on the overall connection behaviour.

4.3.1.2 Gauge distance

The influence of gauge distance can be assessed by comparing the results of Specimens T10-A8-d65-M and T-10-A8-d40-M in which the gauge distance was varied from 65 mm in the former to 40 mm in the latter while retaining all other geometric and material characteristics. Figures 4.4a and 4.4b depict the observed yield deformation patterns of the above mentioned tests. As expected, Specimen T10-A8-d65-M, which has the longer leg attached to the 10 mm thick channel, developed two clear plastic hinges in the vertical leg with negligible deformation observed in the face of the channel

4. COMBINED CHANNEL/ANGLE CONNECTIONS

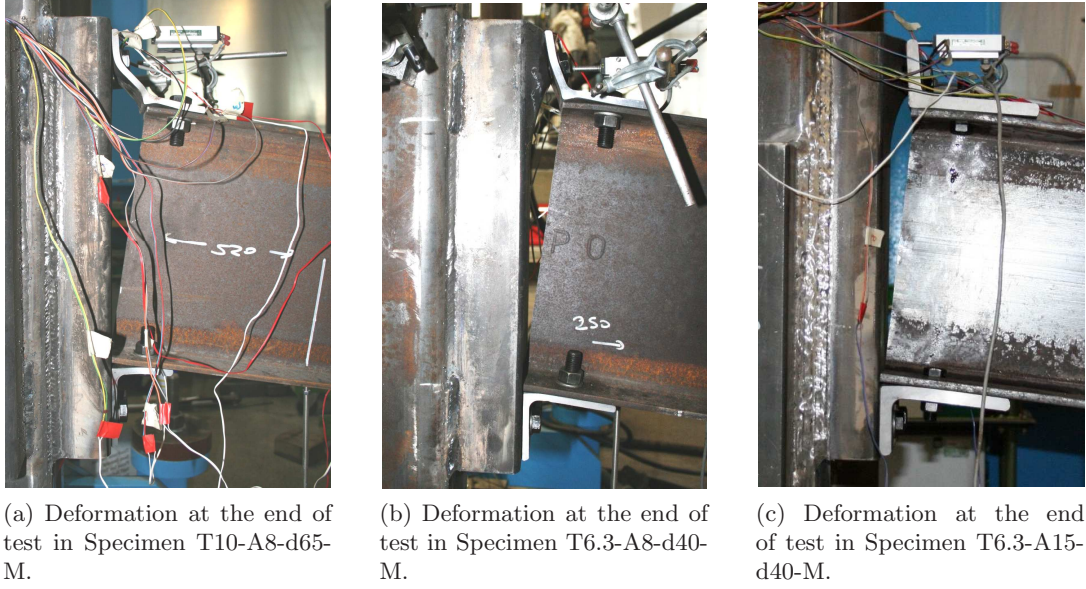


Figure 4.2: Main deformation patterns observed in monotonic tests.

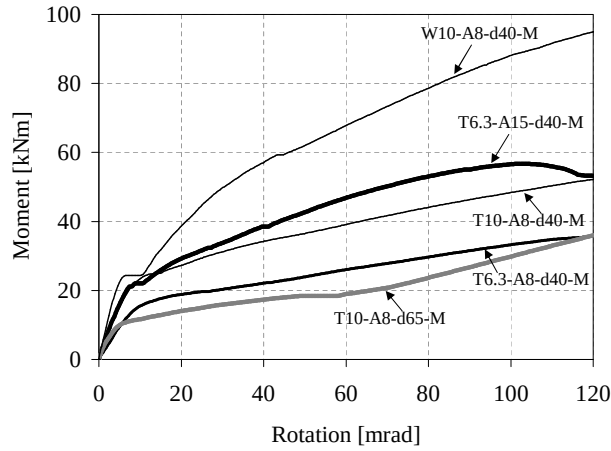


Figure 4.3: Moment-rotation relationships for monotonic tests.

component. On the other hand, in Specimen T10-A8-d40-M more significant plastic deformations were induced in the reverse channel component. This can be further confirmed by observing the strain measurements depicted in Figures 4.4c and 4.4d. From these figures it can be observed that the strains in the reverse channel in Specimen T10-A8-d65-M remain below yield up to rotation demands of over 100 mrad. On the other hand, yielding was observed in the reverse channel of Specimen T10-A8-d40-M at a rotation of around 40 mrad. The location of the plastic hinges near the bolts in

4. COMBINED CHANNEL/ANGLE CONNECTIONS

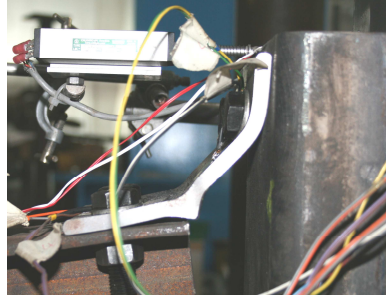
the vertical leg was also clearly different, with plasticity occurring below the bolt head in Specimen T10-A8-d65-M. On the other hand, the plastic mechanism that formed in Specimen T10-A8-d40-M was displaced towards the bolt centerline and more significant bolt rotations were evident.

The observed plastic deformation patterns have a direct influence on the resulting moment rotation relationships as illustrated in Figure 4.4e. Although the difference in the initial stiffness is insignificant, the connection capacity is greatly reduced by increasing the gauge distance. Increasing the gauge distance from 40 mm to 65 mm causes a 40% reduction in the overall connection capacity. As shown in Figure 4.4e, three clear behavioural stages are identifiable on the moment-rotation curve of Specimen T10-A8-d65-M, corresponding to an initial elastic phase maintained up to a rotation of around 5 mrad, a second plastic stage and a third stiffening phase starting at 60 mrad caused by second order tension effects. In contrast, this large displacement tension stiffening stage is not clearly evident in the moment-rotation of Specimen T-10-A8-d40-M which is attributed to the greater rotations observed in the bolts in this case.

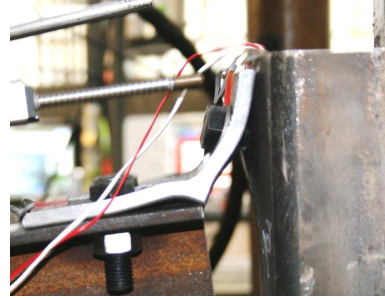
4.3.1.3 Reverse channel thickness

The influence of the reverse channel thickness is illustrated in Figure 4.5. The figure provides a comparison of the response of Specimens T10-A8-d40-M and T6.3-A8-d40-M with channel thickness of 10 mm and 6.3 mm, respectively. As expected, the thickness of the reverse channel affects the flexibility and capacity of this component which in turn have a direct influence on the joint response. Figures 4.5a and 4.5b illustrate such influence on the deformation pattern at large rotation demands. As can be observed from these figures, the more flexible the channel component is, the less is the deformation required from the angle for comparable joint rotations. The influence on the deformation is more evident by comparing Figures 4.5c and 4.5d. When a relatively thick channel is employed (Specimen T10-A8-d40-M), yield strains are reached in the toe of the vertical leg of the angle earlier (at rotations of near 20 mrad). On the other hand, when a relatively thin angle is used (Specimen T6.3-A8-d40-M), yield strains are reached at a later stage (60 mrad rotation) at the same location on the toe of the vertical angle leg. This is due to the concentration of plastic deformations at an early stage in the face of the channel component, leading to a reduction in the deformation demands imposed on the angle component.

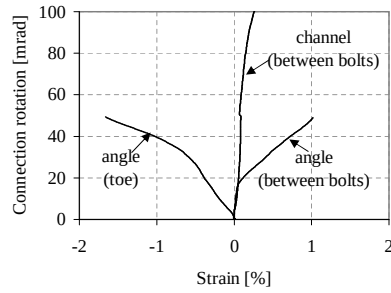
4. COMBINED CHANNEL/ANGLE CONNECTIONS



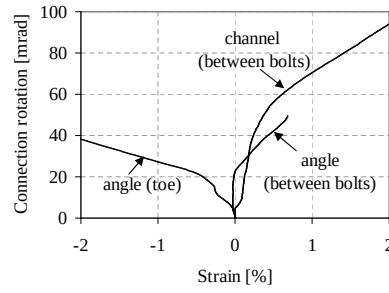
(a) Deformation mode at the end of the test in Specimen T10-A8-d65-M.



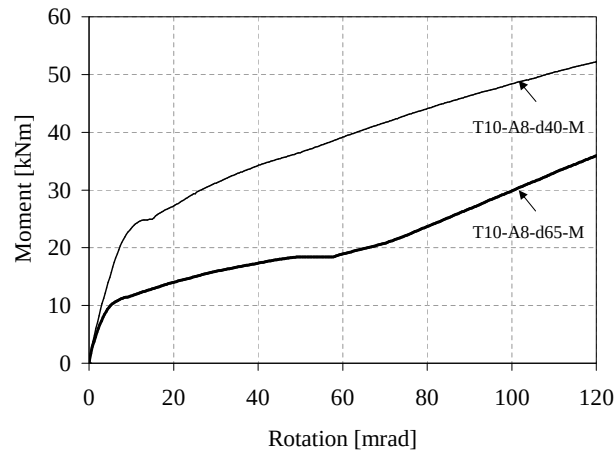
(b) Deformation mode at the end of the test in Specimen T10-A8-d40-M.



(c) Measured strains in the vertical leg of the top angle of Specimen T10-A8-d65-M.



(d) Measured strains in the vertical leg of the top angle of Specimen T10-A8-d40-M.



(e) Moment-rotation relationships for Specimens T10-A8-d65-M and T10-A8-d40-M.

Figure 4.4: Influence of gauge distance.

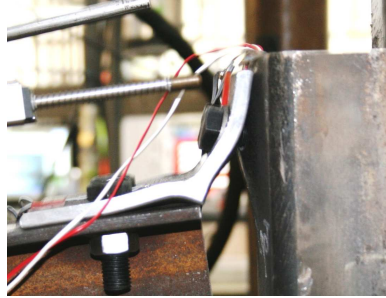
The influence of the reverse channel thickness on the connection stiffness and capacity is depicted in the moment-rotation curves presented in Figure 4.5e as well as in the results summarised in Table 4.3. A reduction in the channel face thickness from 10 mm to 6.3 mm results in nearly 50% drop in stiffness and about 30% decrease in the connection capacity.

4.3.1.4 Web angles

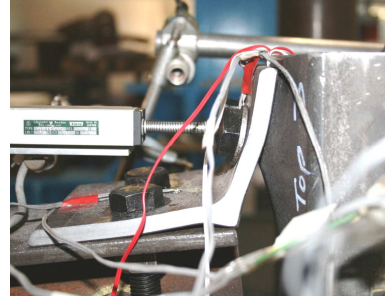
The results presented in the previous chapter with regards to the experimental response of Hollo-bolted angle connections have shown the inclusion of web angles to be effective in increasing the connection stiffness and capacity provided that the local plasticity of the column face is prevented. To this end, Specimen W10-A8-d40-M employs a UB305x165x40 beam of width larger than the effective reverse channel width. The effective width of the reverse channel face refers to the flat part of the SHS (i.e without the curved corners). This proved effective in minimising the channel face deformations on the compression zone of the connection, thus improving the overall joint performance. Figure 4.6 shows views of the deformed shape of Specimen W10-A8-d40-M.

The enhancement in the connection stiffness and capacity due to the addition of web angles can be examined by comparing the results of Specimens W10-A8-d40-M and T10-A8-d40-M where the main difference is the addition of web angles in the former. However, variability in the material properties of the angles used in the two specimens (as indicated in Table 4.2) and some dimensional differences (like the distance h in Table 4.1 and Figure 4.1) require the moment-rotation curves to be normalised prior to the comparison. This normalisation (Figure 4.7) was performed with respect to the initial attainment of yield in the vertical leg of the top angle with due consideration of the different material and geometric properties. It can be concluded from Figure 4.7 and the non-normalised values presented in Table 4.3, that the inclusion of web angles enhances the initial stiffness, capacity and post-elastic hardening properties of the joint.

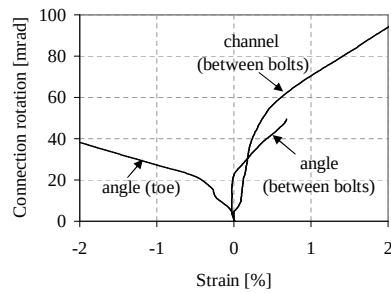
4. COMBINED CHANNEL/ANGLE CONNECTIONS



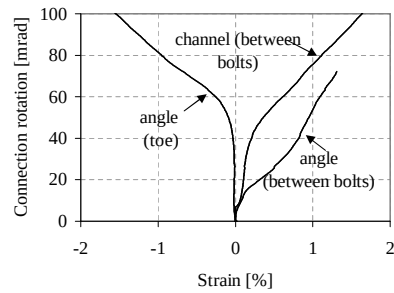
(a) Deformation mode at the end of the test in Specimen T10-A8-d40-M.



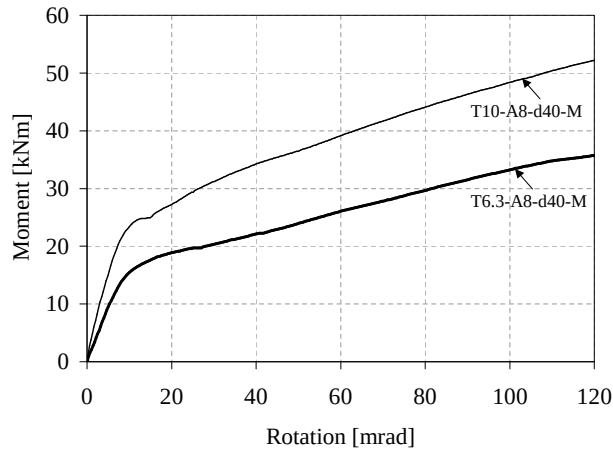
(b) Deformation mode at the end of the test in Specimen T6.3-A8-d40-M.



(c) Measured strains in the vertical leg of the top angle of Specimen T10-A8-d40-M.



(d) Measured strains in the vertical leg of the top angle of Specimen T6.3-A8-d40-M.



(e) Moment-rotation relationships for Specimens T10-A8-d40-M and T6.3-A8-d40-M.

Figure 4.5: Influence of channel thickness.

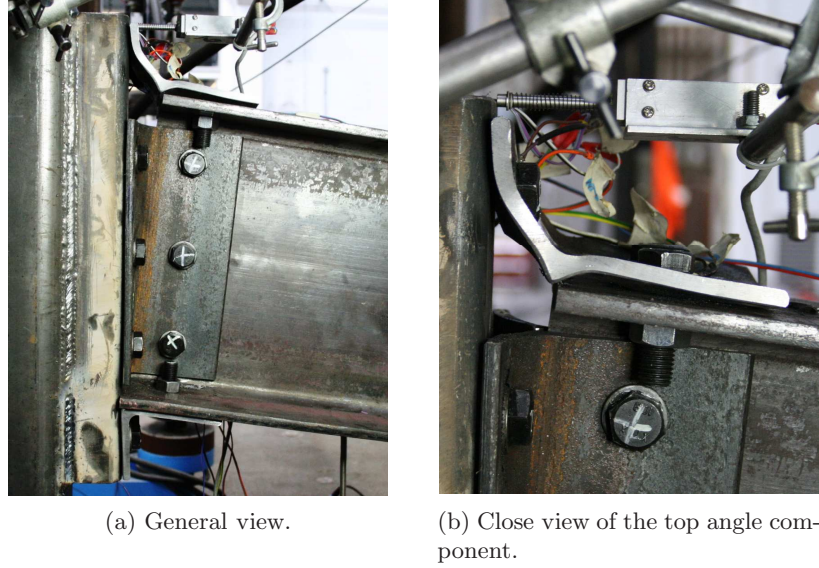


Figure 4.6: Deformation at the end of test in Specimen W10-A8-d40-M.

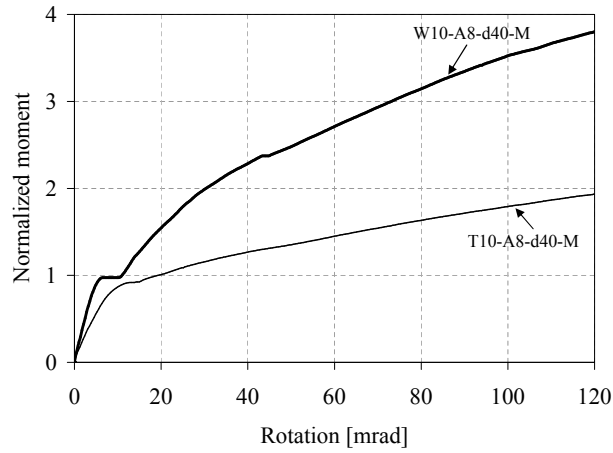


Figure 4.7: Influence of web angles.

4.3.2 Cyclic behaviour

4.3.2.1 Hysteretic response

The cyclic moment-rotation curves obtained for the reverse channel connections under study are presented in Figure 4.8. It is clear from these plots that all specimens exhibited largely stable hysteretic behaviour with gradual transition between elastic and inelastic response. Furthermore, the moments developed were consistent with those observed in the corresponding monotonic tests as it is evident when comparing Tables

4. COMBINED CHANNEL/ANGLE CONNECTIONS

4.3 and 4.4. As shown in Figure 4.8, the reloading branch changes with increasing rotation levels but always reaches the same point at which the previous cycle unloaded. On the other hand, the unloading branches are largely parallel to each other regardless of the level of rotation.

Due to the accumulation of plastic deformations in the angle and channel components, residual deformations develop. This leads to flattening of the hysteresis loops during the load reversal, owing to the reduction in the angle resistance. However, an increase of stiffness occurs when contact with the column is resumed in the compression region. The slippage in the standard bolts connected to the beam also contributes to this phenomenon. As expected, this pinching behaviour is more evident when higher capacities are reached such as in the case of Specimens T10-A8-d40-Y and W10-A8-d40-Y in Figures 4.8a and 4.8e, respectively. In all cases, rotations exceeding ± 50 mrad were reached without signs of notable stiffness or strength deterioration.

The cyclic behaviour of Specimen T6.3-A15-d40-Y which is depicted in Figure 4.8d deserves special attention. As noted before, Specimen T6.3-A15-d40-Y was designed in order to isolate the behaviour of the reverse channel component by forcing most of the deformation to occur within the channel face (i.e. a very stiff angles with 15 mm thickness were used). Therefore, as shown in Figure 4.8d, although considerable moments were mobilised, the pinching effect was, in relative terms, not as severe as in other cases. This is attributed to the domination of the channel rather than the angle components in this test.

4.3.2.2 Ductility and energy dissipation

The reverse channel connections examined in this study exhibited high ductility under monotonic and cyclic loading conditions. One of the additional objectives of the present study was to investigate the low-cycle fatigue behaviour of the connections. Accordingly, high amplitude cycles were imposed on all specimens until fracture occurred in any of the components. However, as discussed in more detail below, it should be noted that this only took place at rotation levels which are well beyond those expected under characteristic seismic loading conditions.

Figure 4.9 presents an assessment of the energy dissipation in the cyclic tests. The

4. COMBINED CHANNEL/ANGLE CONNECTIONS

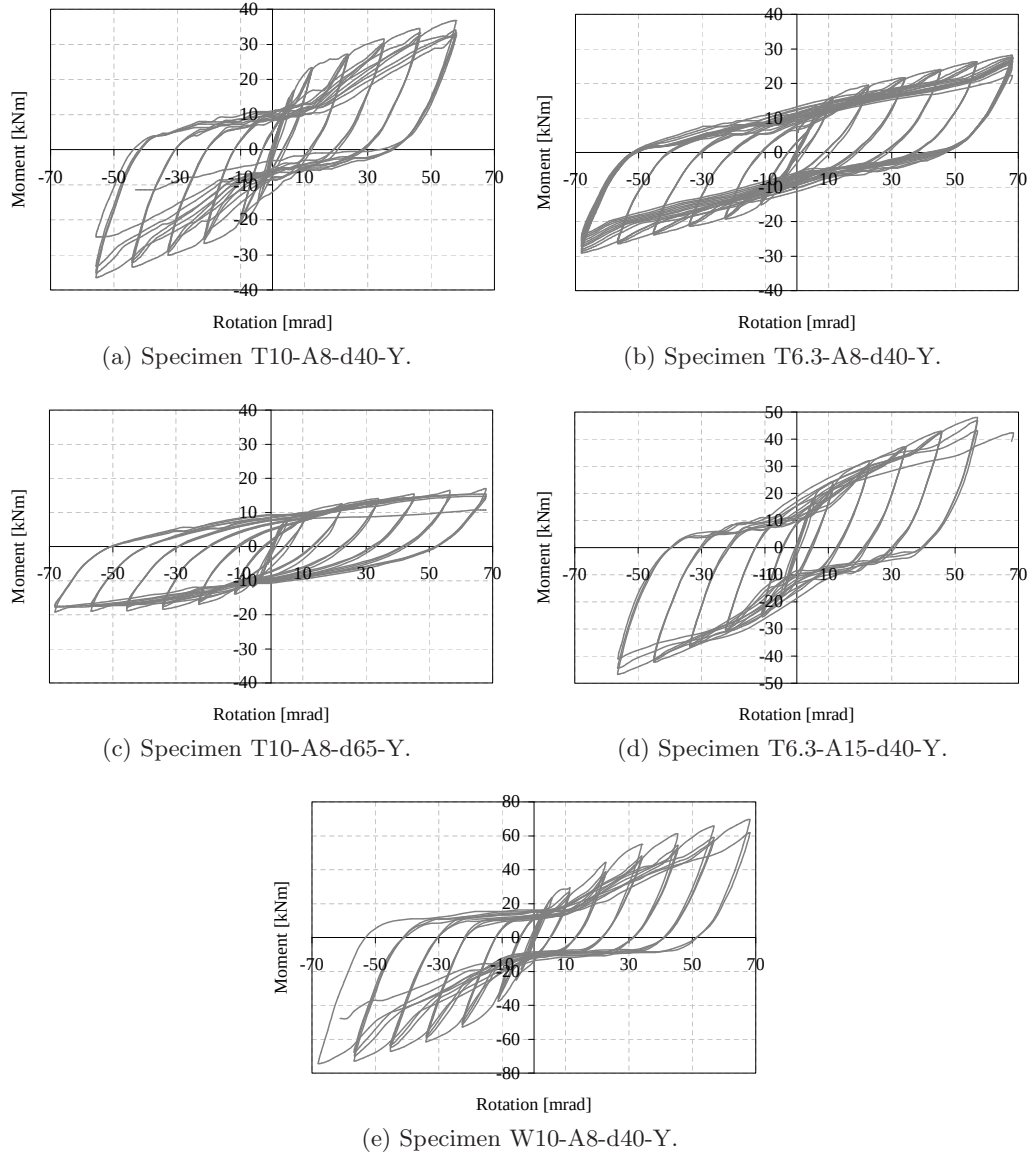


Figure 4.8: Hysteretic moment-rotation relationships.

curves depict the cumulative dissipated energy versus cumulative rotation, whilst the rate of energy dissipated per unit rotation is also given in Table 4.4. As expected, there is a direct relationship between the capacity of the connection and the energy dissipation capabilities. Accordingly, the connections incorporating web angles exhibited comparatively high levels of energy dissipation. Also, the reverse channel component (which is particularly dominant in Specimen T6.3-A15-d40-Y as discussed before) is shown to be a good source of energy dissipation.

4. COMBINED CHANNEL/ANGLE CONNECTIONS

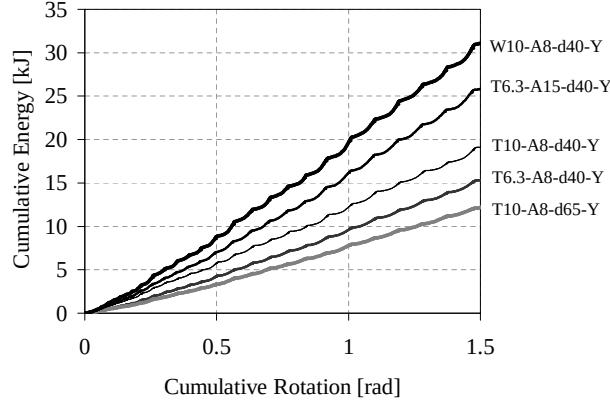


Figure 4.9: Cumulative energy dissipation.

4.3.2.3 Fatigue Damage

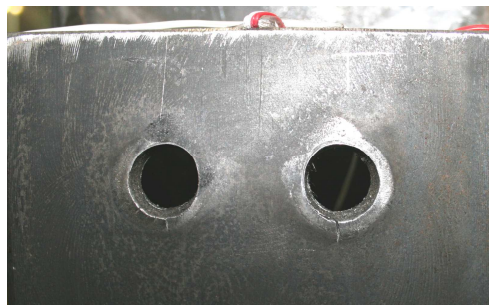
As noted previously, additional cycles were applied up to failure at an amplitude of $\pm 12\Delta_y$ for Specimens T6.3-A8-d65-Y, T10-A8-d65-Y, T6.3-A15-d40-Y and W10-A8-d40-Y while additional cycles at $\pm 10\Delta_y$ were applied for Specimen T10-A8-d40-Y. In all tests, except T6.3-A15-d40-Y fracture was observed at the base of the vertical leg of the top or seat angle, an example of which is presented in Figure 4.10a for Specimen T6.3-A8-d40-Y. On the other hand, owing to the largely elastic behaviour of the top and seat angles, Specimen T6.3-A15-d40-Y experienced fracture in the corners of the reverse channel component as illustrated in Figure 4.10c.

Depending on the angle gauge distance and the level of deformation accumulated in the channel components, the fatigue life was different for each specimen. For example, crack initiation was observed in Specimen T10-A8-d40-Y at the first peak of the twenty first cycle (fifth cycle at $+10\Delta_y$) and full fracture at the next $-10\Delta_y$ peak. As expected, Specimen T6.3-A8-d40-Y accumulated more deformation within the reverse channel face than Specimen T10-A8-d40-Y and hence the angle fracture was delayed until the first peak of the thirtieth cycle (at an amplitude of $+12\Delta_y$). Furthermore, the higher deformation demands experienced by the reverse channel face on Specimen T10-A8-d40-Y led to damage around the bolt holes in the channel component (as shown in Figure 4.10b). This damage was reflected in the slight (but sustained) stiffness degradation observed in the hysteretic response in Figure 4.8b starting from the twenty third cycle onwards. Specimen T10-A8-d65-Y also experienced fatigue fracture in the toe of

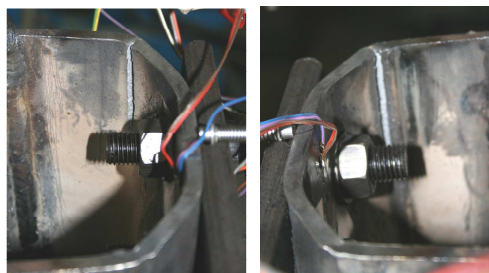
4. COMBINED CHANNEL/ANGLE CONNECTIONS



(a) Typical low-cycle fatigue fracture in top angle(Specimen T6.3-A8-d40-Y).



(b) Fatigue damage around bolt holes in reverse channel component (Specimen T6.3-A8-d40-Y).



(c) Low cycle fatigue fracture in the corners of the reverse channel component of Specimen T6.3-A15-d40-Y.

Figure 4.10: Fatigue damage.

4. COMBINED CHANNEL/ANGLE CONNECTIONS

the angle component which initiated at the last peak of the twenty second cycle and was fully developed at the first peak of the twenty third cycle (both at amplitudes of $\pm 12\Delta_y$). Finally, Specimen W10-A8-d40-Y was also subjected to cyclic loads until fatigue fracture was observed in the bottom angle. This occurred at the first $+12\Delta_y$ inelastic excursion during the twentieth cycle. The data obtained from the test results provide useful information for the validation of detailed damage models [7, 10], that will be performed in the next chapter.

4.4 Concluding Remarks

This chapter examined the behaviour of reverse channel connections with angles for open beam-to-tubular column joints. The experimental programme enabled a direct assessment of the influence of a number of key considerations, such as the angle gauge distance, the reverse channel thickness, and the presence of web angles on the overall connection behaviour. The test results also provide essential information for the validation of the detailed analytical models proposed in the next chapter.

As expected, it was observed that the inelastic mechanisms exhibited by this type of connection stem from the relative contribution of the constituent components. It was shown that the angle gauge distance is inversely proportional to the connection capacity with a less significant effect on the stiffness. On the other hand, the flexibility of the reverse channel component, examined herein by varying the channel thickness, has a direct influence on both the initial stiffness and capacity of the connection. Besides, it was shown that the addition of web angles can significantly enhance the overall resistance in the elastic and post-elastic ranges. The tests also highlighted the importance of ensuring the presence of an adequate mechanism for transferring the compression forces to the column in order to achieve satisfactory performance.

In terms of ductility, the reverse channel connections investigated in this chapter reached rotational levels exceeding 120 mrad under monotonic loading and well over 50 mrad under cyclic loading. These rotational capacities are well beyond the demands expected under typical design scenarios. Relatively stable hysteretic response was observed under cyclic loading in all cases with reasonably good energy dissipation capabilities, although more notable pinching behaviour was observed with the increase in the capacity of the connection. The low-cycle fatigue life was also studied by subject-

4. COMBINED CHANNEL/ANGLE CONNECTIONS

ing the specimens to cycles of high rotation levels up to fracture. Fracture occurred in the top and seat angle components in all the specimens with the exception of Specimen T6.3-A15-d40-Y, in which the channel component dominated the behaviour.

In general, the tests demonstrate the satisfactory inelastic performance of this type of reverse channel connections under monotonic and cyclic loading conditions. In terms of fatigue life, the experimental results described herein show that reverse channel connections are able to withstand inelastic cyclic deformations beyond those experienced by steel structures under severe seismic excitations, without suffering significant stiffness or strength deterioration. This characteristic, together with the considerable post-yield hardening exhibited by this form of connection make such forms particularly suitable for secondary lateral resisting systems where the connections are required to achieve adequate ductility while at the same time maintaining the gravity loading carrying capacity of the frames. These aspects for the potential use of semi-rigid combined channel angle/connections are discussed further in Chapter 8.

Chapter 5

Mechanical Models for Angle Connections

5.1 Introduction

Having studied the experimental behaviour of semi-rigid angle connections between open beams and tubular columns in the previous chapters, this chapter deals with the prediction of their monotonic and cyclic moment-rotation response. It was shown in the corresponding literature review of Chapter 2 (Section 2.2) that analytical research on the response prediction of bolted angle connections to tubular columns is scarce. Ghobarah et al. [63] and Silva et al. [141] suggested analytical models for the evaluation of stiffness and capacity of blind-bolted *end-plate* connections between open beams and tubular columns. However, available models on end-plate connections cannot be directly applied to Holo-bolted details with angles due to the significant influence of contact phenomena as well as the complex interactions between the blind-bolts, column face and angle components arising from the inherent rotational flexibility of the Holo-bolt (as observed in the experimental results described in Chapter 3). Moreover, the applicability of available component models to combined channel/angle connections needs to be verified against the experimental results presented in Chapter 4. Accordingly, there is a need for dedicated models that provide a faithful characterization of the response of semi-rigid open beam-to-tubular column bolted connections. Such models should be capable of predicting the main response characteristics under cyclic loading conditions.

This chapter suggests mechanical models for the prediction of the response of bolted

5. MECHANICAL MODELS

angle connections between open beams and tubular columns. The proposed models are based on the component approach in conjunction with incremental procedures, and are able to trace the full monotonic and cyclic moment-rotation response of angle connections to tubular columns. After validation against experimental results, the models are employed in a parametric investigation into several key factors influencing the behaviour of blind-bolted angle connections, namely: the blind-bolt grade, angle thickness, column face slenderness and gauge distance (defined here as the vertical distance between the centreline of the blind-bolt and the base of the angle connected to the column). Based on the findings of the parametric assessment, simplified semi-analytical expressions are proposed for the estimation of the initial stiffness and moment capacity of Hollo-bolted connections. Besides, several considerations related to failure mode control and rotational ductility are highlighted. The response prediction of combined channel/angle connections is also briefly investigated. In comparison with Hollo-bolted connections, the use of standard bolts in reverse channel configurations eliminates several of the complications imposed by the rotational flexibility of Hollo-bolted details; therefore, emphasis is placed on the characterization of the reverse channel component. Simple expressions for determining the stiffness and capacity of the overall joint are discussed, and the parameters that have a most notable influence on their accuracy are highlighted. Finally, by using the experimental results presented in Chapters 3 and 4, the susceptibility to low-cycle fatigue within critical connection components is discussed and the reliability of available fatigue damage models is assessed.

5.2 Model development

5.2.1 General

The response of angle connections generally involves various complex interactions between the constituent components. Additionally, the experimental assessments of Chapter 3 have demonstrated that blind-bolted angle connections exhibit specific behavioural characteristics that further complicate the response. Overall, the modelling approach should consider, among others: (i) the relatively high axial (and flexural) flexibility of the blind-bolts, (ii) influence of plasticity in the horizontal (beam side) angle leg, (iii) angle membrane effects at large displacements, and (iv) significant strain hardening effects. The mechanical component-based model described in this chapter incorporates these effects and enables a faithful simulation of the response of open beam-to-tubular

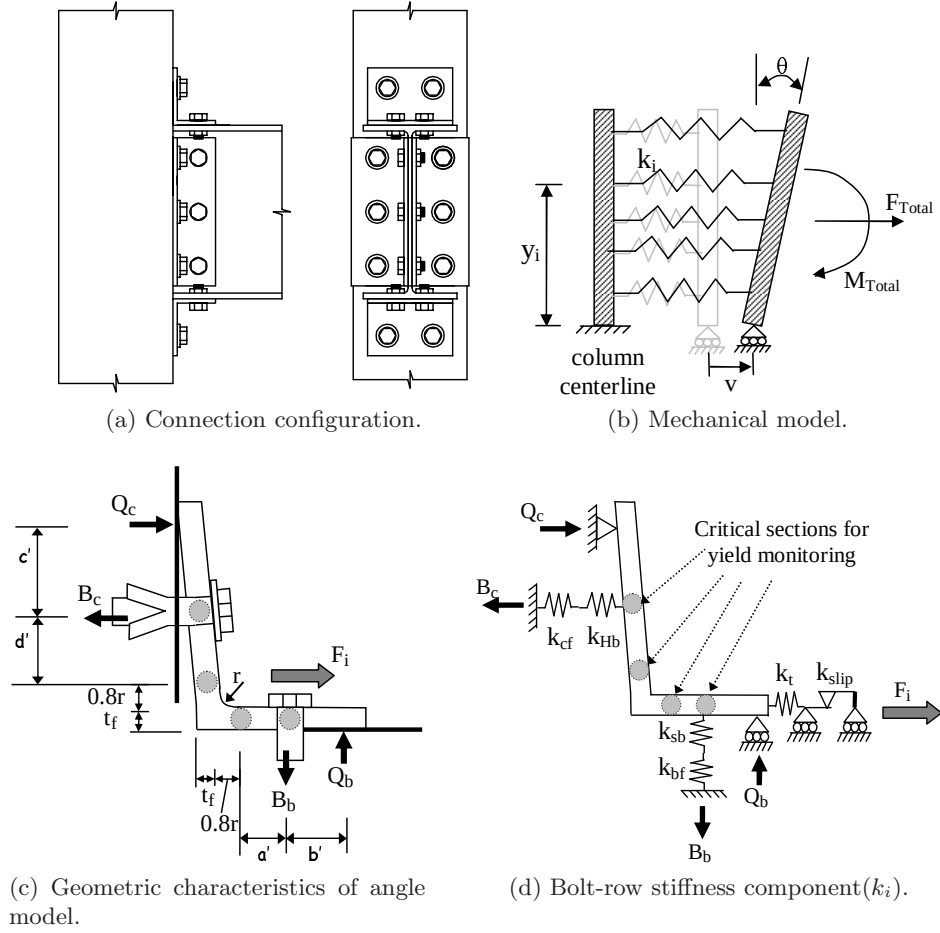


Figure 5.1: Mechanical model for angle connections.

column connections with angles under monotonic and cyclic loading conditions. A multi-linear incremental approach which, in various different forms, has been applied to T-stub components [71, 91, 145], is utilised here in order to trace the non-linear inelastic force-deformation response of the angle component. Furthermore, the flexibility of each component is considered separately and the interaction between them is accounted for through the underlying mechanics with due consideration of the evolution of prying forces.

In general, any typical connection configuration (such as the one presented in Figure 5.1a) can be idealised as an assemblage of uniaxial springs as shown in Figure 5.1b. Each spring in Figure 5.1b represents the contribution of an individual bolt-row. Figure 5.1c shows the notation and dimensions used at the bolt-row level. The stiffness

5. MECHANICAL MODELS

of each bolt-row k_i , is intrinsically non-linear and can be estimated by means of the mechanical model depicted in Figure 5.1d. In this figure, k_{cf} is the stiffness of the column face acting in series with the Holo-bolt axial stiffness (k_{Hb}), k_{bf} is the beam flange stiffness, k_{sb} is the axial stiffness of the standard bolt connecting the angle to the beam, k_t is the tensile stiffness of the horizontal leg of the angle, and k_{slip} represents a rigid-plastic element that takes into account the deformations due to bolt-slippage.

It should be noted that although Figure 5.1c appears to represent a top angle, the same model characteristics described above are used for bolt-rows involving web angle components, provided the appropriate width is used in the model, and k_{bf} is considered as infinite to account for the symmetry in applied forces typical of web angle components. Finally, in order to satisfy equilibrium between internal forces and external actions (moment M_{Total} and force F_{Total} in Figure 5.1b), each bolt-row resists a horizontal force F_i (either in tension or compression) and hence:

$$F_{Total} = \sum F_i \quad (5.1)$$

$$M_{Total} = \sum F_i y_i \quad (5.2)$$

where y_i defines the location of the bolt-row with respect to any reference datum. The following sub-sections describe the characterisation employed for each component and the means by which individual component responses are assembled.

5.2.2 Prying action

The prying forces, Q_c at the vertical (column) leg and Q_b at the horizontal (beam) leg of the angle, are idealised as point loads located at a distance c' or b' , respectively, from the bolts, as indicated in Figure 5.1c. The yielding zone near the root of the angle is assumed to be located at a distance of $0.8r$ from the face of the angle, with r being the root radius.

The axial forces B_c and B_b are transmitted to the bolts connecting the angle to the column and the beam, respectively. Force B_b is assumed to develop at the inside edge of the bolt connecting the angle to the beam based on the prying model suggested by Struik and de Back [144]. The force B_c , is assumed to be located between the bolt centreline and the edge of the bolt connecting the angle to the column in order

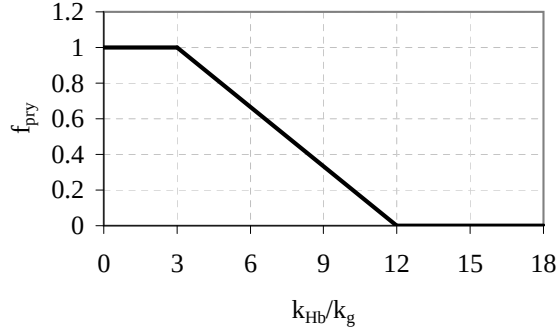


Figure 5.2: Value of factor f_{pry} in Equation 5.3 as a function of the Holo-bolt relative stiffness.

to account for the inherent rotational flexibility of the Holo-bolt. For more flexible angle legs, plastic hinges were observed at the inside edge of the bolt during the tests (Chapter 3). On the other hand, when relatively stiff angles were used, plastic hinges tended to move closer to the Holo-bolt centreline. Accordingly, the distances c' and d' are defined as follows:

$$c' = c + (1 - f_{pry}) \frac{d_{Hb}}{2} \leq 1.25(d - 0.8r - t_f) \quad (5.3)$$

$$d' = d - (0.8r + t_f) - (1 - f_{pry}) \frac{d_{Hb}}{2} \quad (5.4)$$

where dimensions c and d are measured as specified in Figure 3.2, t_f is the angle thickness, r is the angle root radius, d_{Hb} is the Holo-bolt diameter and f_{pry} is a factor that takes into account the change in the location of the plastic hinge. Figure 5.2 presents the assumed variation in the factor f_{pry} as a function of the ratio between the stiffness of the blind-bolt (defined as k_{Hb}) and the value of k_g . In turn, k_g is a parameter that reflects the relative stiffness of the angle leg with respect to that of the Holo-bolt, and is defined here as:

$$k_g = \frac{EI}{(d - t_f - 0.8r)^3} \quad (5.5)$$

where I is the moment of inertia of the longitudinal section of the angle (i.e. $pt_f^3/12$). It is evident from Figure 5.2 that for stiffer angle configurations, the plastic hinge will be displaced towards the centreline of the Holo-bolt (i.e. $f_{pry} = 1$), whereas for more flexible configurations the plastic hinge zone would be located at the inside edge of the Holo-bolt towards the root of the angle (i.e. $f_{pry} = 0$). A linear interpolation between the limit values of $k_{Hb}/k_g = 3$ and $k_{Hb}/k_g = 12$ was found to provide close

5. MECHANICAL MODELS

agreement with experimental results and observations. A value of around 200 kN/mm corresponding to the initial stiffness of the Hollo-bolt was used for k_{Hb} during the calculation of c' and d' , in accordance with typical experimental values reported in Chapter 3 and summarized in Figure 5.3 and Table 5.1. Furthermore, once initially set, the distances c' and d' are assumed to remain constant throughout the stiffness calculation process.

5.2.3 Column face component

A bi-linear idealisation of the force-displacement response of the column face component is used. The initial stiffness is obtained by an adaptation of the stiffness of an infinitely long cantilever plate subjected to a point load used for open columns [43] as:

$$k_{cf} = \frac{\pi E t_c^3}{12(1 - \nu^2) C_t \left(\frac{b_c - t_c}{2} \right)^2} \quad (5.6)$$

where E is the elastic modulus of steel, t_c is the column face thickness, ν is the Poisson's ratio for steel, b_c is the column face width and C_t is a coefficient assumed as 0.18 based on the results of detailed continuum numerical studies [43].

The tension force (B_{yc}) that would cause yielding of the column face is calculated based on an adaptation of the expressions developed by Gomez [67], and is considered as the minimum of the punching resistance (B_{y1}) and local yielding capacity (B_{y2}), thus:

$$B_{yc} = \min(B_{y1}, B_{y2}) \quad (5.7)$$

where B_{y1} is calculated from the following equation:

$$B_{y1} = \begin{cases} \frac{N\pi D_H t_c \sigma_y}{\sqrt{3}} & \text{if } 2(L_{HB} + 0.9D_H) \geq N\pi D_H \\ \frac{2(L_{HB} + 0.9D_H) t_c \sigma_y}{\sqrt{3}} & \text{if } 2(L_{HB} + 0.9D_H) < N\pi D_H \end{cases} \quad (5.8)$$

and B_{y2} is determined from:

$$B_{y2} = f_k \gamma m_{plc} \quad (5.9)$$

where m_{plc} is the plastic moment of the column face by unit length, D_H is the bolt hole diameter, N the number of bolts, σ_y the yield stress, L_{HB} is the spacing between

two Hollo-bolts in the same bolt-row (e.g. i in Figure 3.2), and factors f_k and α are calculated from Equations 5.10 and 5.11 as follows:

$$f_k = \begin{cases} 1 & \text{if } \frac{L_{HB} + 0.9D_H}{b_c} \geq 0.5 \\ 0.7 + \frac{0.6(L_{HB} + 0.9D_H)}{b_c} & \text{if } \frac{L_{HB} + 0.9D_H}{b_c} < 0.5 \end{cases} \quad (5.10)$$

$$\gamma = \frac{4}{1 - L_{HB}/b_c} \left(\pi \sqrt{1 - \frac{L_{HB}}{b_c}} \right) + 1.8 \frac{D_H}{b_c} \quad (5.11)$$

Finally, the post yield stiffness is assumed to be 10% of the initial stiffness in line with typical values found by other researchers [63].

5.2.4 Hollo-bolt response

A multi-linear idealization of the axial force-displacement relationship is used to model the Hollo-bolt response. Figure 5.3 presents the comparison between typical experimental results (3.2.3) and the proposed idealization for M16 Hollo-bolts of Grade 10.9 and 8.8. The corresponding force-displacement values are given in Table 5.1. It is important to note that the influence of the elastic flexibility of the plates used for the Hollo-bolt tension test assembly was assumed to be negligible for idealization purposes.

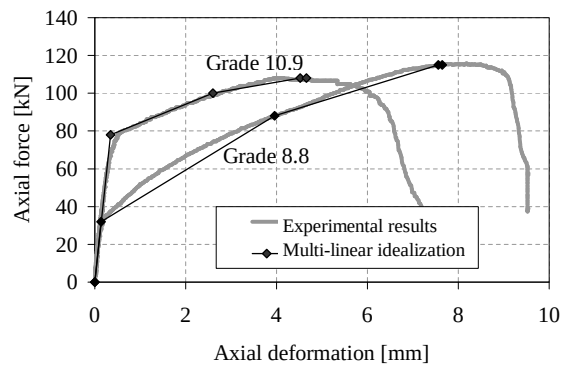


Figure 5.3: Axial response of M16 Hollo-bolts.

5. MECHANICAL MODELS

Table 5.1: Hollo-bolt axial force-displacement relationships.

Hollo-bolt grade			
8.8		10.9	
Displacement [mm]	Force [kN]	Displacement [mm]	Force [kN]
0.0	0	0.0	0
0.2	32	0.4	78
4.0	88	2.6	100
7.6	115	4.5	108

5.2.5 Beam flange stiffness

The beam flange connected to the top or seat angle is assumed to remain elastic, and its stiffness is evaluated through a simplification of Equation 5.6 as follows:

$$k_{bf} = \frac{2E\pi t_b^3}{m_b^2} N \quad (5.12)$$

where t_b is the flange thickness and m_b is the lever arm to the point of application of load B_b measured from the face of the beam web, as depicted in Figure 3.2a. Importantly, as mentioned before, when web angle components are being evaluated, the value of k_{bf} can be assumed as infinite. This is a reasonable assumption, given negligible net forces that act in the out of plane direction for the web of the beam due to symmetrically applied forces.

5.2.6 Standard bolts

The elastic stiffness of the standard bolts that connect the angle to the beam is calculated as:

$$k_{sb} = \frac{A_b E}{L_b} N \quad (5.13)$$

where N is the number of bolts considered to contribute to the stiffness of the bolt-row, A_b is the cross sectional area of a single bolt and L_b is the effective length of a single bolt subjected to axial tension. This effective length corresponds to the bolt segment subjected to tension and is typically measured between the head and the shank of the tightened bolt [27].

5.2.7 Tensile behaviour of horizontal leg (beam side)

Axial tensile deformation is considered only in the horizontal leg of the angle connected to the beam, whereas any tensile deformation in the vertical leg (column side) of the angle is ignored. This was considered adequate in light of the relatively limited contribution of the axial flexibility of the angle legs, a contribution that is further reduced in the case of the vertical leg due to the orthogonality of the applied load, particularly at early stages. The axial tensile stiffness of the horizontal leg of the angle is determined from:

$$k_t = \frac{Ept_f}{a'} \quad (5.14)$$

where p is the angle width (h in Figure 3.2) for top and seat angles or an effective width in the case of web angle components.

5.2.8 Bolt slippage

A rigid-plastic force-displacement relationship is considered for the slippage in the standard bolts. No deformation is assumed up until the applied force overcomes the friction slip resistance P_f . Such friction resistance can be calculated as the product of the friction coefficient c_f and the pre-stressing force p_{pf} applied during the tightening process, hence:

$$P_f = c_f p_{pf} N \quad (5.15)$$

Once P_f is reached, a slip displacement up to the bolt hole tolerance occurs in the bolt-row component. A friction coefficient of 0.2 is assumed as a default value in the present model, and a value of 40kN was assumed for p_{pf} in accordance with tests.

5.2.9 Force-displacement solution procedure

The model presented above, is solved using an incremental procedure within which the stiffness k_i can take values corresponding to any of the possible states depicted in Figures 5.4 and 5.5. In this study, the solution process is divided in two phases. Figure 5.4 presents the scheme of possible behavioural stages for the first phase (Phase I) in which all deformations are assumed to concentrate in the vertical (column side) leg of the angle component. As depicted in Figure 5.4, eleven deformational stages are possible, including two corresponding to complete angle-column separation modes that may occur in blind-bolted connections given the inherent flexibility of the Holo-bolts. Such

5. MECHANICAL MODELS

angle-column separation occurs when the prying force Q_c (see Figure 5.1d) becomes zero and the vertical leg of the angle acts as a cantilever beam; nevertheless, stresses will start to concentrate near the root radius, hence a plastic hinge will eventually develop and the contact between the angle tip and the column face will be re-established.

The effects of material hardening that were observed to have a significant influence on the behaviour of angle connections [32, 45, 139] are accounted for in the model by: (i) introducing rotational springs at the locations where the angle section becomes fully plastic and (ii) assuming a variable strain hardening modulus (E_{sh}). To this end, it was observed that, some plastic regions experience relatively high strain levels which, depending on the type of material, can result in significant hardening effects. Accordingly, a varying strain hardening modulus was used in the calculations. For States k_{Iep} and k_{Ipe} in Figure 5.4, the strain hardening modulus is set to 0.5% of the steel elastic modulus (i.e. $E_{sh} = 0.005E$), whereas for states k_{Iyp} , k_{Ipy} and k_{Ipp} the strain hardening modulus can change to a higher value (assumed as 1.5%). Appendix A presents the expressions used to calculate the stiffnesses at each stage of Phase I.

The solution algorithm outlined by Swanson and Leon [145] for T-stub components was adapted here for angle components and implemented in MATLAB [108]. The process starts with the calculation of moment limits by considering: (i) the flexural plastic capacity of the angle leg, given as:

$$m_{pla} = \frac{\sigma_y p t_f^2}{4} \quad (5.16)$$

and (ii) the moment (m_{ya}) at the instant in which the yield stress is attained in the outer fibres of the angle section, represented by:

$$m_{ya} = \frac{\sigma_y p t_f^2}{6} \quad (5.17)$$

Subsequently, limiting values for the increments in force ΔF_i are evaluated as those that would cause: (i) yielding or (ii) full plasticity at the critical sections, (iii) change of stiffness values in the multi-linear relationship of the Hollo-bolt as given in Table 5.1, or (iv) reduction of the prying force Q_c of Figure 5.1d to zero. The actual force increment ΔF_i is considered as the lowest of these four possible values, hence governing which branch of the decision tree of Figure 5.4 is subsequently followed.

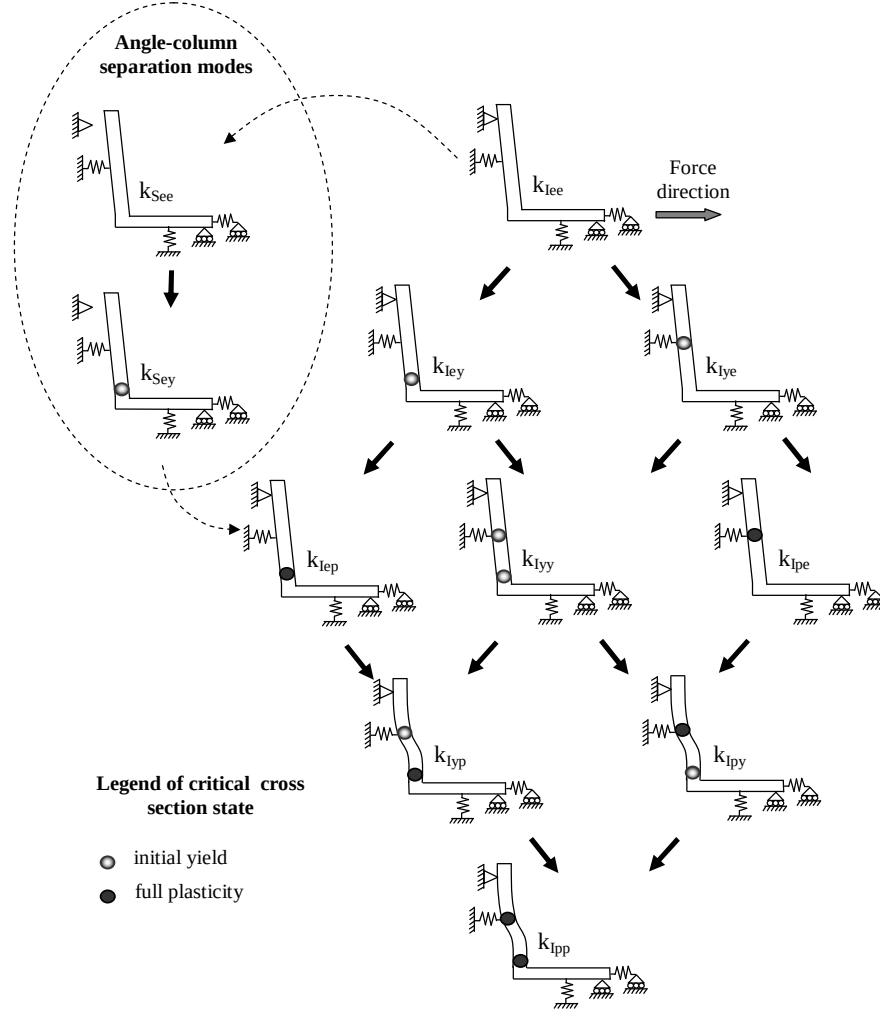


Figure 5.4: Schematic representation of the decision tree used for the mechanical model. Phase I.

The second deformation phase (Phase II), shown in Figure 5.5, starts with the angle vertical leg acting as a mechanism, with rotational stiffness provided by the strain hardening at the plastified critical sections. Prying forces are considered in the same manner as before but no plasticity is expected at the standard bolt location in the beam-side (horizontal) leg of the angle. During Phase II, similar expressions as those presented in Appendix A for the calculation of horizontal displacements in the vertical (column side) leg of the angle, are used to calculate vertical displacements in the horizontal (beam side) leg of the angle. Subsequently, by assuming axial inextensibility in the vertical leg, these vertical displacements of the horizontal leg are transformed into additional force-displacement pairs at the bolt-row level through pure geometrical

5. MECHANICAL MODELS

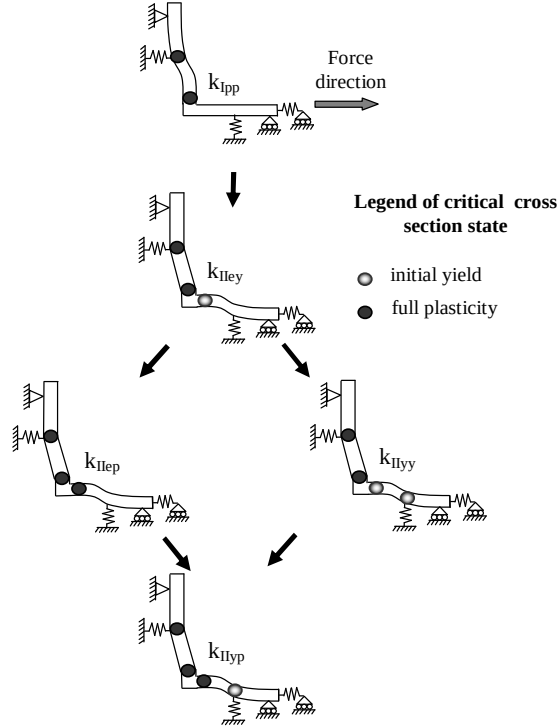


Figure 5.5: Schematic representation of the decision tree used for the mechanical model. Phase II.

considerations. In this way, geometric non-linearity is directly accounted for and hence the experimentally-observed stiffening effects at large displacements are reasonably incorporated. It is important to note that the deformation patterns of Phase II were experimentally observed only when the stiffness of the combined column and Holo-bolt provided sufficient deformational restraint. Hence, the model assumes that Phase II modes become inactive whenever significant lateral deformations start to accumulate in the column-side (vertical) leg of the angle. Such limiting condition is reached if yielding in the column face occurs in the model (as evaluated from Equation 5.7). The solution process is continued until the ultimate capacity of the Holo-bolt is reached. The same overall solution process is followed for bolt-rows located either in the top or web angle regions, with due consideration of the appropriate effective width.

5.2.10 Model adaptation to combined channel/angle connections

The development of the mechanical model described above was mainly motivated by the peculiarities found in the response of Holo-bolted connections (Chapter 3) with a

view to performing the parametric analyses presented later in this chapter. However, the mechanical model is versatile enough to capture the principal behavioural characteristics of more general bolted joint configurations with angles. Nevertheless, in order to predict the response of combined channel/angle connections, two main modifications need to be introduced into the component characterization of Figure 5.1: (i) the consideration of the standard bolt axial stiffness (k_{sb}) in lieu of the Holo-bolt stiffness (k_{HB}) and (ii) the inclusion of the stiffness (k_{RC}) and capacity (B_{RC}) of the reverse channel component.

The behaviour of the reverse channel component can be assessed by considering the response of Specimens T6.3-A15-d40-M and T6.3-A15-d40-Y as described in Chapter 4 presented in Figures 4.3 and 4.8d, respectively. By providing very stiff angles as well as beams of width comparable to that of the reverse channel, these tests ensured that the plastic deformation occurred mainly in the tension side of the reverse channel as illustrated in Figure 5.6a. The plastic deformation observed at large rotations can be idealised through the plastic mechanism indicated in Figure 5.6b. L is defined as the reverse channel width, c and i as the bolt pitch dimensions depicted in Figure 4.1, R_m as the radius of the circular portion of the yield line mechanism, t_c as the reverse channel thickness, σ_y as the material yield strength, and d_h and d_{bh} as the bolt hole and bolt head diameters, respectively. On this basis, the plastic capacity of the reverse channel in tension B_{RC} can be derived as:

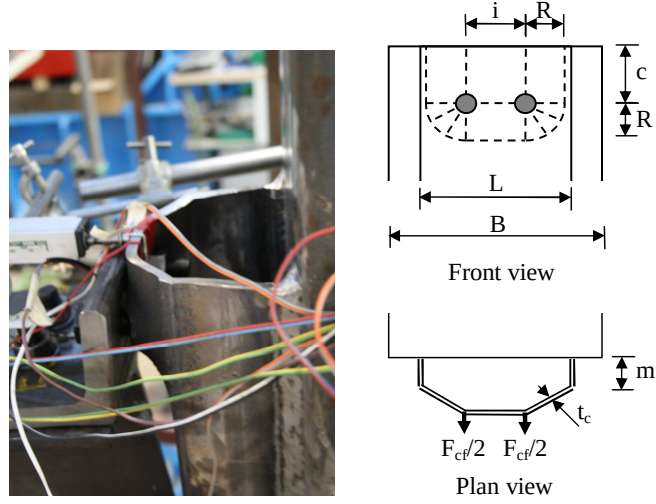
$$B_{RC} = \frac{\sigma_y t_c^2}{2} \left(\frac{2R_m - d_h}{2R_m - d_{bh}} \right) \left[\pi \frac{d_h - R_m}{d_h - 2R_m} + 2 \frac{i + 2c - d_h}{2R_m - d_h} \right] \quad (5.18)$$

where R represents:

$$R = \frac{L - i - t_c}{2} \quad (5.19)$$

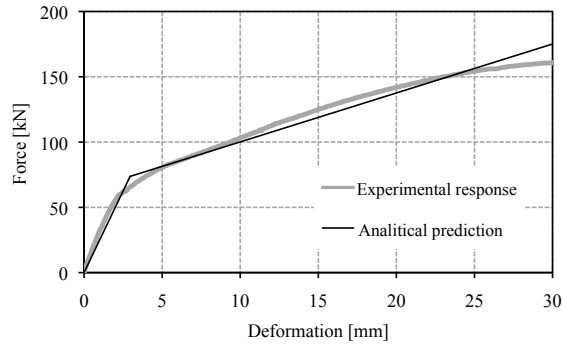
Additionally, the stiffness of the column face can be determined through the relationship proposed for the initial stiffness of Hollow-section faces as presented in Equation 5.6. Finally, assuming a post elastic stiffness of 15% of the initial stiffness as suggested for components of similar out-of-plane plate behaviour [117], a bi-linear characterization of the force-displacement relationship of the reverse channel component can be

5. MECHANICAL MODELS



(a) Reverse channel plastic deformation observed in Specimen T6.3-A15-d40-M.

(b) Idealized plastic mechanism for the reverse channel component.



(c) Analytical prediction of channel behaviour in Specimen T6.3-A15-d40-M.

Figure 5.6: Force-deformation characterization of reverse channel component.

constructed. Such bi-linear relationship is presented in Figure 5.6c and compared with the experimental data. This comparison demonstrates that the above described modelling assumptions appear to be representative.

With regards to the multi-linear bolt stiffness idealization, the model proposed by Swanson and Leon [145] was used for idealizing the full axial force-displacement relationship of standard bolts located in the vertical (column side) leg.

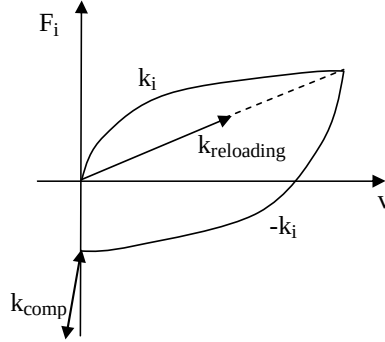


Figure 5.7: Hysteretic load-displacement response of bolt-row under cyclic loading.

5.2.11 Hysteretic rules and overall joint response

Figure 5.7 depicts the hysteretic model assumed in the present study for a bolt-row. A non-linear characterisation of the horizontal force-displacement relationship for a single bolt-row is possible by means of the incremental mechanical model described previously. Hence, the loading stiffness (denoted k_i in Figure 5.7) can be evaluated. The unloading branch of the curve presented in Figure 5.7 can be obtained by assuming a force-displacement relationship equal to two times the initial loading branch in accordance with conventional practice [60, 139]. Once zero displacement is reached (i.e. contact between the column face and the angle is fully re-established), additional compression forces will push the column face inwards with a column face compression stiffness k_{comp} taken as that previously proposed for tension effects in Equation 5.6 for simplicity. Additional re-loading stages (denoted $k_{reloading}$ in Figure 5.7) are always assumed to start from the origin and to develop linearly towards the maximum displacement reached previously in order to account, in a simple manner, for the stiffness degradation effects observed during the tests.

In the case of connections with web angle components, the hysteretic rules described above are applied for each bolt-row component forming the multi-spring joint model considered in Figure 5.1b. Under either monotonic or cyclic loading, compatibility and equilibrium conditions are then imposed in order to obtain the full moment-rotation response at the overall joint level. This solution process involves the evaluation of the equilibrium of forces at a number of rotation increments $\Delta\theta$. Equilibrating moment-rotation points are identified by binary interpolation and successively stored until the specified final rotation is reached.

5.3 Comparison with test results

The mechanical model proposed in the previous section was validated against the results of the experimental programmes described in Chapters 3 and 4. Figure 5.8 presents the comparisons between the moment-rotation curves predicted by the mechanical model and the experimental results for Hollo-bolted connections under monotonic loading, whilst Figure 5.9 presents the same comparisons for reverse channel configurations. It should be noted that the bolt slippage component was insignificant and can therefore be ignored when simulating the response of *top and seat* angle connections under monotonic loading as the slip onset forces were reached at relatively large rotations in the tests. On the other hand, slip behaviour can occur comparatively early in the case of *top, seat and web* angle connections and hence needs to be adequately represented in the model by incorporating the actual clearance (typically 1 - 2 mm). This was particularly evident for Specimen B10-G8.8-d40-M (see Figure 5.8i) where the difference between the experimental and analytical responses can be attributed to the gradual slippage occurring between rotations levels of 5 and 18 mrad. In contrast, slippage is considered to occur instantaneously in the mechanical model once the assumed friction resistance is reached.

As demonstrated in Figures 5.8 and 5.9, the mechanical model predictions are in close agreement with the experimental results. The model is able to represent the key response parameters and features of the blind-bolted and reverse channel connections, namely the initial stiffness, connection capacity, post-yield hardening and onset of membrane effects at large displacements. The initial stiffness and capacity are predicted within an accuracy of under 5% in all cases. Some observed discrepancy in the response at large rotation levels (more than 40 mrad) can be attributed to modelling idealization of the material hardening. It is worth noting that Specimen A10-G10.9-d40-M in Figure 5.8g reached the Hollo-bolt ultimate deformation capacity at a moment of 43 kNm both in the test and in the mechanical model prediction.

Figures 5.10 and 5.11 depict the comparative hysteretic curves for Hollo-bolted and combined channel/angle connections, respectively. Only the response between the fifth and ninth cycle is presented for clarity. It is evident from Figures 5.10 and 5.11 that the mechanical model provides a reasonably good prediction of the hysteretic response including strength degradation effects. The degradation in stiffness during the initial stages of the unloading/reloading phases is, nevertheless, slightly under-predicted for

5. MECHANICAL MODELS

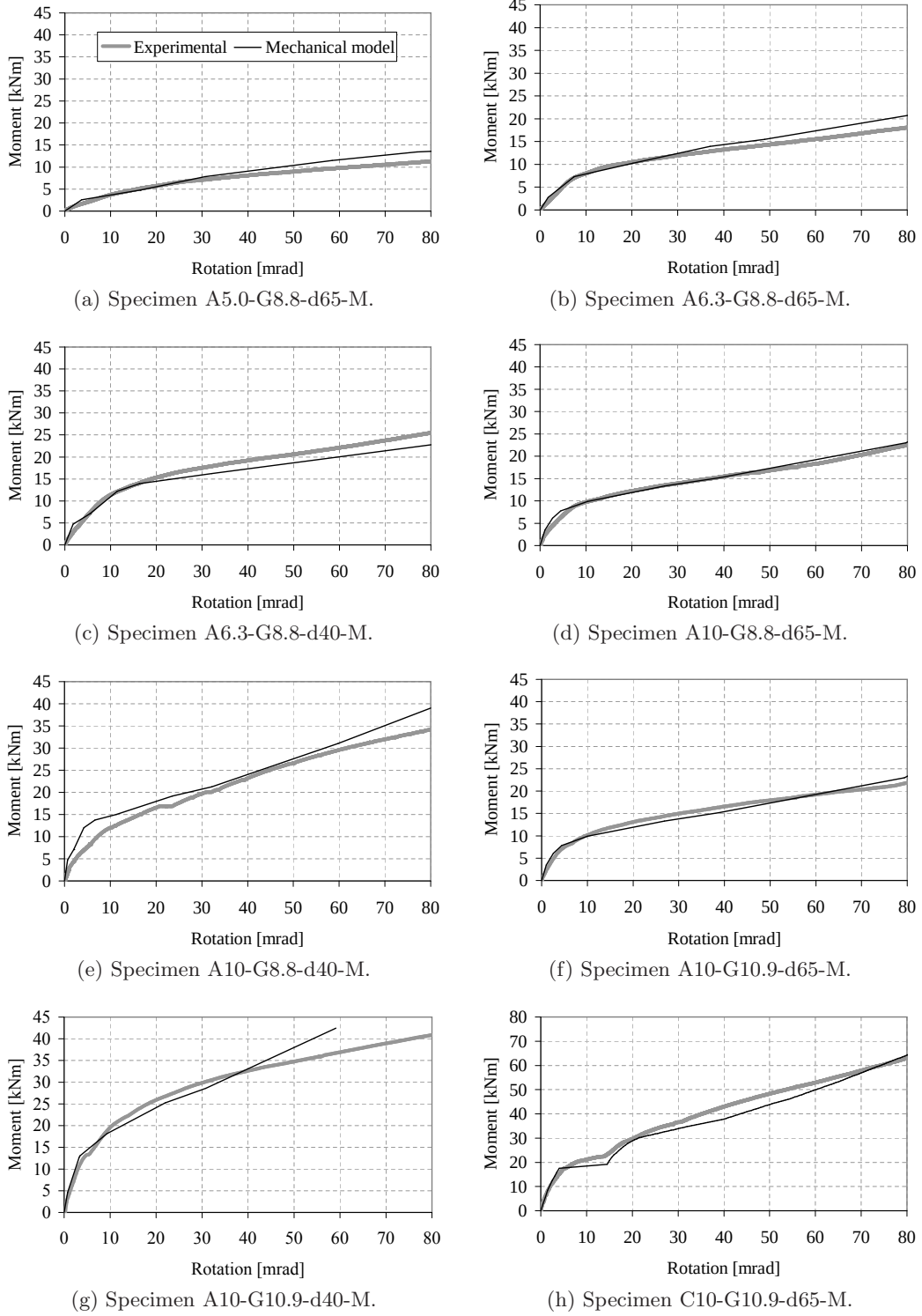


Figure 5.8: Comparison of experimental and analytical response of Hollo-bolted angle connections under monotonic loading.

5. MECHANICAL MODELS

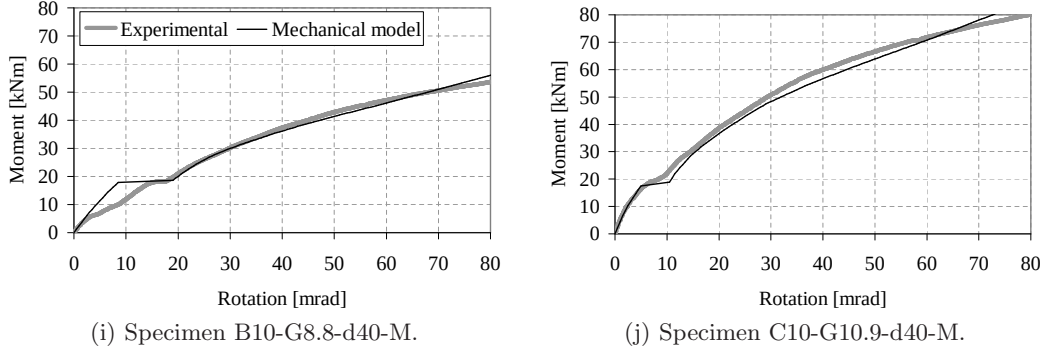


Figure 5.8: Comparison of experimental and analytical response of Holo-bolted angle connections under monotonic loading (Cont.).

Holo-bolted connections (notably for Specimen A6.3-G8.8-d40-C in Figure 5.10c) and over-predicted for some reverse channel details (e.g. Specimen W10-A8-d40-C in Figure 5.11e). This is attributed to the idealisations considered in the adopted hysteretic rule for the bolt-row (see Figure 5.7). In general, however, the mechanical model captures the main features of the cyclic response.

5.4 Parametric assessment of Holo-bolted angle connections

Having gained confidence in the reliability of the detailed component model, a number of parametric and sensitivity studies were carried out in order to investigate the influence of key factors on the stiffness, capacity and ductility of blind-bolted connections. As mentioned before, the behaviour of reverse channel connection lacks the complications brought forward by the complex interactions caused by the Holo-bolt intrinsic flexibility and thus will not be studied in this section. Furthermore, in order to facilitate the interpretation of results, the response of a single bolt-row is examined herein. The selected bolt-row consists of a top angle blind-bolted to a tubular RHS column by means of two Holo-bolts and attached to the beam through two M16 standard bolts. The distance between the centrelines of the Holo-bolts (i.e. dimension i with reference to Figure 3.2) is set to 45 mm and Grade S275 steel is used. All other geometric parameters and material properties are varied within expected practical ranges.

Specific assessments, which offer additional insights into the behaviour or provide useful information for calibrating more simplified models, are selected and discussed below.

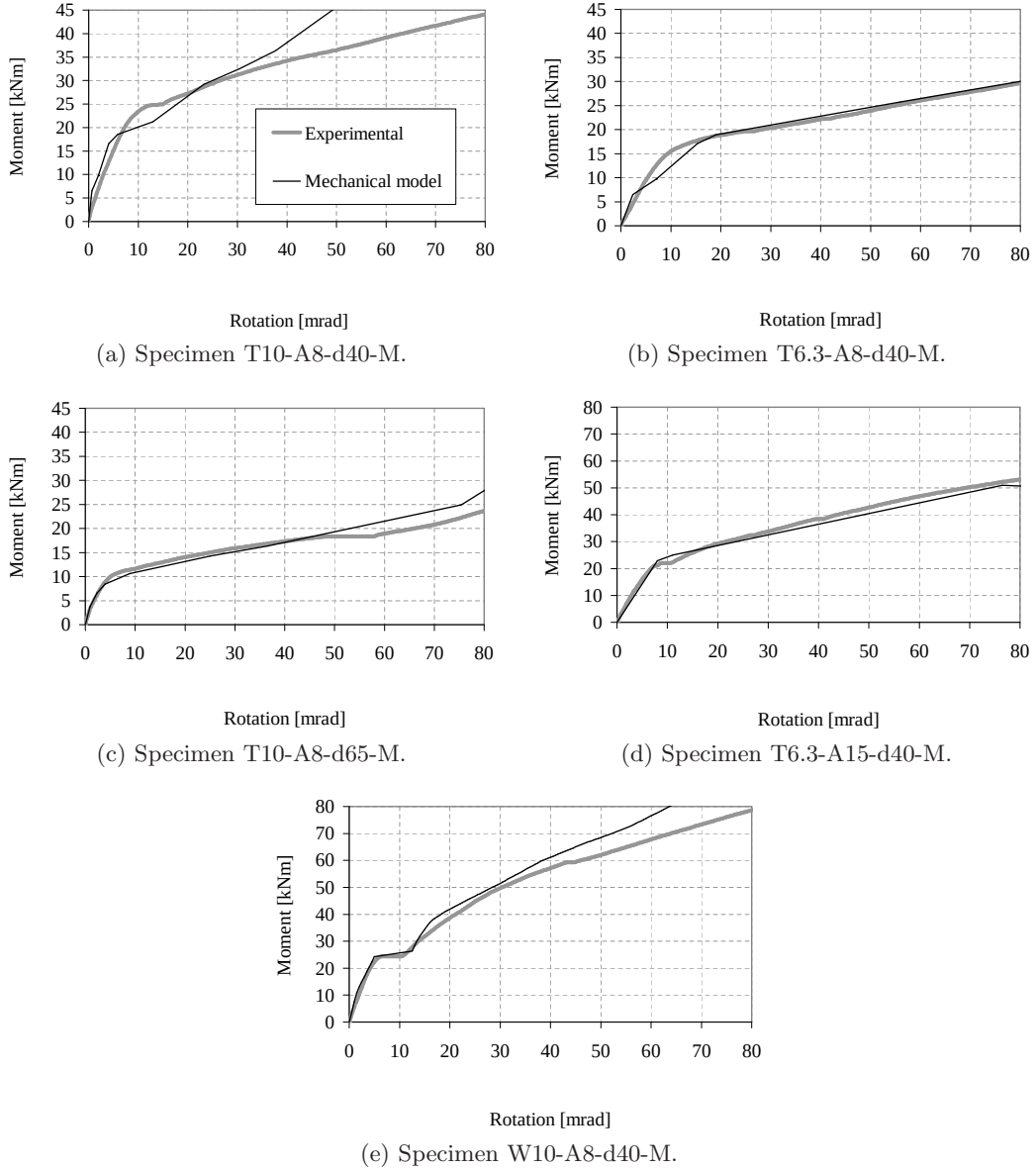
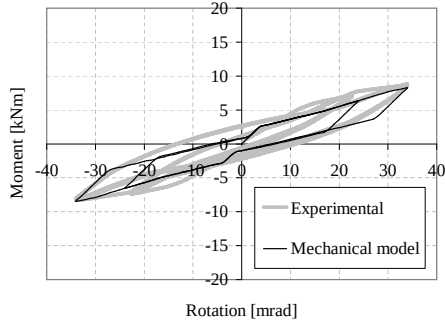


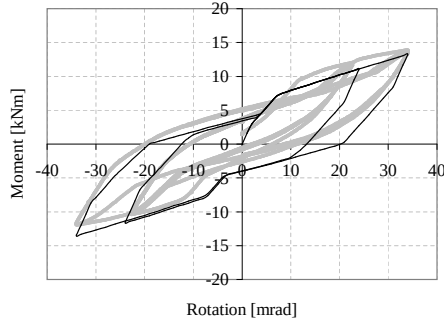
Figure 5.9: Comparison of experimental and analytical response of combined channel/angle connections under monotonic loading.

This is carried out with reference to Figures 5.12 to 5.17 in which the bolt-row force (F_i) against the bolt-row displacement (v), as well as the separation of the Hollo-bolt relative to the column face, are presented.

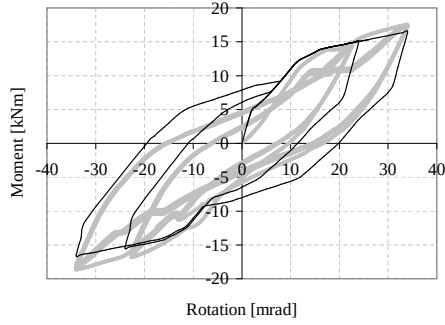
5. MECHANICAL MODELS



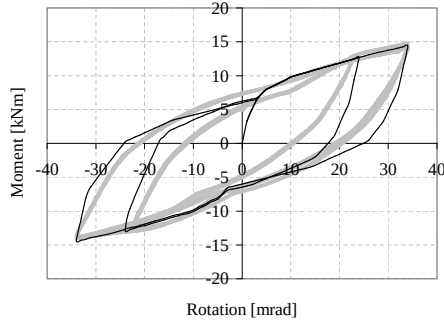
(a) Specimen A5.0-G8.8-d65-Y.



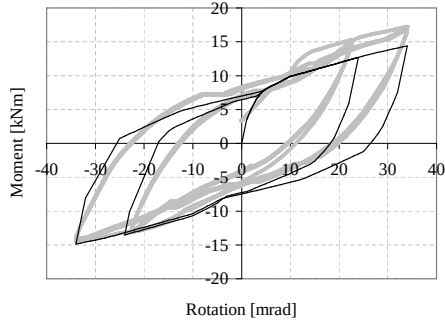
(b) Specimen A6.3-G8.8-d65-Y.



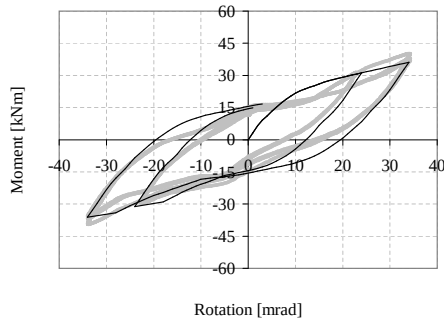
(c) Specimen A6.3-G8.8-d40-Y.



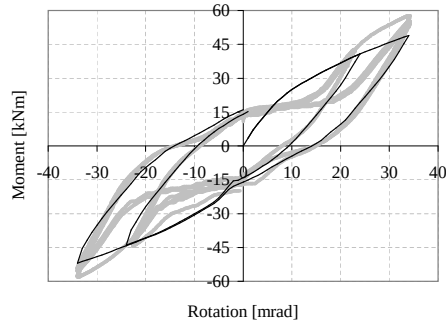
(d) Specimen A10-G8.8-d65-Y.



(e) Specimen A10-G10.9-d65-Y.



(f) Specimen C10-G10.9-d65-Y.



(g) Specimen C10-G10.8-d40-Y.

Figure 5.10: Comparison of experimental and analytical response of Hollo-bolted angle connections under cyclic loading.

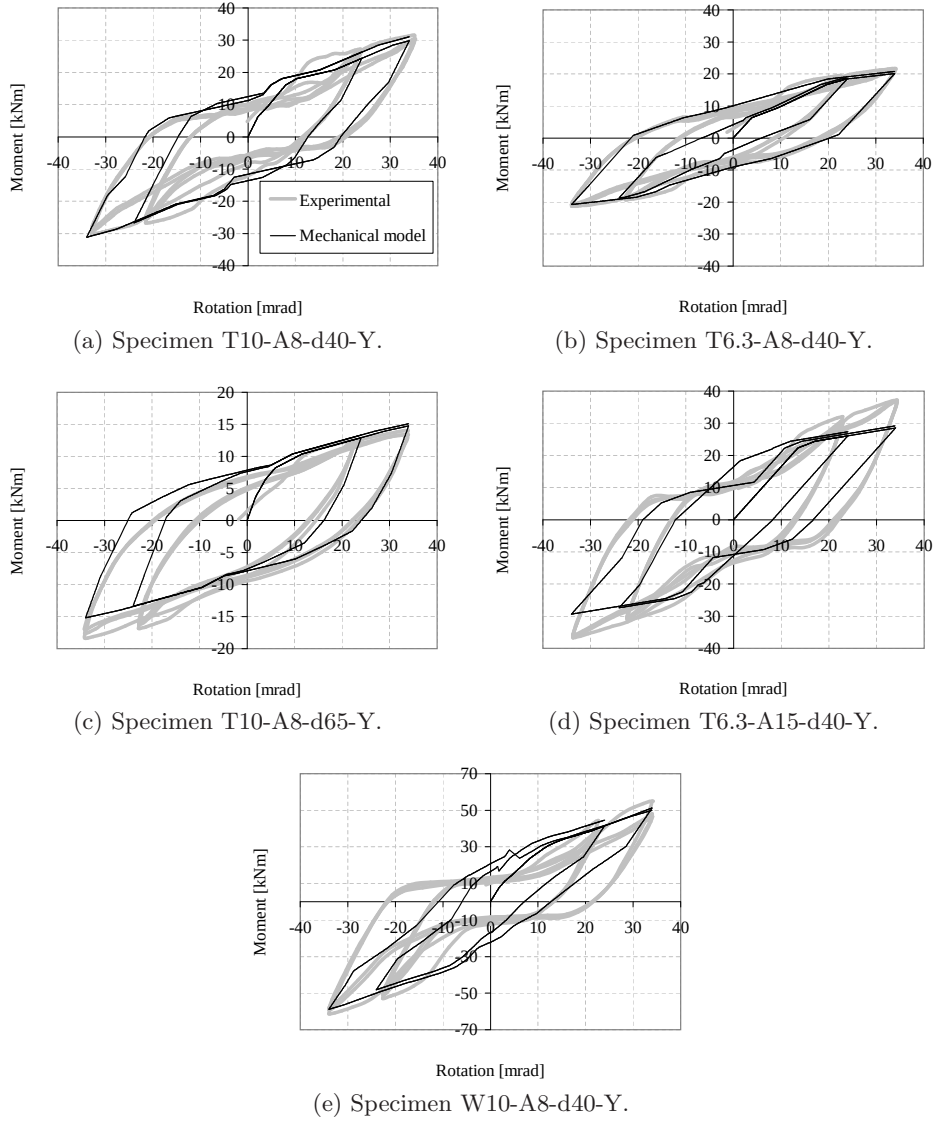


Figure 5.11: Comparison of experimental and analytical response of combined channel/angle connections under cyclic loading.

5.4.1 Angle thickness and Hollo-bolt grade

The influence of the angle thickness is examined in Figures 5.12 and 5.13 which present the results for blind-bolted connections between a 150x18 SHS column and a 100x75 angle with varying thickness. Other geometric dimensions of Figure 3.2 are set as $a=45$ mm, $b=30$ mm, $c=35$ mm, $d=65$ mm and $h=100$ mm, and M16 Hollo-bolts of Grade 10.9 or 8.8 are used. It is worth noting that whenever the Hollo-bolt deformation capacity is reached, the corresponding force-displacement curve is terminated. This occurs

5. MECHANICAL MODELS

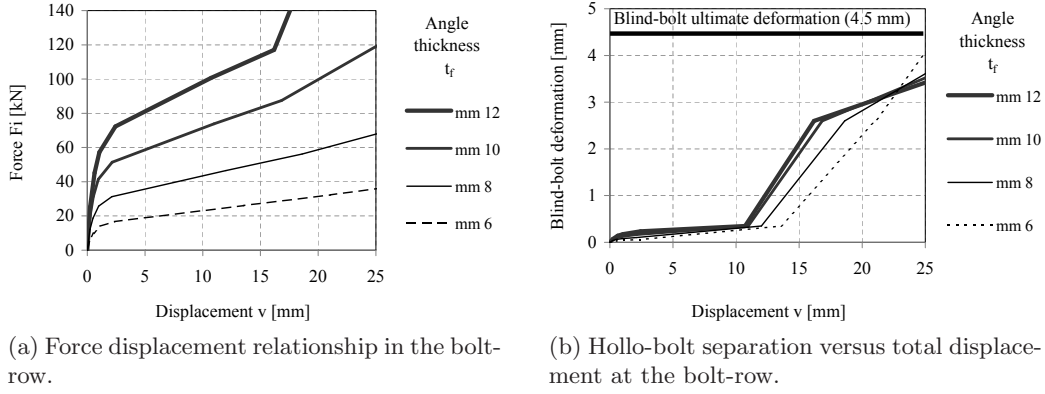


Figure 5.12: Influence of angle thickness for Grade 10.9 Hollo-bolts.

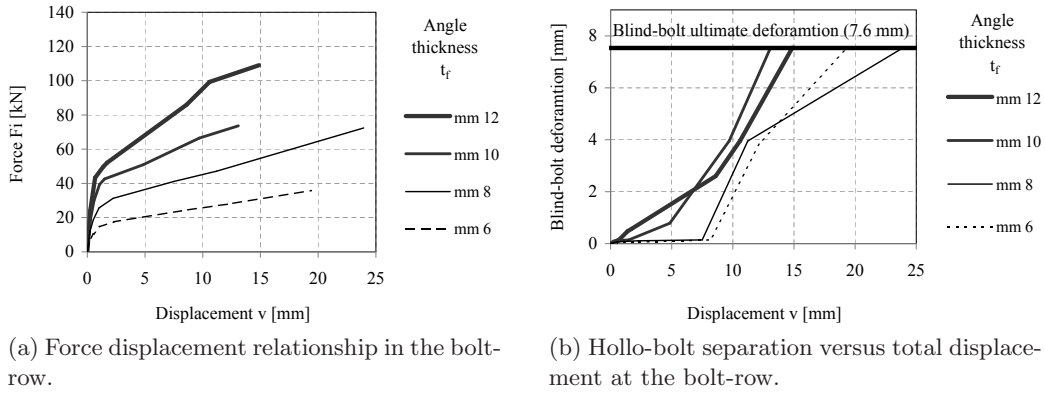


Figure 5.13: Influence of angle thickness for Grade 8.8 Hollo-bolts.

at blind-bolt deformations equal to 4.5 mm for Grade 10.9 Hollo-bolts and 7.6 mm for Grade 8.8. As expected, the angle thickness has a significant and direct effect on the bolt-row capacity which increases proportionally with its increase.

Largely similar force-deformation responses are obtained irrespective of the Hollo-bolt grade, and the direct relationship between angle thickness and yield strength is maintained by comparing Figures 5.12a and 5.13a. The main difference, however, is that of the reserve capacity as can be more easily observed in Figures 5.12b and 5.13b. Grade 10.9 bolts yield at bolt-row displacements of about 11-13 mm and attain fracture conditions only at large angle displacements (over 25 mm). Grade 8.8 Hollo-bolts, on the other hand, start accumulating displacements well before and reach ultimate deformation capacity (assumed as approximately 7.5 mm) at angle displacements as low as 12 mm.

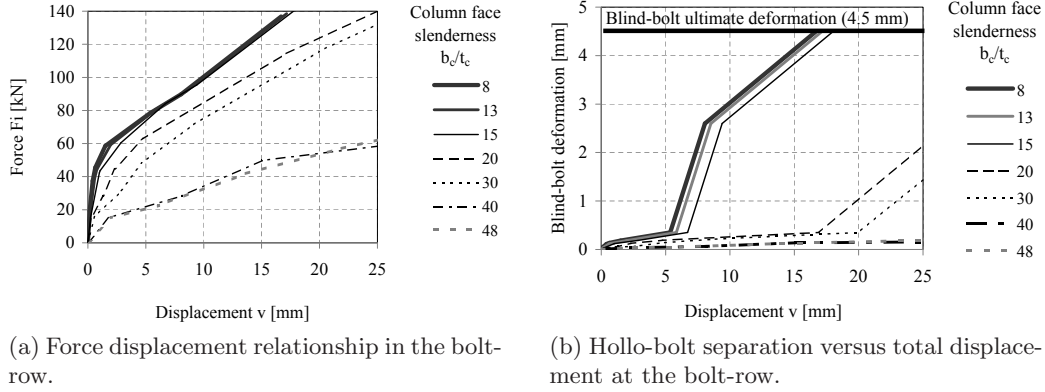


Figure 5.14: Influence of column face slenderness (b_c/t_c) for Grade 10.9 Holo-bolts.

In addition to the above, when Grade 10.9 Holo-bolts are employed, the bolt-row displacement associated with yielding in the blind-bolt (between 11 and 13 mm) is only slightly affected by the change in angle thickness as shown in Figure 5.12b. This is due to the improved stiffness characteristics of Grade 10.9 Holo-bolts caused by the allowance of higher tightening torque. In contrast, when Grade 8.8 Holo-bolts are used, the relationship between global bolt-row displacement and blind-bolt yielding is more involved (as can be appreciated from Figure 5.13b). This has direct implications on the reliability of possible expressions for ductility assessment and verifications of serviceability displacement limits in the Holo-bolts.

5.4.2 Column face flexibility

The column face flexibility is characterised here by means of its slenderness, defined as the ratio between the width b_c and thickness t_c of the face of the structural hollow section under consideration. The bolt-row analysed is formed by a 100x75x8 mm angle connected with the shorter leg to the column, hence the dimensions of Figure 3.2 are maintained as $a = 50$ mm, $b = 50$ mm, $c = 35$ mm, $d = 40$ mm. The column section is varied between SHS of dimensions 150x18, 200x16, 150x10, 200x10, 300x10, 200x5 and 300x6.3, resulting in ratios of b_c/t_c of 8, 13, 15, 20, 30, 40 and 48, respectively. Figure 5.14 depicts the bolt-row force versus bolt-row displacement as well as the blind-bolt separation versus bolt-row displacement for Grade 10.9 Holo-bolts. Figure 5.15 shows the corresponding results for Grade 8.8 Holo-bolts.

The direct influence of the column face flexibility on both the connection stiffness and

5. MECHANICAL MODELS

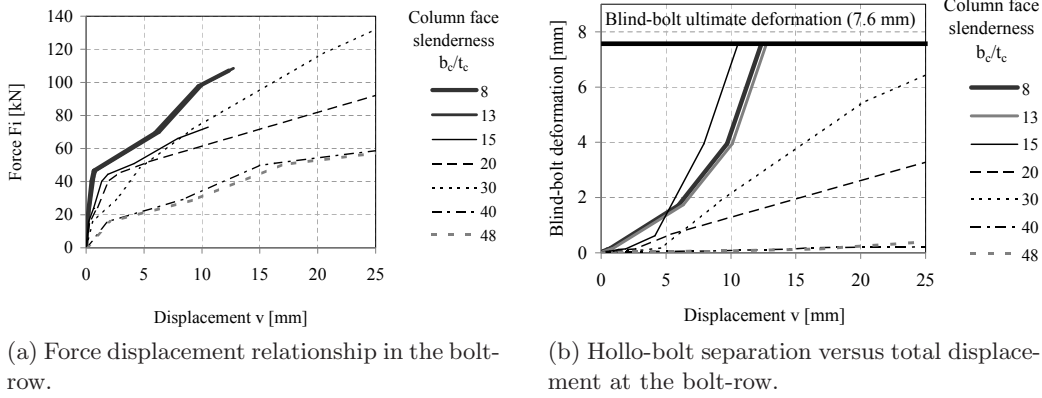


Figure 5.15: Influence of column face slenderness (b_c/t_c) for Grade 8.8 Hollo-bolts.

capacity is evident from Figures 5.14 and 5.15. Stiffer column faces will cause inelasticity to be developed either in the angle or ultimately in the blind-bolts, whereas relatively flexible column faces will yield first subsequently reducing the strength demands imposed on other connection components, but at the expense of lower connection capacities. It is important to note, however, that significant column face yielding is associated with high strain hardening and ductility capacities and can form a desirable failure mode when initial stiffness considerations are not the primary concern. In this respect, the use of Grade 10.9 Hollo-bolts offers a favourable behaviour as the onset of column face yielding can be more clearly defined. This is corroborated by comparing Figures 5.14b and 5.15b, where yielding of the column face occurs for b_c/t_c values of 20 or greater for Grade 10.9 Hollo-bolts. In contrast, the comparatively early yielding for Grade 8.8 Hollo-bolts initiates intricate component interactions that can render the definition of such behavioural boundaries more complex and less reliable.

5.4.3 Gauge distance

In order to assess the influence of the gauge distance, a set of connections between a 150x10 SHS column and various angle sections is considered herein. The angle sections utilised are 75x100x10, 100x75x10, 125x75x10, 150x75x10 and 200x100x10 mm, hence the distance d (see Figure 3.2) varies between 40, 65, 85, 115, and 165 mm, respectively. The load-deformation curves at the bolt-row level are shown in Figures 5.16a and 5.17a, whilst the curves for the blind-bolt separation versus the bolt-row deformation are depicted in Figures 5.16b and 5.17b.

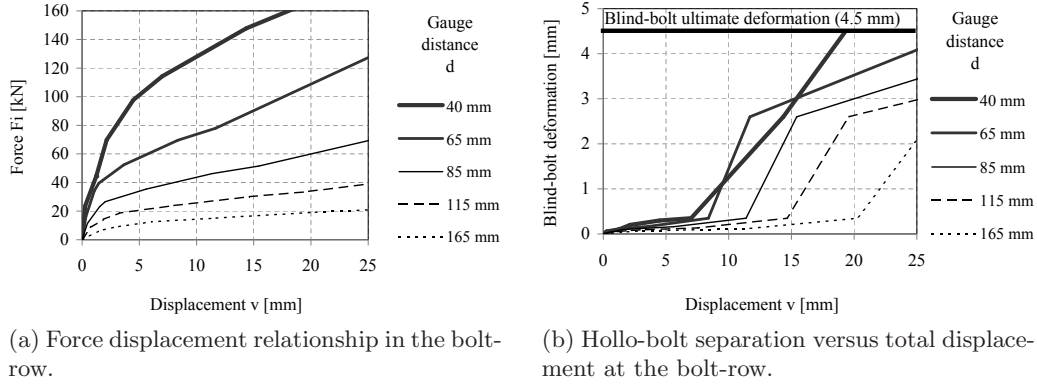


Figure 5.16: Influence of gauge distance d for Grade 10.9 bolts.

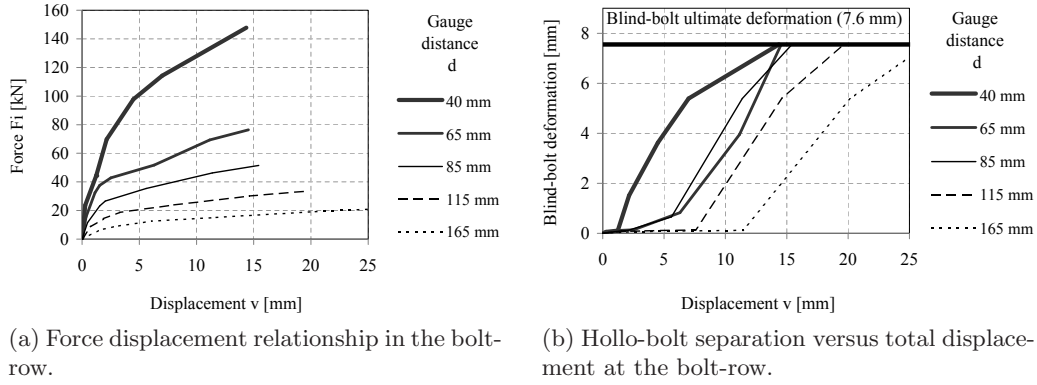


Figure 5.17: Influence of gauge distance d for Grade 8.8 bolts.

As shown in Figures 5.16a and 5.17a, as the gauge distance d increases, the stiffness (and capacity) of the angle connection decreases irrespective of the grade of the blind-bolt used. More flexible angle components also cause the blind-bolt yield forces to be achieved at proportionally larger displacements. This direct relationship between the gauge distance and overall displacement at the yield of the blind-bolt is maintained as indicated in Figures 5.16b and 5.17b (with the yielding occurring relatively earlier in Grade 8.8 bolts as expected).

5.5 Design considerations for Holo-bolted angle connections

The detailed component model described in this chapter can be used for assessing accurately the behaviour of connections in non-linear analytical simulations of framed structures or for conducting further parametric examinations. However, for practical

5. MECHANICAL MODELS

design purposes, there is a need for more simplified procedures. Accordingly, the observations and findings discussed in previous sections are used herein to propose simplified expressions that are suitable for practical design, and to highlight various design implications of blind-bolted angle connections. Particular focus is given to the evaluation of the initial stiffness, yield capacity, post-elastic stiffness and failure mode. For this purpose, the mechanical model described previously in this chapter, was used to generate additional data on key parameters. This information was subsequently used to propose simplified expressions for the force-displacement response at the bolt-row level as described in the following sections.

5.5.1 Initial stiffness

For determining the initial stiffness, a direct application of the initial stage of the mechanical model described previously is proposed. This initial stiffness corresponds to k_{Iee} in Figure 5.4 and can be simplified as:

$$k_{Iee} = k_i = \frac{12EI \left(3EI + \left(\frac{1}{1/k_{cf} + 1/k_{Hb}} \right) c'^2 (c' + 3d') \right)}{12EI (c' + d')^3 + \left(\frac{1}{1/k_{cf} + 1/k_{Hb}} \right) c'^2 d'^3 (4c' + 3d')} \left(1 - \frac{d'}{3(d' + a)} \right) \quad (5.20)$$

where I is the moment of inertia of the longitudinal section of the angle (i.e $pt_f^3/12$), c' and d' are the distances defined by Equations 5.3 and 5.4, a is the gauge distance of the horizontal leg of the angle measured as indicated in Figure 3.2, k_{cf} is the column face stiffness which can be evaluated from Equation 5.6, while k_{Hb} represents the Holo-bolt initial stiffness which can be obtained from the force-displacement values reported in Table 5.1. In order to ensure conservative estimates, Equation 5.20 includes a factor that considers the influence of the horizontal leg flexibility on the overall initial stiffness [138].

5.5.2 Failure mode

A reliable prediction of the plastic mechanism in blind-bolted angle connections is essential in order to ensure an adequate structural performance. In particular, the im-

plications of the dominant mechanism on the capacity and ductility of the blind-bolted connection should be appropriately considered. In this context, a clear definition of the necessary conditions for the attainment of a specific plastic mode can ensure the elimination of undesirable failure mechanisms. Based on the analytical results and experimental observations obtained within the present study, the following considerations with regards to failure mode control can be identified:

- The use of Grade 10.9 Holo-bolts is useful in delaying the onset of yielding in the blind-bolt, while at the same time ensuring a more reliable definition of the angle deformation patterns. This, considered alongside the fact that the bolt-separation at the initial deformation stages (Stages k_{See} and k_{Seey} in Figure 5.4) was mostly observed when Grade 8.8 bolts were used, makes it desirable to use Grade 10.9 Holo-bolts instead of the conventional Grade 8.8.
- As expected, the column face flexibility has a direct influence on the stiffness and capacity of blind-bolted angle connections. In general, two main related failure states are possible: (i) concentration of deformation in the column face, for more flexible column faces, and (ii) deformation mainly concentrated in other joint components with subsequent failure of the Holo-bolt at large deformation demands for stiffer column faces. The column face slenderness values (b_c/t_c) for which concentration of the deformation in the column face is expected are found to be greater or equal to 20 for Grade 10.9 Holo-bolts. Also, Equation 5.7 could be compared with the yield strengths reported for Holo-bolts (e.g. Table 5.1) in order to assess the likelihood of column face yielding around the Holo-bolt insert.
- Provided Grade 10.9 Holo-bolts are employed, relationships of the form depicted in Figure 5.18 can be used to estimate the bolt-row displacement at which yielding of the blind-bolt occurs. This may result in undesirable separations between the blind-bolt nut and the column face, which is likely to violate serviceability requirements. As illustrated in Figure 5.18, such bolt-row displacement limit is found to be a function of the ratio ht_f/d'^2 where h is the angle width (from figure 3.2), t_f the angle thickness and d' the gauge distance as defined before.

5.5.3 Bolt-row capacity

The plastic axial force (F_{pi}) at the bolt-row is largely a function of the plastic moment capacity of the angle leg (m_{pla}), as defined by Equation 5.16, and the gauge distance

5. MECHANICAL MODELS

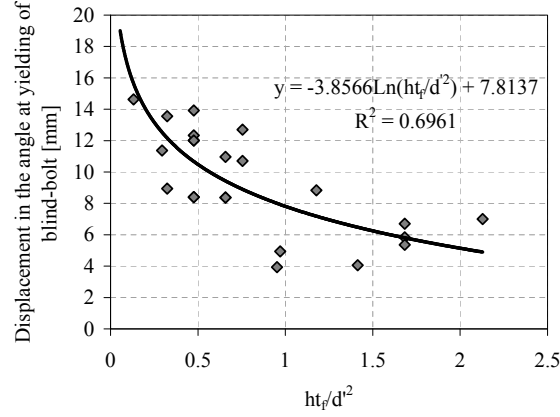


Figure 5.18: Total bolt-row displacement at which blind-bolt yield would be expected as a function of ht_f/d'^2 for M16 Grade 10.9 Holo-bolts.

d' . As discussed before, the number and location of plastic hinges result from complex interactions between the blind-bolt, angle and column components. However, provided that (as discussed in Section 5.5.2): (i) the deformation in the Holo-bolts is limited, and (ii) the capacity is not mainly governed by column face yielding, then a general simplified equation of the following form could be sought:

$$F_{pi} = \alpha F_{pg} = \alpha \frac{2m_{pla}}{d'} \quad (5.21)$$

Figure 5.19 presents the values of α as a function of column face slenderness b_c/t_c for different values of d'/t_f . Using linear curve fitting and linear interpolation, the general equation for the determination of the value of F_{pi} can be defined as:

$$F_{pi} = \left(\alpha_1 - 0.02 \frac{b_c}{t_c} \right) \frac{2m_{pla}}{d'} \quad (5.22)$$

where

$$\alpha_1 = 1.11 + 0.1 \frac{d'}{t_f} \quad (5.23)$$

In situations where restrictions on detailing results in the column face governing the connection moment capacity, alternative expressions which are provided elsewhere [117] can be adopted.

An appropriate representation for the post-yield stiffness of the bolt-row can also be derived. Figure 5.20 depicts the values of the post-yield stiffness coefficient defined

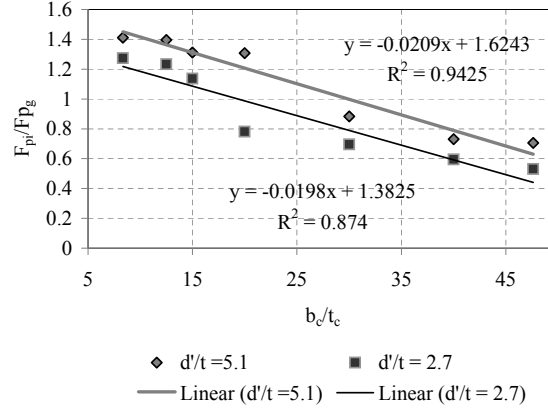


Figure 5.19: Relationship between F_{pi}/F_{pg} (as defined by Equation 5.22) and b_c/t_c for different d'/t_f ratios and M16 Grade 10.9 Hollo-bolts.

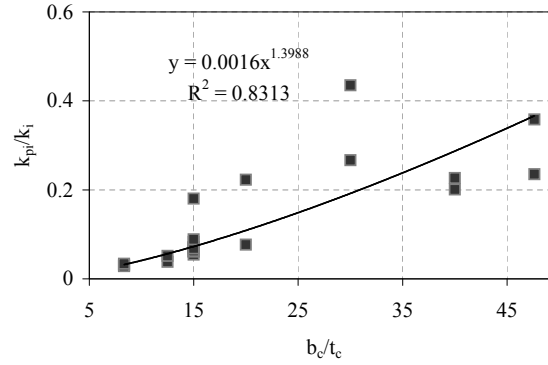


Figure 5.20: Relationship between strain hardening stiffness (k_{pi}) and initial stiffness (k_i) as a function of the column face slenderness (b_c/t_c).

as the ratio between the strain hardening k_{pi} and initial stiffness k_i as a function of column face slenderness b_c/t_c . Following a curve fitting procedure as before, a simplified equation for estimating the strain hardening stiffness is suggested as:

$$k_{pi}/k_i = 0.0016 \left(\frac{b_c}{t_c} \right)^{1.4} \quad (5.24)$$

5.5.4 Overall moment-rotation response

The previous sections provided relationships for determining the stiffness and capacity of the constituent bolt-rows within a *Top and seat* or *Top, seat and web* angle blind-bolted connection. The connection moment-rotation response can now be determined based on such relationships. A simplified component model such as that suggested in

5. MECHANICAL MODELS

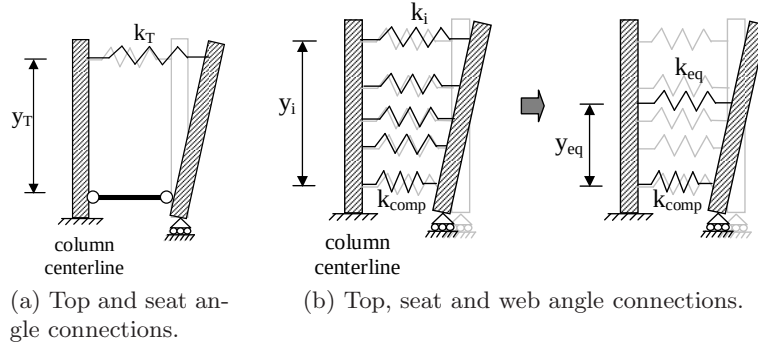


Figure 5.21: Design-oriented simplified mechanical model of angle connection.

Eurocode 3 Part 1.8 [27] is adopted for top and seat angle connections, and the extension proposed by Pucinotti [129] is adopted for top, seat and web angle connections, as indicated in Figure 5.21b. For simplicity, a pre-defined point of rotation can be assumed at the horizontal leg of the bottom angle for top and seat angle connections. Hence, the initial rotational stiffness of the connection is given by:

$$S_T = k_T y_T^2 \quad (5.25)$$

where k_T is the stiffness of the top angle bolt-row and y_t is the distance between the top angle bolt-row and the horizontal leg of the bottom angle.

On the other hand, when web angles are included, the higher capacities reached and the additional rotational restraint imposed by the web cleats can alter the point of rotation. This effect would be aggravated if the compression contact region is substantially smaller than the tubular column width [45]. A mechanical model that considers the column face compression stiffness k_{comp} (which in this study was taken equal to the tension stiffness k_{cf} defined in Equation 5.6), and replaces the web and top angle rows by an equivalent spring with stiffness k_{eq} , can be constructed (Figure 5.21b). In this case, for connections with top, seat and web angle components, the following expression can be derived for S_T [129]:

$$S_T = \frac{y_{eq}}{1/k_{comp} + 1/k_{eq}} \quad (5.26)$$

in which k_{eq} and y_{eq} are given by:

$$k_{eq} = \frac{\sum k_i y_i}{y_{eq}} \quad (5.27)$$

$$y_{eq} = \frac{\sum k_i y_i^2}{\sum k_i y_i} \quad (5.28)$$

where k_i is the stiffness of the i^{th} bolt-row, and y_i defines the location of the bolt-row with reference to the assumed point of rotation. It is also important to note that stiffness values may depart from linearity at early loading stages when large forces are coupled with significant column face deformations (which may be particularly important in the compression zones of web angle connections as is the case of Specimen B10-G10.9-d40-M). In such cases, and as pointed out in the experimental study of Chapters 3 and 4, this undesirable behaviour can be designed-out by either employing wider beam flanges or adding plates so as to ensure adequate load distribution to the stiffer parts of the tubular column.

Similarly, the rotation at the attainment of plastic capacity in the top angle (i.e. F_{pi} calculated from Equation 5.22) will be:

$$\theta_{pi} = \frac{v_{pi}}{H} \quad (5.29)$$

where H is the summation of the beam height (D in Figure 3.2) and the angle thickness t_f , while $v_{pi} = F_{pi}/k_i$. Assuming elastic behaviour in the remaining bolt-rows, the plastic moment can be evaluated as:

$$M_y = F_{pi}H + \sum_{j=1}^{n-1} F_j y_j \quad (5.30)$$

where j is the number of remaining bolt-rows and F_j is the axial force in the j^{th} bolt-row. The bilinear moment-rotation relationships proposed above are compared with experimental results in Figure 5.22. Specimen A5.0-G8.8-d65-M which involved yielding of the column face at an early stage, and Specimen B10-G8.8-d40-M where significant localised deformations occurred in the compression zone of the column, represent undesirable behaviour which does not conform with the assumptions of the proposed simplified expressions and are hence excluded from the comparisons in Figure 5.22.

In general, as illustrated in Figure 5.22, reasonably good predictions of the response are obtained by the simplified bi-linear model, with more favourable estimations obtained for longer angle gauge distances. The connection initial stiffness is well predicted in all cases, whereas conservative estimates of the capacity are obtained for stiffer angles

5. MECHANICAL MODELS

(shorter gauge distances). The only exceptions is Specimen C10-G10.9-d40-M where a discrepancy with the yield capacity of the specimen is evident. This is attributed to the significant slippage of the standard bolts which occurred in this test, an effect which cannot be captured by the simplified model. Accordingly, provisions should be made to limit the extent of slippage in practice to ensure compliance with the conventional range of clearances for standard bolts (e.g. 1 to 2 mm) over which the simplified expressions have been validated.

5.6 Design considerations for reverse channel connections

This section discusses the key parameters and criteria that can be of direct relevance to practical design application of Reverse Channel connections. It also offers a simplified assessment of possible models for the evaluation of the connection stiffness and capacity.

5.6.1 Connection initial stiffness

In order to examine the stiffness characteristics of reverse channel connections with angles, the simplified component-based model of top and seat angle connections suggested in Eurocode 3 Part 1.8 [27] and the extension for connections with web angle components proposed by Pucinotti [129] (Figure 5.21) are considered herein. The simplified mechanical model is similar to that previously presented in Equation 5.25, although in this case the bolt row stiffness is calculated from the contribution of each individual connection component assumed to act in series. Hence, the initial rotational stiffness of the connection is given by:

$$S_T = k_T y_T^2 = \sum 1/K_j^{-1} y_T^2 \quad (5.31)$$

where K_j is the stiffness of the j^{th} component and y_T is the distance between the top angle bolt row and the horizontal leg of the bottom angle. The simplified model can be fully defined by considering the reverse channel component characterization of Section 5.2.10 together with available information on the behaviour and modelling of angle components. In particular, the component characterization suggested in Eurocode 3 [27] was used here, as discussed below.

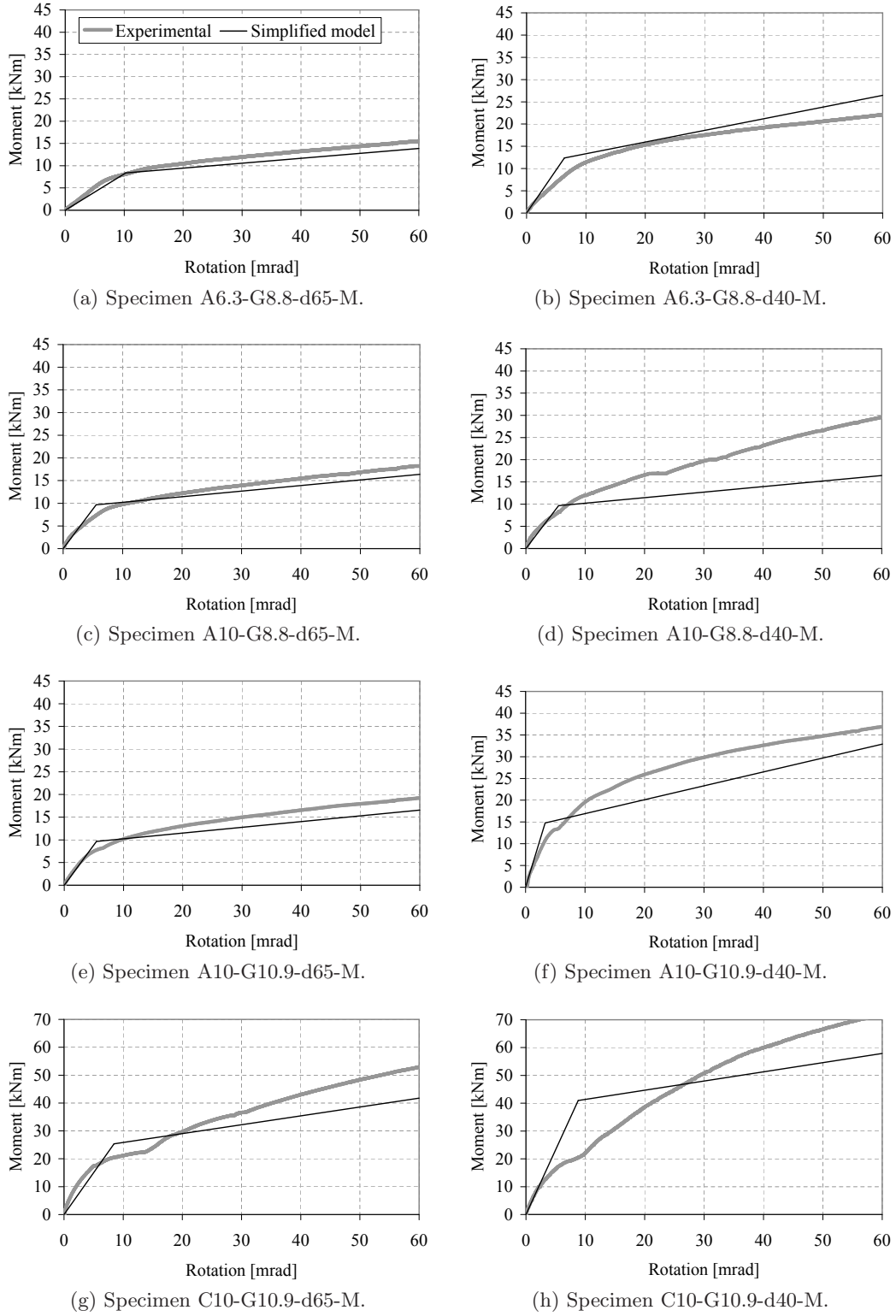


Figure 5.22: Comparison of experimental results and predictions of the more simplified design-oriented component model for monotonic tests on Hollo-bolted angle connections.

5. MECHANICAL MODELS

5.6.2 Connection capacity

With regards to capacity, a first approach, often leading to conservative estimates of capacity, would be to ignore the plasticity in the horizontal leg of the angle assemblage. This leads to the following set of expressions, which are analogous to the equivalent T-stub concept suggested in Eurocode 3 [27]:

$$M_{y1} = \min (M_{A,Rd}, M_{bA,Rd}, M_{b,Rd}) \quad (5.32)$$

in which the yield capacity of the connection M_{y1} is determined as the smallest value obtained from Equations 5.33, 5.34 or 5.35 corresponding to the different plastic patterns as follows:

a) Mode 1: formation of plastic mechanism in the angle:

$$M_{A,Rd} = \frac{2m_{pl}}{d'}y \quad (5.33)$$

b) Mode 2: mixed failure mode involving yielding of the bolt and a plastic hinge in the angle:

$$M_{HbA,Rd} = \frac{2m_{pl} + \sum B_y c'}{c' + d'}H \quad (5.34)$$

c) Mode 3: Bolt yielding:

$$M_{Hb,Rd} = \sum B_y H \quad (5.35)$$

where B_y is the smaller of the yield force in the bolt, or in the reverse channel component as defined by Equation 5.18; m_{pl} is the plastic moment capacity of the longitudinal angle section, and c' and d' are the effective gauge distances (Equations 5.3 and 5.4 with ratio k_{HB}/k_g replaced by k_{SB}/k_g).

Figure 5.23 presents comparisons between the experimental monotonic moment-rotation curves and the bi-linear estimations obtained by means of the expressions described in this section. A post-yield hardening stiffness of 5% was used in accordance with the experimental data. It is clear from Figure 5.23, that the suggested simplified procedure provides adequate estimations of the stiffness and capacity in all cases, provided appropriate gauge lengths are employed. The comparisons are particularly encouraging for specimens T6.3-A8-d40-M and T10-A8-d65-M. Nevertheless, the inclusion of web

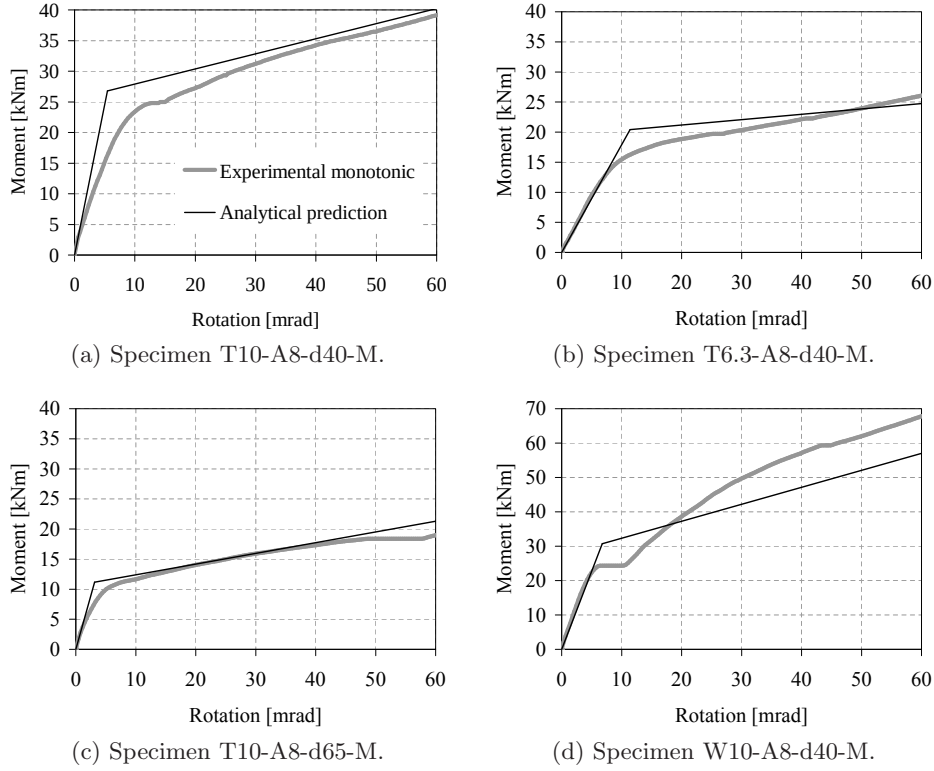


Figure 5.23: Comparison of experimental and predicted moment-rotation relationships for monotonic tests on combined channel/angle connections.

angles in Specimen W10-A8-d40-M induce complex interactions between the connection components, hence requiring more detailed models to trace the full response more accurately; nonetheless, the initial stiffness is well predicted and the proposed simplified expressions offer a reasonable lower bound estimate of the moment capacity in the connections. This underestimation in the joint moment capacity is attributed to the fact that at the attainment of yielding in the top angle component (i.e. the point at which the capacity is calculated in the simplified model) there is still significant reserve in capacity that is supplied by the web angle components at later stages.

5.7 Validation of fatigue damage models

The data obtained from the test results (Chapters 3 and 4) provide useful information for the validation of detailed damage models [7, 10]. In such models, fatigue life is usually expressed as a function of the plastic strain in the fracture zone - a value that

5. MECHANICAL MODELS

is not always easy or practical to measure accurately. Alternatively, fatigue damage models based on the connection rotation have been proposed [104, 105]. Mander et al. [104] developed fatigue life relationships for top and seat angle connections based on the effective joint plastic rotation. The model was subsequently extended to deal with cycles of variable amplitude [105]. These models were validated for rotation levels below 40 mrad.

To assess the suitability of the models discussed above, Figures 5.24 5.25, 5.26 and 5.27 present the fatigue life analysis for Specimens A10-G10.9-d65-Y, T10-A8-d40-Y, T6.3-A8-d40-Y and T10-A8-d65-Y, respectively. In these figures, the effective rotation θ_{jp} is defined as the net joint plastic rotation induced by plastic deformation in the angles only (i.e. discounting elastic rotations as well as rotations induced by the deformations in the column face or reverse channel component). Figures 5.25b, 5.26b and 5.27b depict the strain histories obtained from strain gauge measurements closest to the zone of fracture. No strain measurements were obtained for Specimen A10-G10.9-d65-Y.

Two fatigue damage models are compared in Figures 5.24b, 5.25c, 5.26c and 5.27c. Miner's model considers linear damage accumulation obtained by summation of the damage caused by individual cycles, such that:

$$D_T = \sum D_i = \sum \frac{1}{N_f} \quad (5.36)$$

where D_T is the total accumulated damage, D_i is the damage fraction corresponding to cycle i , and N_f is the number of cycles to failure defined by Mander et al. [105] as:

$$\theta_{jp} = 0.0849 (2N_f)^{-0.333} \quad (5.37)$$

The second fatigue damage model presented in Figures 5.24b, 5.25c, 5.26c and 5.27c corresponds to an energy based approach [105] for which the damage $D(t)$ at time t is expressed as:

$$D(t) = \sum \left[\left(\frac{M_i + M_{i-1}}{2} \right) (\theta_{ji} - \theta_{ji-1}) \frac{\theta_{jpi}}{0.0229M_{jp}} \right] \quad (5.38)$$

where i is the data point in the cyclic history, θ_{jpi} is the effective plastic equiamplitude rotation defined as the effective plastic rotation minus the rotation obtained from the cyclic moment-rotation envelope, whilst M_i and θ_i are the observed moment and rotation, respectively, at instant i .

It can be observed from Figures 5.25c, 5.26c and 5.27c that the energy-based model gives more favourable results than the linear Miner's rule damage accumulation for all cases in comparison with the experimental results. This seems to suggest the validity of the range of applicability of the selected models for rotation amplitudes higher than the 40 mrad for which the models were originally proposed.

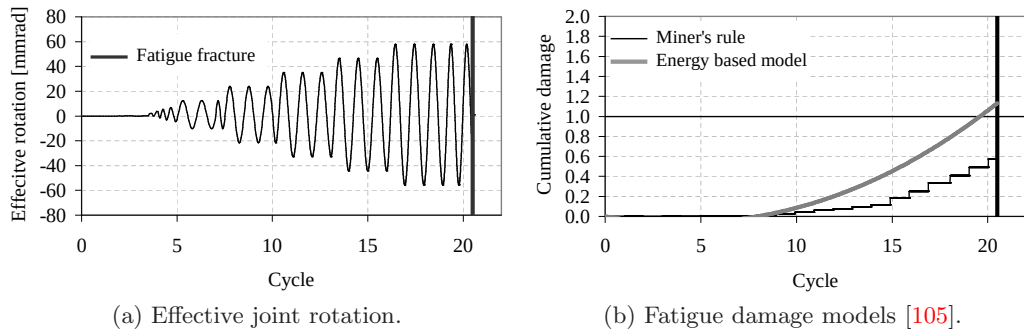


Figure 5.24: Fatigue damage analysis for Specimen A10-G10.9-d65-Y.

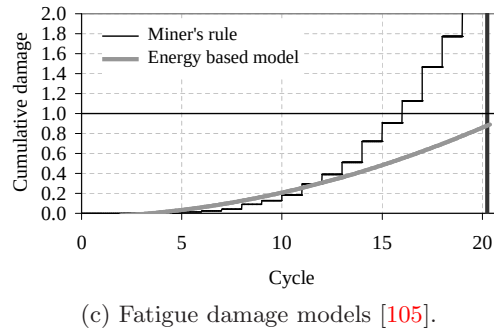
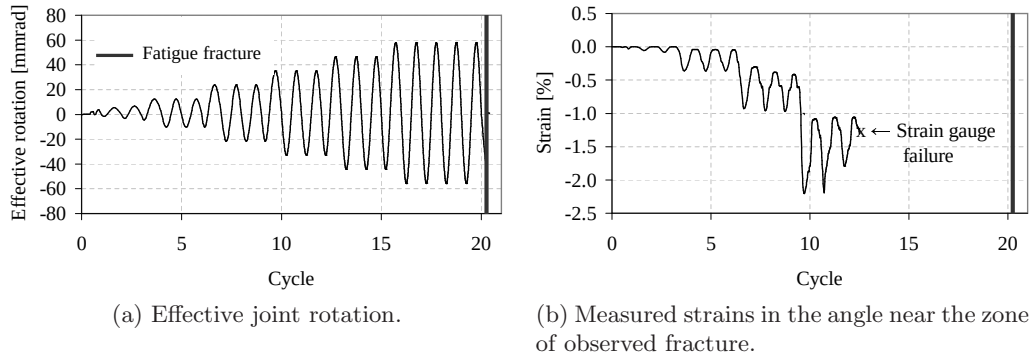


Figure 5.25: Fatigue damage analysis for Specimen T10-A8-d40-Y.

5. MECHANICAL MODELS

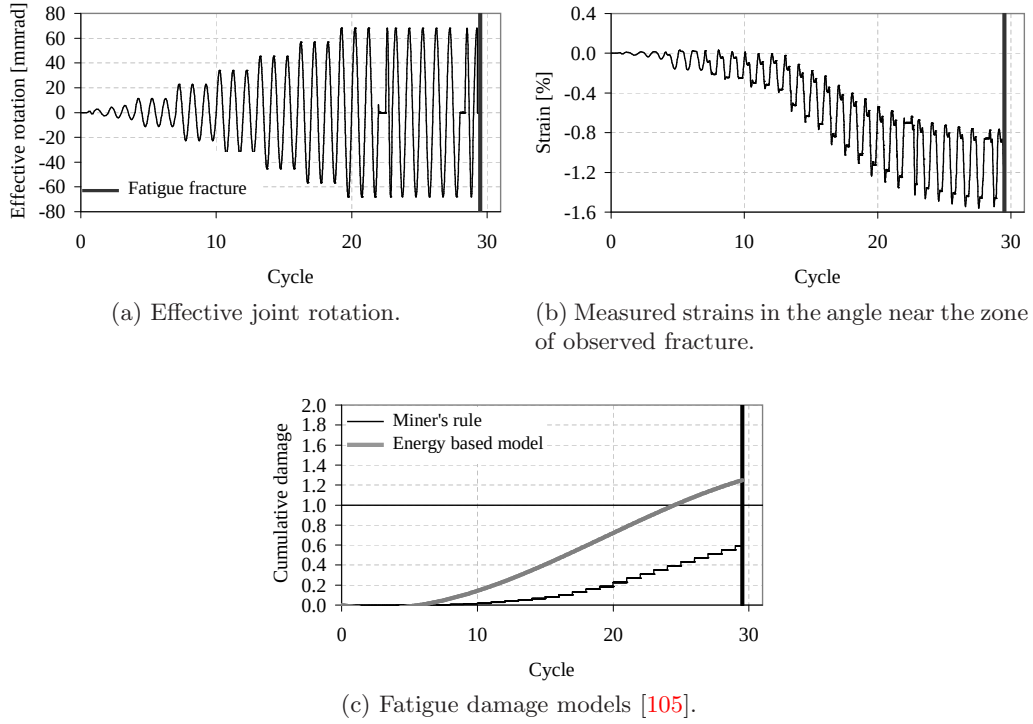


Figure 5.26: Fatigue damage analysis for Specimen T6.3-A8-d40-Y.

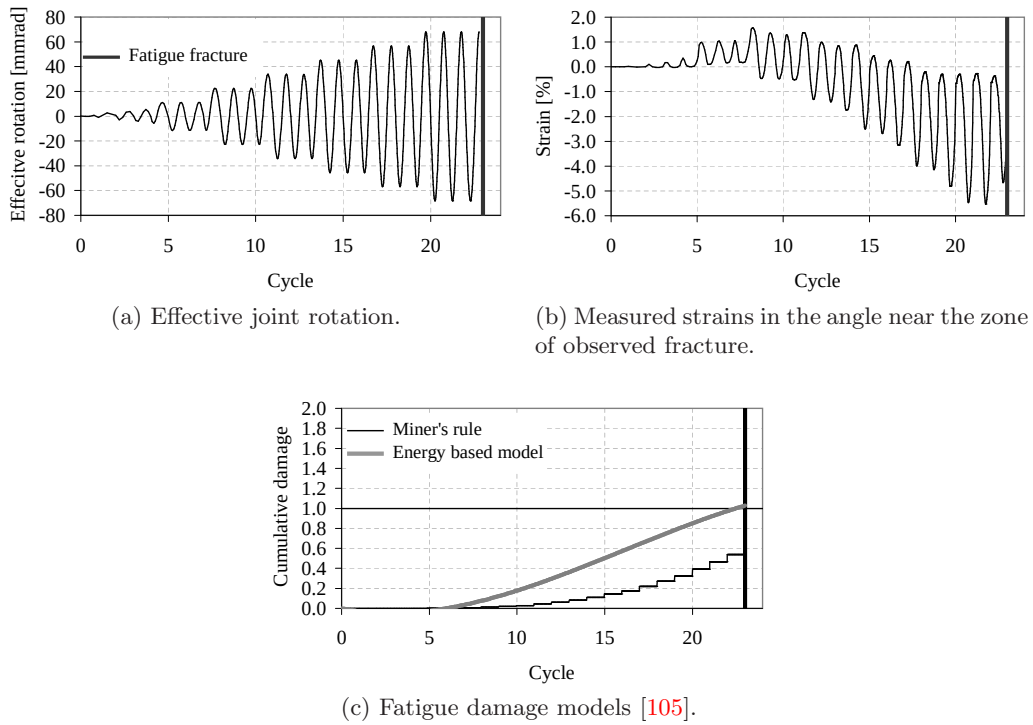


Figure 5.27: Fatigue damage analysis for Specimen T10-A8-d65-Y.

5.8 Concluding remarks

This chapter has proposed a detailed mechanical model for blind-bolted and combined reverse channel/angle connections between open beams and tubular columns. The model is based on the component approach and utilises a multi-linear representation in order to trace the full monotonic and cyclic response of *top and seat* and *top, seat and web* angle connections. The validity of the model has been assessed against the experimental results of Chapters 3 and 4 and the detailed model was found to provide reliable predictions of the response for the range of connection configurations considered. In particular, the initial stiffness and capacity are predicted within an accuracy of under 5% in all cases. Besides, although the detailed mechanical model proposed in this chapter has been developed from fundamental mechanical concepts and should, in principle, be applicable to a wide range of angle connection configurations; this needs to be examined further in future validation studies covering different connection arrangements as more experimental datasets become available.

A set of parametric assessments was carried out using the detailed model, with the aim of examining the key factors influencing the behaviour of semi-rigid blind-bolted connections. It was shown that the Holo-bolt grade, angle thickness, column face flexibility and gauge distance can have a significant influence on the stiffness and capacity of the connections. Particular attention was given to tracing the development of the force and deformation within the blind bolts thus enabling the assessment of the available reserve of strength as well as the verification of serviceability limits. Importantly, it was observed that the improved initial stiffness characteristics of Grade 10.9 Holo-bolts enable a more consistent definition of the failure mode and can enhance the reliability of blind-bolted connections.

In order to provide preliminary information for the purpose of practical design implementation, simplified expressions for estimating the stiffness and capacity of blind-bolted angle connections were proposed based on a data-set generated from the detailed mechanical model and considering the salient parameters that were identified. The suitability of the simplified expressions for representing the connection response was also validated against the experimental results.

Similarly, after the characterization of the reverse channel force-deformation relationship, simplified component models for estimating the initial stiffness and moment ca-

5. MECHANICAL MODELS

capacity of combined channel/angle connections were assessed. In general, it was shown that simplified procedures adapted from the equivalent T-stub approach employed in Eurocode 3 [27], in conjunction with the use of appropriate gauge distances, provide reliable estimates of the connection stiffness and capacity. To this end, this work constitutes a fundamental step towards the provision of codified design procedures for blind-bolted and reverse channel connections.

It is important to note that the connection configurations considered herein represent average levels of stiffness and relatively modest capacity ranges. When conventional classification conventions [27] are used, the specimens considered in this study would lie in the lower half of the *semi-rigid/partial-strength* category. Clearly, besides the specific connection details, such classification would strongly depend on the attached beam. If the approach adopted in Eurocode 3 [27] is used, the connection stiffness would be normalised to that of the beam expressed as EI_b/L_b , where I_b is the second moment of area of the beam and L_b is the length of the beam, respectively. In turn, the connection capacity would be normalised with respect to the plastic capacity of the beam. In this case, the boundary between the semi-rigid/partial-strength and the flexible/pinned classifications is suggested in Eurocode 3 [27] as 0.5 in terms of normalised stiffness, and 0.25 in terms of normalised capacity. Considering possible practical ranges of I_b and L_b and the results obtained in this study, the normalised stiffness falls between approximately 1 and 2.5 for top and seat angles only, and increases to above 4 when web angles are included. In terms of normalised yield capacity, the range would be between 0.15 and 0.3 for top and seat angle connections and can reach 0.6 when web angles are incorporated. Overall, the insights obtained from the refined mechanical models and the simplified analytical treatments, coupled with the experimental findings of previous chapters, highlight the suitability of this form of semi-rigid/partially-restrained connections for secondary or braced primary frame systems, depending on the structural configuration and loading conditions under consideration. Subsequent parts of this thesis provide further discussions and assessments of the seismic response of these systems.

Part III

Frame Behaviour

Chapter 6

Evaluation of Inelastic Seismic Demands

6.1 Introduction

As mentioned before in the literature survey (Chapter 2), modern performance-based seismic design and assessment methodologies hinge on the reliable determination of structural deformations [5]. To this end, considerable research has already been carried out into the probabilistic distribution of peak inelastic displacements under several suites of ground-motions [17, 23, 114, 134, 135, 136, 152, 154]. To date, most of the available studies offer statistical results based primarily on elastic-plastic single-degree-of-freedom (SDOF) systems. Importantly those studies that include pinching behaviour incorporate simultaneously severe levels of strength deterioration typical of reinforced concrete members [65, 116, 136, 143, 152]. In contrast, the partially-restrained (PR) connections studied in the previous part of this thesis (Chapters 3 to 5) do not present significant deterioration in strength up to considerably high levels of deformation demand. Similarly, it can be shown that the response of concentrically-braced (CB) systems can differ significantly from other pinching models when subjected to low or moderate levels of seismic demand, highlighting the necessity of employing dedicated models for studying the response of CB structures.

In light of the above discussion and the discussions presented in Chapter 2 (Section 2.3), this chapter seeks to improve the understanding of the deformation response of SDOF systems representative of commonly used steel structures, with the aim of informing their seismic assessment. Therefore, the objective of this chapter is threefold:

6. INELASTIC DEMANDS

- (i) Firstly, to offer a detailed characterization of inelastic demands in steel structures with particular emphasis on pinching SDOFs representative of PR and CB steel frames.
- (ii) Secondly, to assess the influence of different frequency content parameters on the inelastic demands of SDOF systems simulating typical steel structures with both bi-linear and pinching hysteresis.
- (iii) Finally, and on the basis of the findings of the previous two stages, propose simplified expressions for the assessment of displacement demands in steel structures in the form of non-iterative equivalent linearisation procedures based on secant periods.

6.2 Structural systems and earthquake ground-motions

Bi-linear (i.e. elastic-plastic) systems are used in this study as benchmark models for comparison purposes. In addition, the response of SDOF models representative of PR and CB structures is considered, for which the characteristics of the structural models employed are described below.

6.2.1 PR Pinching Model

The Modified Richard Abbott model as proposed and validated by Nogueiro et al. [122] is used here to represent the response of Partially-Restrained (PR) steel structures. The Modified Richard Abbott model is based on the alternation between two limiting curves of the Richard Abbott type [133]. As shown in Figure 6.1a, the boundary curves are characterized by their initial stiffness (k), post-elastic stiffness (k_p) and strength capacities (F_{sp} and F_{yp} for the lower and upper bound curves, respectively). Also presented in Figure 6.1a is the corresponding bi-linear approximation of the Modified Richard Abbott backbone. The pinching factor (P) is defined here as the ratio between the structural capacity during pinching intervals and the overall capacity:

$$P = F_{sp}/F_{yp} \quad (6.1)$$

Additionally, the transition from the lower to the upper curve depends on a shape parameter (t_s) defined as [122]:

$$t_s = \left(\frac{(\delta/\delta_{lim})^{t_1}}{(\delta/\delta_{lim})^{t_1} + 1} \right)^{t_2} \quad (6.2)$$

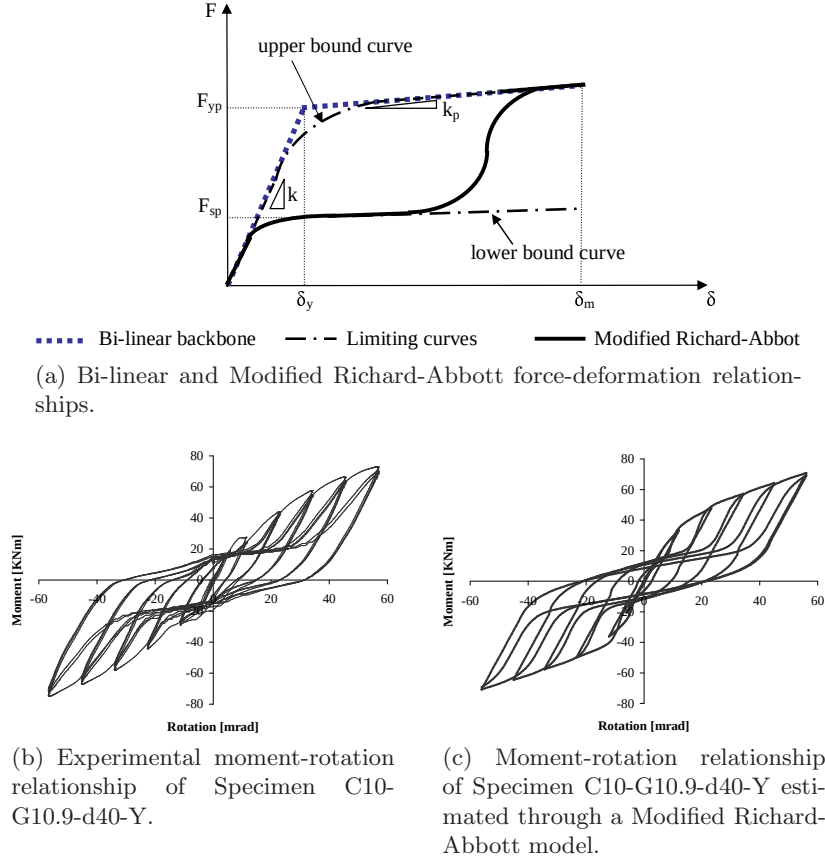


Figure 6.1: PR behaviour and modelling.

where δ is the deformation (displacement or rotation), and t_1 , t_2 and δ_{lim} are experimentally calibrated parameters. The parameter δ_{lim} is related to the maximum deformation δ_m by:

$$\delta_{lim} = C (|\delta_0| + \delta_m) \quad (6.3)$$

where $|\delta_0|$ is the absolute value of the deformation corresponding to the starting point of the current excursion, and C is a calibration parameter taken as 1 in the current study in accordance with typical values reported for several connection details [122]. Figures 6.1b and 6.1c compare the moment rotation responses of Specimen C10-G10.9-d40-Y from Chapter 3 and its respective PR model.

6. INELASTIC DEMANDS

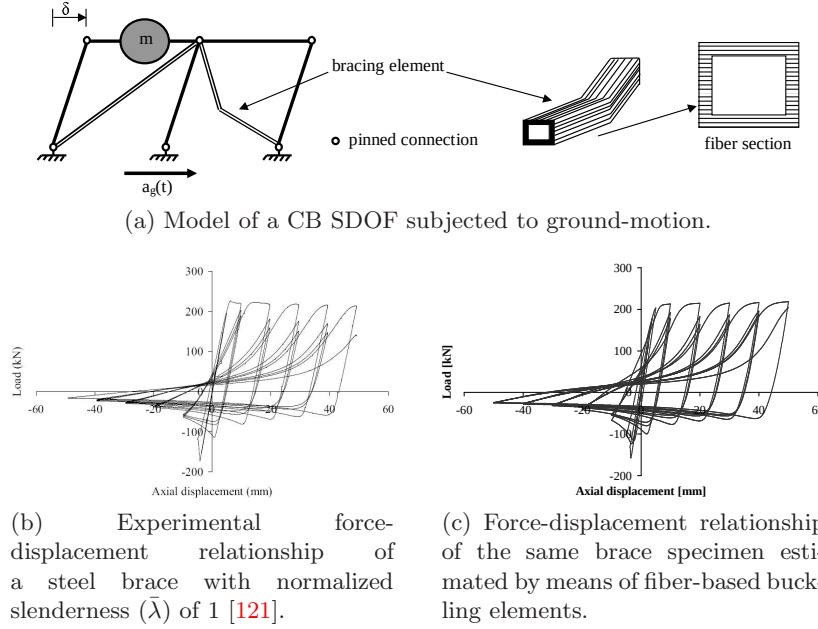


Figure 6.2: CB behaviour and modelling.

6.2.2 CB Pinching Model

Figure 6.2 presents the SDOF model used to study the response of Concentrically-Braced (CB) frames. The idealized SDOF consists of pin-connected rigid members forming a 1-storey high 2-bay CB frame. The structure is modelled in OpenSees [109] with due account for geometric and material non-linearities. The braces are modelled using fiber-based buckling elements following the approach proposed by Uriz and Mahin [149]. A typical comparison of experimental [121] and predicted axial force-displacement relationships of a brace member is also depicted in Figure 6.2. Importantly, the elastic stiffness and yield capacity of CB frames were determined from base shear-displacement relationships obtained via non-linear static (pushover) analysis with due consideration of the contribution of both tension and compression diagonals. Furthermore, these definitions of stiffness and capacity are consistently employed throughout the present chapter.

6.2.3 Ground-motions considered

A total of 100 records from 27 earthquakes with magnitudes M_w ranging from 5.65 to 7.51, and distances ranging from 6.28 to 293 Km, were used in this study. The accelera-

6. INELASTIC DEMANDS

Table 6.1: Summary of earthquake ground-motions used in this study.

Earthquake name	Magnitude	Number of records	Distance [km]		PGA [cm/s ²]		NEHRP site class
	M _w		Min.	Max.	Min.	Max.	
1992 Cape Mendocino	7.01	4	10.36	53.34	151.11	1468.85	C(2) D(2)
1986 Chalfant Valley-01	5.77	2	10.54	10.54	202.59	279.58	D(2)
1986 Chalfant Valley-02	6.19	2	14.33	14.33	392.12	438.32	D(2)
2002 Denali, Alaska [‡]	7.9	6	290.70	293.06	10.02	22.39	E(6)
1999 Duzce, Turkey [‡]	7.14	4	24.26	206.09	24.73	144.64	C(2) E(2)
1976 Friuli, Italy-01	6.5	2	20.23	20.23	308.83	344.65	C(2)
1976 Gazli, USSR	6.8	2	12.82	12.82	596.70	703.92	C(2)
1999 Hector Mine	7.13	2	52.29	52.29	143.04	186.05	C(2)
1979 Imperial Valley-06 [‡]	6.53	4	22.65	30.35	216.87	320.53	D(2) E(2)
1980 Irpinia, Italy-01	6.90	4	22.65	30.35	136.71	350.97	B(4)
1952 Kern County	7.36	2	43.39	43.39	153.00	174.41	C(2)
1995 Kobe, Japan	6.90	2	25.40	25.40	284.57	304.56	B(2)
1999 Kocaeli, Turkey [‡]	7.51	6	47.03	112.26	134.64	243.94	A(2) B(2) E(2)
1992 Landers	7.28	2	44.02	44.02	713.03	774.17	C(2)
1994 Little Skull Mtn,NV	5.65	3	14.12	30.17	116.71	208.81	B(3)
1989 Loma Prieta [‡]	6.93	15	16.51	114.87	94.64	388.07	B(3) D(2) E(8)
1990 Manjil, Iran	7.37	2	37.90	37.90	486.92	504.78	C(2)
1984 Morgan Hill	6.19	2	38.20	38.20	190.58	196.59	D(2)
1986 N. Palm Springs	6.06	2	6.28	6.28	201.08	214.06	D(2)
1985 Nahanni, Canada	6.76	2	6.80	6.80	959.25	1074.88	C(2)
2002 Nenana Mountain, Alaska [‡]	6.70	8	275.28	277.70	7.08	10.73	E(8)
1994 Northridge-01	6.69	12	18.99	45.77	110.19	1554.79	A(4) B(8)
1971 San Fernando	6.61	2	31.55	31.55	145.57	148.98	C(2)
1986 San Salvador	5.80	2	9.54	9.54	398.66	600.51	D(2)
1987 Superstition Hills-02	6.54	2	29.91	29.91	113.76	152.94	D(2)
1978 Tabas, Iran	7.35	2	55.24	55.24	819.93	835.58	B(2)
1981 Westmorland	5.90	2	20.47	20.47	152.18	237.25	D(2)

[‡] Records used for the study on the influence of frequency content parameters

tion records were obtained from the PEER-NGA database (<http://peer.berkeley.edu/nga>), and involve different site classes (according to the NEHRP classification) in order to address the impact of site conditions on the variability in inelastic response. Special attention was given to the lowest usable frequency in order to avoid undesired noise and filtering effects. Table 6.1 summarizes the catalogue of earthquakes used while a more detailed information can be found in Appendix B. The numbers in parenthesis under the heading NEHRP site class represent the number of records associated with each soil site group. Figure 6.3 presents the median acceleration response spectra normalized by Peak Ground Acceleration (PGA) for different soil classes. It is worth noting that Site Class C in the NEHRP provisions (with $360 < v_{s,30} < 760$ m/s) is broadly equivalent to Ground Type B as defined in Eurocode 8 (with $360 < v_{s,30} < 800$ m/s). Similarly, based on the limiting values of shear-wave velocity in the upper 30 metres ($v_{s,30}$), NEHRP Classes D and E are equivalent to Ground Types C and D in Eurocode 8 [28].

6. INELASTIC DEMANDS

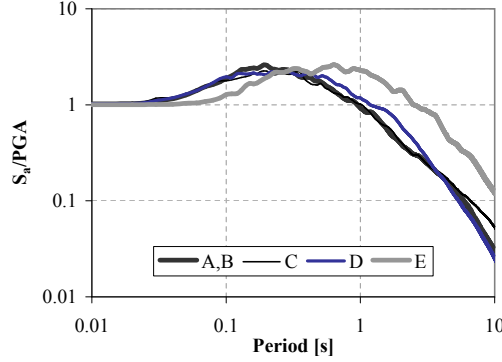


Figure 6.3: Median normalized acceleration response spectra for different soil classes.

6.2.4 Scope of parametric analysis

A statistical study was performed on the peak inelastic displacement demands for several SDOF systems with 5% viscous damping under constant strength ratio scenarios. The inelastic displacement ratio (C_R) is defined as the ratio between the peak lateral inelastic displacement ($\delta_{inelastic}$) and the peak lateral elastic displacement demand ($\delta_{elastic}$) on an infinitely elastic SDOF with the same mass and initial stiffness:

$$C_R = \frac{\delta_{inelastic}}{\delta_{elastic}} \quad (6.4)$$

$\delta_{inelastic}$ is calculated from response history analyses on structures with constant relative strength in proportion to the strength required to keep the system elastic (F_y). The constant relative strength scenarios are characterized by the strength ratio R defined as:

$$R = \frac{mS_a}{F_y} \quad (6.5)$$

where m is the mass of the system and S_a is the acceleration spectral ordinate. Four values of lateral strength ratios R were considered (i.e. $R = 2, 3, 4$ and 5).

As noted before, the shape of the pinching loop for the PR Modified Richard Abbott model depends on the pinching ratio P and two empirically calibrated coefficients (t_1 and t_2). A sensitivity study was performed in order to assess the effects of the shape coefficients t_1 and t_2 on the prediction of inelastic displacements. It was concluded that the variation of t_1 and t_2 over a typical practical range does not introduce notable differences in the estimations of mean peak displacements, and hence a constant

pair of $(t_1, t_2) = (15, 0.5)$ can be considered as representative of PR configurations and is hence used throughout this study. On the other hand, the influence of P on the inelastic displacement ratios is expected to be significant, as discussed in subsequent sections. Therefore, parametric analyses were performed for 3 levels of pinching (i.e. $P = 0.15, 0.3$ and 0.6) representative of pinching levels usually observed in PR connections. Similarly, the effects of 3 normalized slenderness values ($\bar{\lambda} = 1.3, 1.7$ and 2.1) characterizing practical ranges of conventional braced frame designs were considered for CB models. Where $\bar{\lambda}$ is defined as: $\bar{\lambda} = \sqrt{AF_y/N_{cr}}$. It should be noted that only square hollow section members of cross-section Class 1 according to Eurocode 3 [26] were used as braces. C_R ratios were calculated for a range of initial structural periods (T) between 0.10 s and 5 s. In the case of bi-linear and PR systems, intervals of 0.05 for periods less than 1.5 s were used whereas intervals of 0.2 s for structures with longer periods were employed. In the case of CB models, initial elastic periods between 0.1 and 1.5 s with 15 intervals of 0.1 s were considered while intervals of 0.2 s were used for structures with longer periods.

It is interesting to note that the monotonic force-displacement relationships of PR and CB models are broadly similar, despite being dependent on different parameters and modelling assumptions. For example, Figure 6.4 depicts the hysteretic behaviour of a PR model with $P = 0.15$ and a CB model with $\bar{\lambda} = 2.1$. The bi-linear backbone is also presented in Figure 6.4a for comparison. It can be observed from Figure 6.4a that some differences in cyclic behaviour may occur during the initial elastic stage and during loading at large inelastic displacements. Nonetheless, based on the apparent resemblance in Figure 6.4a, it would seem reasonable to consider using displacement predictions based on pinching SDOF models as proxies for CB structures. However, due to the intricate tension-compression balance of bracing members and dynamic effects, notable differences in the structural hysteresis loops may arise under earthquake loading as illustrated in Figure 6.4b. This difference in deformation estimations highlights the necessity for dedicated CB models that incorporate realistic representations of the brace tension and buckling behaviour.

It is worth noting that results for structural response parameters other than peak displacement (e.g. Fatigue damage and Park and Ang damage indices), as well as the effects of various post-elastic stiffness, have also been examined and are presented in Appendix B.

6. INELASTIC DEMANDS

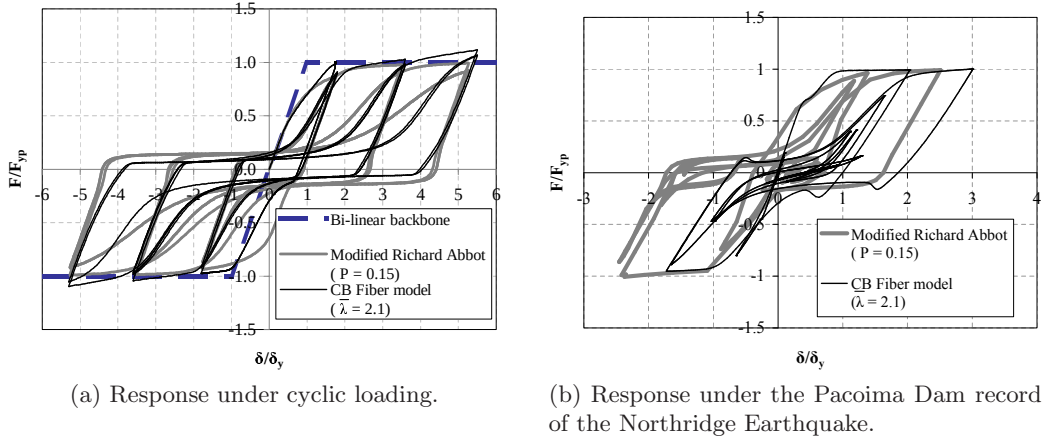


Figure 6.4: Comparison of hysteretic models of PR and CB systems.

6.3 Assessment of inelastic demands

The results of more than 120 000 response history analyses, based on the considerations and definitions described in previous sections, are presented and discussed herein. Mean inelastic displacement ratios were computed by averaging the results for each period, strength ratio and hysteretic model. On the basis of the spectral shapes depicted in Figure 6.3, a clear distinction is made between moderately stiff to stiff soils sites (Classes A, B, C and D) and soft soils sites (Class E) for purposes of presentation and discussion [135, 136]. The effects of structural model characteristics, strength demand and soil conditions on inelastic displacement ratios are discussed below.

6.3.1 Inelastic displacement demands

Bi-linear Models

Figure 6.5 presents mean inelastic displacement ratios for moderately stiff to stiff soils sites and soft soils sites together with their associated dispersion for bi-linear systems. The curves for C_R follow the general trends observed by other researchers [134, 136] where inelastic displacement ratios increase as the structural period tends to zero. The dispersion, quantified here by means of the coefficient of variation (COV), is observed to increase with the strength ratio R and decrease with increasing period. The values of COV presented in Figure 6.5b are also largely in accordance with findings from previous studies [134].

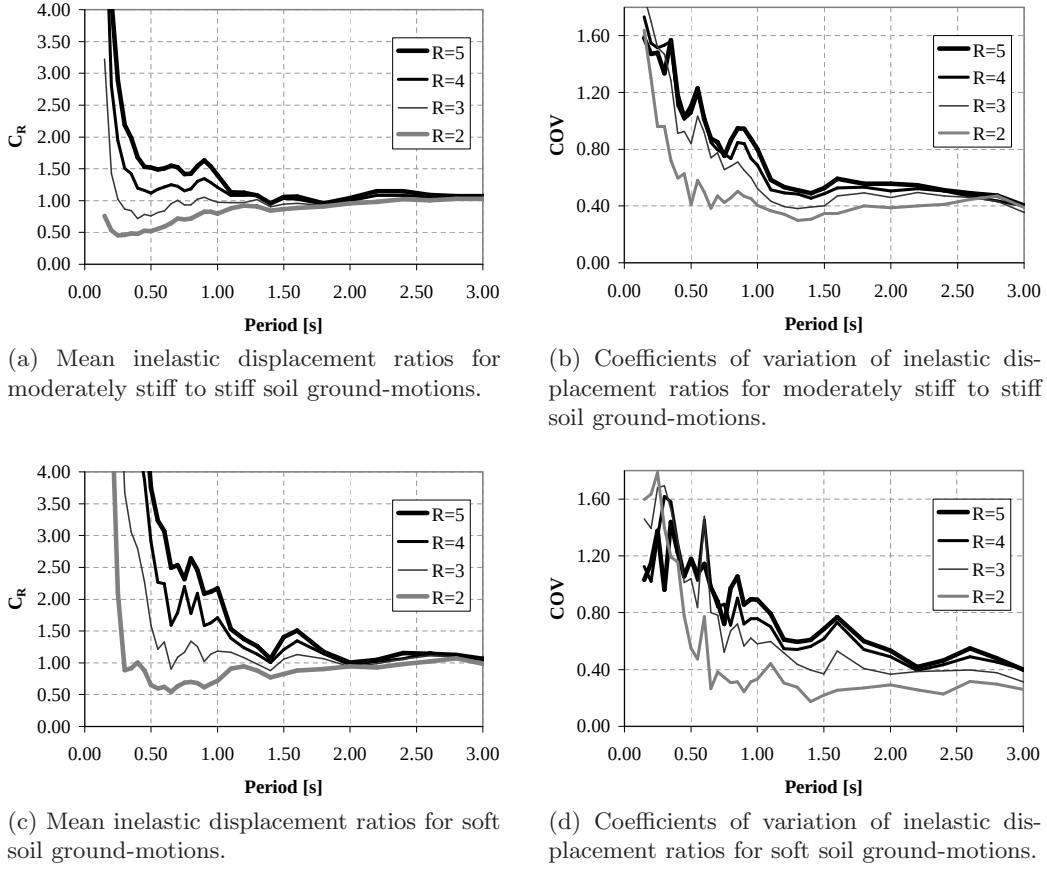


Figure 6.5: Inelastic displacement demands for bi-linear systems.

PR Models

Figure 6.6 depicts the results of mean inelastic peak displacement ratios of PR models normalized by mean inelastic displacement ratios of the corresponding bi-linear systems. It can be observed from Figure 6.6a that for periods longer than 1 s, the results of bi-linear and PR models with $P = 0.3$ are broadly coincident and that this occurs regardless of the strength ratio. However, for stiffer structures, with initial periods lower than 1 s, PR models exhibit an increasingly higher displacement that can reach values of twice the expected displacements in bi-linear systems. Similar results to those presented in Figure 6.6a were found for other levels of pinching.

Based on the negligible variability in displacement ratios as a function of R observed in Figure 6.6a, Figure 6.6b presents the normalized mean displacement ratios of PR over bi-linear models for different levels of pinching averaged over all R values considered (i.e. 2, 3, 4 and 5). It can be observed from Figure 6.6b that there is some

6. INELASTIC DEMANDS

degree of dependence of C_R ratios on the level of pinching, examined here by varying the factor P , particularly for relatively stiff systems. As expected, the displacement amplification of short-period PR models built on moderate to stiff soils with respect to bi-linear predictions tend to increase proportionally with P owing to the reduced energy dissipation in systems with higher pinching levels. Figure 6.6 also presents the results for soft soils sites (Figures 6.6c and 6.6d). Similar trends as those identified for moderate to stiff soils (Figures 6.6a and 6.6b) are observed, although higher inelastic demands and increased variability with respect to R are evident in the case of soft soils. Additionally, for structures with elastic periods longer than 2 s, peak displacements of PR models are on average about 10% lower than those observed for bi-linear structures, even for large pinching levels (e.g. $P = 0.15$ in Figure 6.6d). The levels of dispersion were also found to be similar to those observed for bi-linear systems and the COV followed the same trends (i.e. increasing with larger strength ratios R and decreasing with increasing structural flexibilities).

CB Models

Figure 6.7 presents the mean inelastic displacement ratios of CB systems normalized by the corresponding inelastic displacement ratios of bi-linear structures. It can be observed from Figure 6.7a that the mean displacements obtained with a CB fiber model exceed the bi-linear predictions most notably for structures with $T \leq 1$ s, while peak deformations on CB structures are lower than elastic-plastic estimations for periods longer than 2 s. This effect becomes more pronounced as the structural period shortens and as the level of inelastic behaviour decreases, particularly for moderate to stiff soils. The same increment in average peak displacements with decreasing period is evident for CB structures on soft soils (Figure 6.7c). Conversely, a greater variability of peak displacements with strength ratios can be observed for soft soils. On the other hand, the average ratio of peak displacement values for a constant $R = 2$ as a function of normalized slenderness is depicted in Figures 6.7b and 6.7d for moderate to stiff soils and soft soils, respectively. Slightly higher displacements are expected when more slender braces are used on stiffer soils sites for structures with elastic periods up to 1.5 s due to the improved energy dissipation characteristics of the hysteresis of more stocky braces, whereas the brace slenderness influence is negligible in the long period range. The same tendency towards greater peak deformations in more slender CB systems of shorter periods is manifested in the case of soft soils. On the other hand, for longer periods, bi-linear estimations seem to match those of CB systems with $\bar{\lambda} = 1.3$ but slightly underestimate peak deformations of more slender configurations (Figure 6.7d).

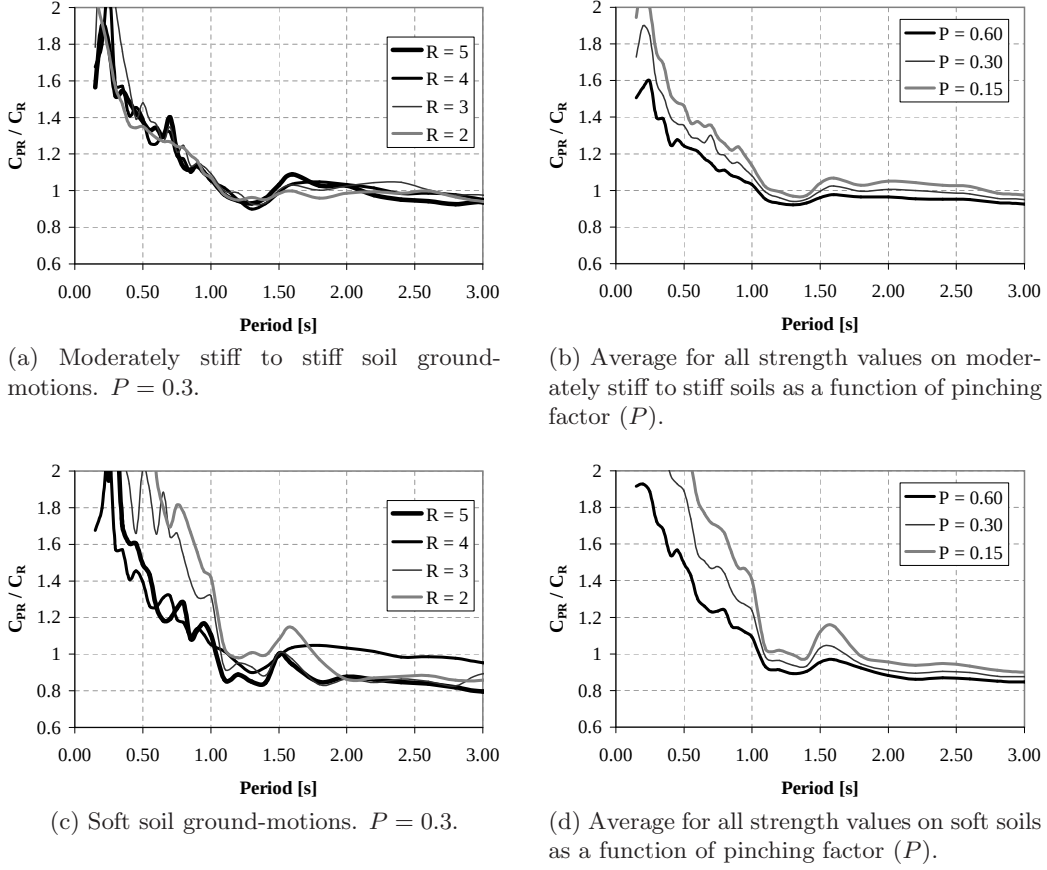


Figure 6.6: Mean inelastic displacement ratios of PR systems normalized by mean displacement ratios of bi-linear systems.

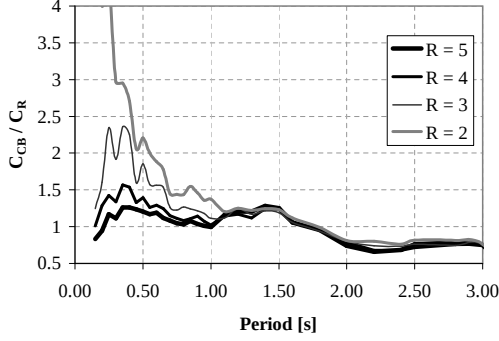
6.3.2 Effect of level of inelastic behaviour

The influence of the lateral strength ratio on C_R for PR and CB systems as compared against bi-linear systems is further studied with reference to Figures 6.8 and 6.9. Results are presented for mean peak inelastic displacements for structural periods of 0.4, 1 and 3 s for various PR pinching levels (Figure 6.8) and brace slenderness (Figure 6.9) for structures built on moderate to stiff soils.

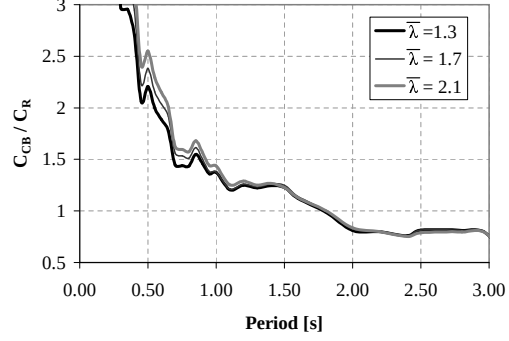
PR Models

In the case of PR models, it can be seen that at $T = 0.4$ s the average peak displacements increase almost linearly with lateral strength ratios R , an observation that is maintained for $T = 1.0$ s albeit at lower deformation levels. A significant and steady

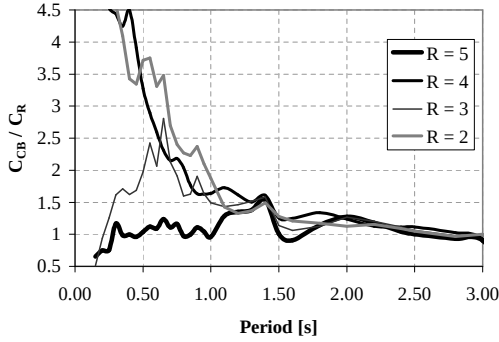
6. INELASTIC DEMANDS



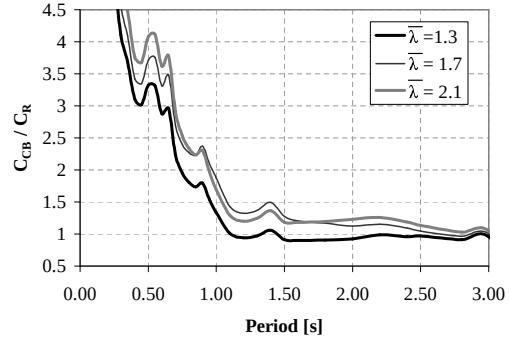
(a) Moderate to stiff soil ground-motions. (For $\bar{\lambda} = 1.7$).



(b) Displacement ratio as a function of normalized slenderness ($\bar{\lambda}$) for moderate to stiff soil ground-motions and $R = 2$.



(c) Soft soil ground-motions. (For $\bar{\lambda} = 1.7$).



(d) Displacement ratio as a function of normalized slenderness ($\bar{\lambda}$) for soft soil ground-motions and $R = 2$.

Figure 6.7: Mean inelastic displacement ratios for CB systems normalized by mean ratios of bi-linear systems.

dependence on the P factor is also evident from Figures 6.8a and 6.8b, whereas no significant influence of R or P is observed for longer periods (e.g. $T = 3.0$ s) where the equal displacement rule seems to be clearly applicable (Figure 6.8c).

CB Models

For CB systems (Figure 6.9) the influence of the level of lateral strength ratio on the displacement response is different as expected from the discussion on the dynamic behaviour in Section 6.2.4. Although C_R values also increase with increasing R for systems with $T = 0.4$ s, the dependence on brace slenderness does not follow a clear trend. Also, while mean peak displacements in CB structures can be in the order of two times the corresponding displacements for bi-linear systems when $R = 2$, such displacement amplification is reduced for higher strength ratios (Figure 6.9a). Additionally, the in-

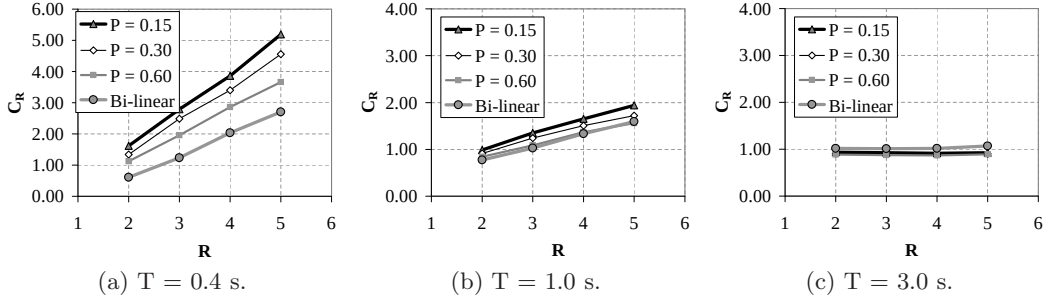


Figure 6.8: Effect of lateral strength ratio on the inelastic displacement ratios of PR systems.

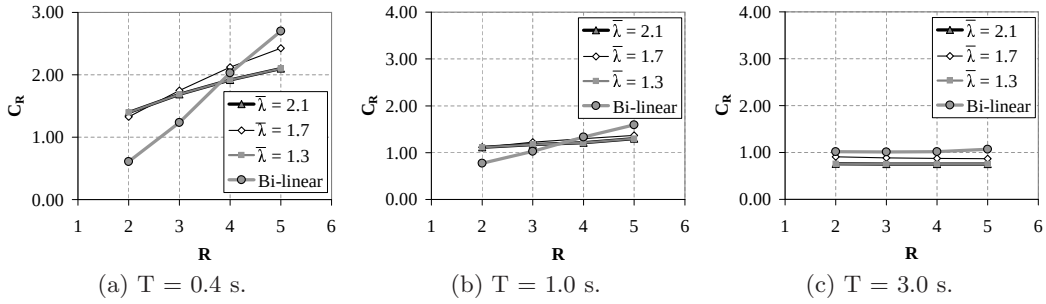


Figure 6.9: Effect of lateral strength ratio on the inelastic displacement ratios of CB systems.

fluence of the level of inelastic behaviour appears to be insignificant for CB structures with $T = 1.0$ and $T = 3.0$ s, and the equal displacement rule again seems to be largely applicable for CB structures in the long period range (Figures 6.9b and 6.9c).

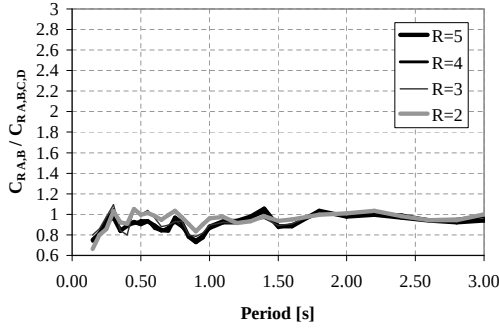
6.3.3 Effect of soil conditions

Besides the obvious differences in inelastic behaviour for stiff and soft soils sites, as highlighted in previous sections, it is also important to quantify the dissimilarities in inelastic displacements from ground-motions recorded in various moderate to stiff soil classes. Therefore, ratios of mean C_R on each soil class to mean C_R computed from all ground-motions on moderate to stiff soils (classes A,B,C and D) are computed and shown in Figure 6.10 for PR models and in Figure 6.11 for CB models. The ratios of C_R values on soft soils to C_R values on stiffer soils are also presented in Figures 6.10 and 6.11 for comparison purposes.

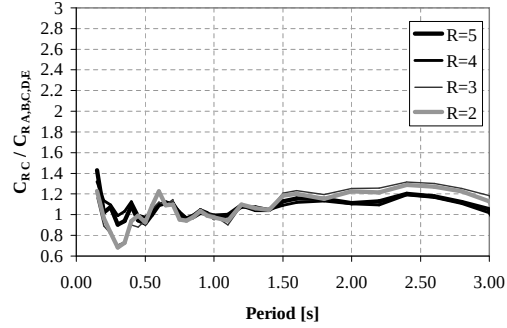
PR Models

In the case of PR structures built on rock sites (Classes A and B in Figure 6.10a), neglecting local site effects by using the average of all soil classes in moderate to stiff

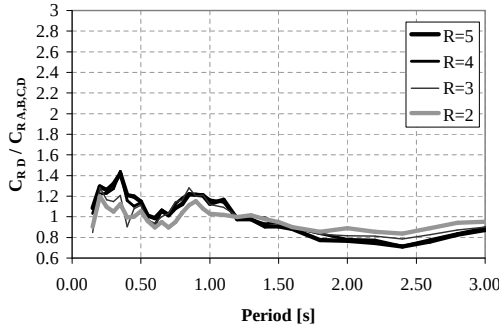
6. INELASTIC DEMANDS



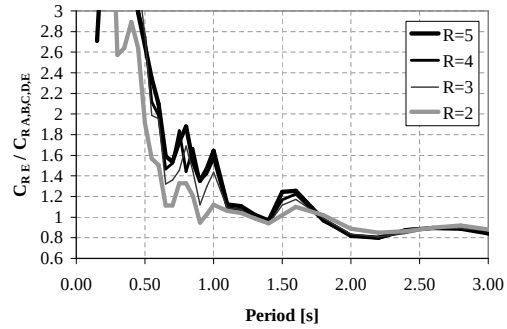
(a) Mean inelastic displacement ratios on Site Class A and B normalized by mean ratios from stiff soils.



(b) Mean inelastic displacement ratios on Site Class C normalized by mean ratios from stiff soils.



(c) Mean inelastic displacement ratios on Site Class D normalized by mean ratios from stiff soils.



(d) Mean inelastic displacement ratios on soft soil (Site Class E) normalized by mean ratios from stiff soils.

Figure 6.10: Effect of soil conditions on displacement demands of PR systems.

soils (Classes A, B, C and D) would lead to an overestimation of less than 20% in mean inelastic displacement predictions for periods shorter than 1.7 s whereas practically no differences occur for longer periods. For structures on Site Class C (Figure 6.10b) disregard of site effects leads to slight displacement under-predictions for periods longer than 1.2 s and no significant difference within the range of 0.7 to 1.2 s. Conversely, using mean displacement predictions from all stiff sites for structures built on Soil Class D will result in over-predictions in the order of 20% for periods longer than 1.2 s while comparable under-estimations can arise for stiffer structures, especially for periods shorter than 0.5 s (Figure 6.10c). Finally, when compared against the average of stiffer soils, the displacement amplification on soft soils sites is expected even for periods as long as 1.5 s and this soft site amplification increases notably with decreasing periods as shown in Figure 6.10d.

CB Models

As depicted in Figure 6.11, CB structures built on rock (soil Site Classes A and B) the peak displacements are 20% on average lower than the average peak inelastic displacement of the four stiff soils site classes (A, B, C and D) for periods smaller than 0.5 s. On the other hand, peak displacements would be higher than the mean inelastic displacements of the four sites for periods between 1 and 2 s (Figure 6.11a). Also, in the case of CB structures built on Site Class D, slight underestimations are apparent for very short periods while some degree of overestimation (less than 15%) is evident for longer periods ($1 \leq T \leq 2$ s in Figure 6.11c). Conversely, CB structures built on soil class C seem to be well predicted by the average of all four sites over the entire period range with minor underpredictions near 2.5 s. As in the case of PR structures, CB systems on soft soils sites experience much larger displacements than those expected for CB systems on stiffer soils (Figure 6.11d) with some dependence on R , highlighting the necessity for further dedicated studies and specific models for soft soils.

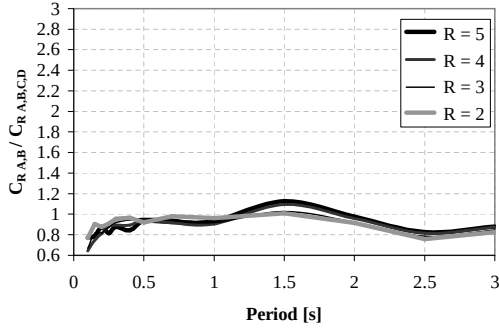
6.4 Influence of scalar frequency content parameters

In order to assess the influence of the frequency content of the ground-motion, as characterized by a single scalar parameter, peak displacements were determined for bi-linear, PR and CB models with a number of ratios of T normalized over five different period parameters. A sub-set of 30 records selected to reflect a wide range of frequency content characteristics was used as indicated in Table 6.1. In the following sub-sections, the definitions of the frequency content parameters employed are presented followed by a discussion on the comparative merits of their use on the basis of the generated statistical results.

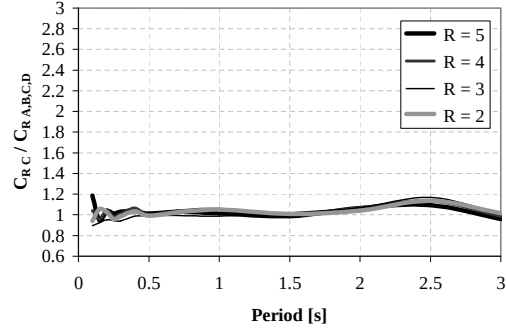
6.4.1 Frequency content parameters investigated

Five commonly employed frequency content indicators are evaluated in the present study: the average spectral period (T_{aver}), mean period (T_m), predominant period (T_g), characteristic period (T_c) and smoothed spectral predominant period (T_o).

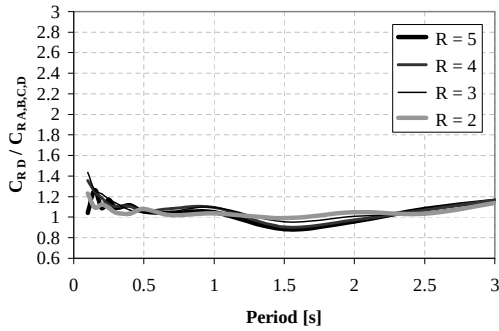
6. INELASTIC DEMANDS



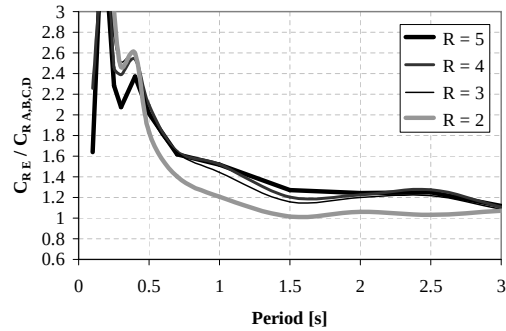
(a) Mean inelastic displacement ratios on Site Class A and B normalized by mean ratios from stiff soils.



(b) Mean inelastic displacement ratios on Site Class C normalized by mean ratios from stiff soils.



(c) Mean inelastic displacement ratios on Site class D normalized by mean ratios from stiff soils.



(d) Mean inelastic displacement ratios on soft soil (Site Class E) normalized by mean ratios from stiff soils.

Figure 6.11: Effect of soil conditions on displacement demands of CB systems.

The average spectral period T_{aver} is calculated as:

$$T_{aver} = \frac{\sum_{i=1}^n T_i (S_a(T_i)/PGA)^2}{\sum_{i=1}^n (S_a(T_i)/PGA)^2} \quad (6.6)$$

where T_i are equally spaced periods in the acceleration spectra with $0.05 \leq T_i \leq 4$ s, $\Delta T_i = 0.01$ s, $S_a(T_i)$ is the spectral acceleration at T_i and PGA is the peak ground acceleration.

The mean period T_m can be determined as:

$$T_m = \frac{\sum_{i=1}^n 1/f_i C_i^2}{\sum_{i=1}^n C_i^2} \quad (6.7)$$

where C_i is the Fourier amplitude coefficient at frequency f_i with $0.25Hz \leq f_i \leq 20Hz$ and $\Delta f_i = 0.01Hz$.

The predominant period, T_g , is defined as the period at which the input energy is maximum throughout the period range and can be computed as the period at which the maximum ordinate of a 5% damped relative velocity spectrum occurs [113].

The characteristic period T_c is the period defining the transition between the acceleration sensitive and the velocity sensitive regions of the response spectra.

Finally, the smoothed predominant spectral period, T_o , is defined as:

$$T_o = \frac{\sum_{i=1}^n T_i \ln(S_a(T_i)/PGA)}{\sum_{i=1}^n \ln(S_a(T_i)/PGA)} \quad (6.8)$$

where T_i are periods in the acceleration response spectrum equally spaced on a log axis with $S_a/PGA \geq 1.2$ and $\Delta \log T_i = 0.01$.

6.4.2 Results of statistical study

Previous studies [22, 135, 136] have suggested the use of frequency content parameters in the context of reducing the variability of response estimations or differentiating between earthquake types by plotting the results in terms of normalized periods. On the other hand, Kumar et al. [86] identified the influence of T_m after fitting two exponential trend-lines for $T/T_m < 1$ and $T/T_m > 1$ to clouds of displacement ratios and noticing a steady increment in displacements for periods of $T/T_m < 1$. The former approach is used in this study in order to identify the influence of a given ground-motion characteristic period and to assess the benefits of using one scalar frequency content parameter over the other. This is carried out in recognition that the increments in displacement ratios for shorter normalized periods primarily reflect a general feature of short-period SDOF behaviour rather than a specific effect of a certain ground-motion frequency parameter. Therefore, in this study the effects of different frequency content indicators are compared in light of the relative improvements in displacement predictability. Furthermore, the Coefficient of Variation (COV) is used herein as a normalized measure of dispersion and as the basis for comparative purposes. To this end, peak displacement ratios were calculated over a number of period ratios normalized over a range of characteristic period indicators. The corresponding statistical results are presented and discussed below for bi-linear, PR and CB systems.

6. INELASTIC DEMANDS

Bi-linear Models

Figure 6.12 presents the COV of peak inelastic displacements for bi-linear structures over a range of initial elastic periods ($T/1$) and normalized periods (T/T_m , T/T_c , T/T_{aver} , T/T_g and T/T_o). It can be observed from Figure 6.12 that by considering the predominant period of the ground-motion (T_g), the variability in the response is reduced rapidly (up to a COV of 0.3 at $T/T_g = 1$) and is kept at lower levels over a wider range of structural periods. Other scalar frequency content parameters provide mixed merits in displacement estimations, with T_m and T_c reducing the scatter for short period structures but increasing it for longer period systems. It can also be noted from Figure 6.12 that the smoothed predominant spectral period (T_o) appears to be the least able to characterize frequency content effects on structural deformations, thus producing the highest variability in the estimations. In turn, this reduction in the dispersion offers a clear advantage when differentiating among record bins on the basis of their frequency content. To this end, it can be shown that earthquake suites defined in terms of T_g follow a consistent pattern which is not achievable through the use of any of the other ground-motion period indicators considered.

PR Models

Figure 6.13 presents the variation of the COV of C_R ratios for PR systems normalized by different frequency content period indicators and for various levels of pinching. It is evident from Figure 6.13 that the inclusion of T_g in the estimation of peak deformations reduces considerably the associated dispersion. As noted previously for bi-linear systems, in the case of PR structures the COV rapidly decreases to values below 0.3 at $T/T_g = 1$ and is remarkably consistent and low for longer period structures regardless of the level of pinching. Normalization against other frequency content parameters seem to reduce the variability in displacement estimations to a lesser degree and only for limited period ranges. Similarly, this means that T_g is better suited for distinguishing among earthquake accelerograms when estimating peak deformations in PR structures.

CB Models

Figure 6.14 presents the COV of the mean peak inelastic displacement ratios as a function of normalized periods for CB structures with braces of various normalized slenderness values. It can be observed from Figure 6.14 that overall, in the case of CB structures, the period normalization by T_g continues to give lower dispersion values. This is particularly the case for $T/T_g > 1$ in CB structures with braces of $\bar{\lambda} = 1.3$ or 1.7. Nevertheless, the reduction in COV is not as clearly beneficial over all periods as

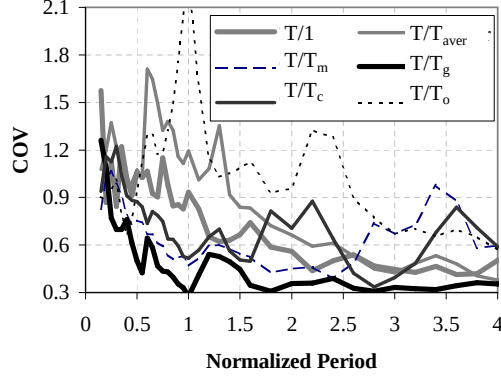


Figure 6.12: Coefficient of variation of inelastic displacement ratios of bi-linear systems for different normalized periods. $R = 5$.

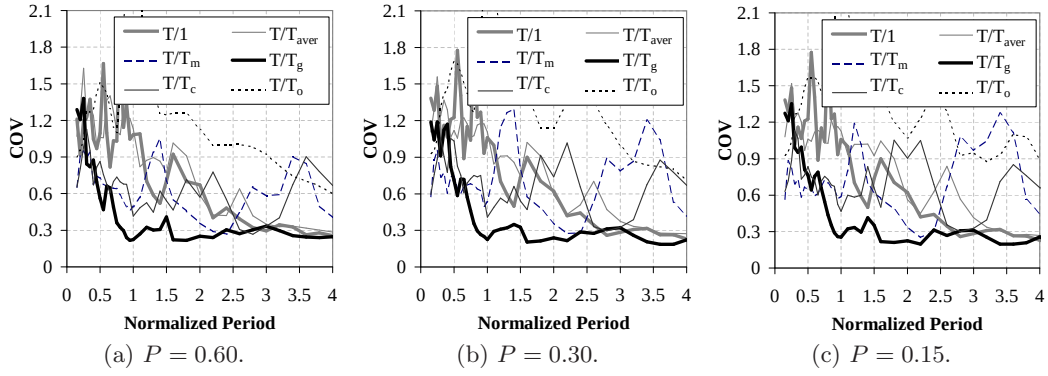


Figure 6.13: Coefficient of variation of inelastic displacement ratios of PR systems for different normalized periods, $R = 5$ and different Pinching Factors P .

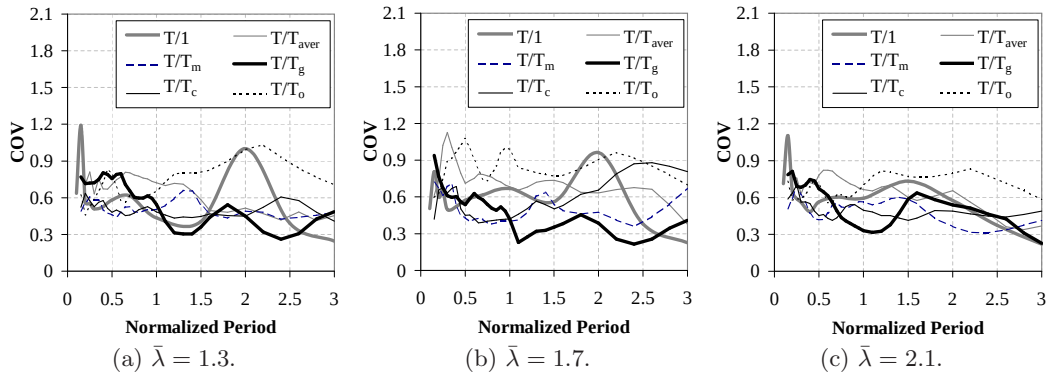


Figure 6.14: Coefficient of variation of inelastic displacement ratios of CB systems for different normalized periods, $R = 5$ and different normalized slenderness $\bar{\lambda}$.

6. INELASTIC DEMANDS

in the case of bi-linear or PR models. In particular, using T_m or T_c seem to be more effective for shorter periods ($T/T_m = T/T_c < 1$) irrespective of the slenderness of the bracing system, and for normalized periods between 1.5 and 2.5 for CB models with $\bar{\lambda} = 1.7$.

6.5 Proposed equivalent linear models

As discussed before, reliable estimation of peak displacements is a primary consideration in the seismic assessment of steel structures. To this end, the previous section has shown that the inclusion of a frequency content indicator leads to an improved characterization of central tendencies in inelastic displacement responses. Additionally, it has been argued [97] that equivalent linear systems based on secant periods offer several advantages over optimally defined equivalent systems, particularly in terms of enabling a direct and meaningful comparison of capacity and demand. In light of the above discussion as well as results presented in previous sections, this section proposes and examines expressions for equivalent period (T_{eq}) and equivalent damping (ξ_{eq}) parameters for PR and CB structures. The proposed equivalent linear models are based on the secant stiffness and aim to complement those suggested by Lin and Lin [97] for bi-linear systems.

6.5.1 Non-iterative equivalent linearization models

The equivalent period T_{eq} is defined hereafter as a function of the strength ratio R , the predominant period of the ground-motion T_g and the initial structural period T as:

$$\begin{aligned} T_{eq}/T &= \sqrt{\frac{(R-1)T_g + T}{1.6T}} & \text{if } T < T_g \\ T_{eq}/T &= \sqrt{\frac{RT_g}{2.8T_g - 1.2T}} & \text{if } T_g \leq T \leq 1.5T_g \\ T_{eq}/T &= \sqrt{R} & \text{if } T > 1.5T_g \end{aligned} \quad (6.9)$$

Equation 6.9 was obtained by employing strength-ductility relationships for target ductility scenarios over normalized periods (T/T_g) as described in Appendix B. In those simplified relationships, the strength ratio R was observed to increase from a value of 1 at $T/T_g \approx 0$ to 1.6 times the ductility (μ) at $T/T_g = 1$ before keeping a constant value of $R = \mu$ for period ratios greater than $T/T_g = 1.5$.

On the basis of the extensive database generated in the course of this study and considering an equivalent period defined by Equation 6.9, it is possible to determine an equivalent damping ratio (ξ_{eq}) for each period-displacement pair that would minimize the error in peak deformation estimations. Multivariate regression analyses can then be performed in order to develop prediction expressions for mean equivalent viscous damping values [97]. The equivalent damping expressions obtained from such regression analyses are given below for PR and CB models. Importantly, the high variability and peculiar features of structural response observed for soft soils in the previous sections, calls for dedicated models which can be subject of future research. Accordingly, only data for moderate to stiff soils sites (70 records) were used in the computation of the regression expressions for equivalent damping presented herein.

Equivalent Damping for PR Models

For PR structures with $T \leq 1.0$ s, the equivalent damping can be estimated as:

$$\begin{aligned}\xi_{eq} &= 0.05 + ae^{bT} \\ a &= 1.425R^{-0.25} \\ b &= (0.28 + P/1.5)\ln(R) - 2.7\end{aligned}\tag{6.10}$$

Whereas for PR structures with $T \geq 2.0$ s:

$$\begin{aligned}\xi_{eq} &= 0.05 + a\ln R + b \\ a &= 0.0299P + 0.14 \\ b &= 0.02826\end{aligned}\tag{6.11}$$

Values for PR structures with $1 < T < 2$ s can be found by linear interpolation. Additional expressions for equivalent PR structures with different levels of post-elastic stiffness can be found in Appendix B.

Equivalent Damping for CB Models

Similarly, expressions for the estimation of equivalent damping in CB systems with $T \leq 1.5$ s can be established as follows:

$$\begin{aligned}\xi_{eq} &= 0.05 + a\ln(T) + b \\ a &= -0.0523R - 0.0539 \geq -0.2665 \\ b &= 0.9568R^{-0.79}\end{aligned}\tag{6.12}$$

6. INELASTIC DEMANDS

For the estimation of equivalent damping for CB structures with elastic period greater than $T > 1.5$ s, the value of $\xi_{eq}(T = 1.5)$ can be assumed to be applicable. In light of the minor dependence of CB peak displacements on the normalized brace slenderness ($\bar{\lambda}$) and the high variability in the response observed (e.g. Figure 6.14), Equation 6.12 does not incorporate a slenderness regression function.

6.5.2 Verification and application

The accuracy of Equations 6.9 to 6.12 to predict peak displacement demands can be evaluated by the median ratio (E) of approximate (δ_{ap}) to exact (δ_{ex}) peak inelastic displacements defined as:

$$E(T, R, \alpha, T_g) = med \left(\frac{\delta_{ap}(T_{eq}, \xi_{eq})}{\delta_{ex}(T, \xi, R, \alpha, T_g)} \right) \quad (6.13)$$

A value of E close to 1 indicates that the proposed equivalent linear model accurately describes the displacement response, whereas values of $E < 1$ or $E > 1$ represent under or overestimations of peak displacement, respectively. With reference to the previously-generated dataset, for a given structural period T , strength ratio R and ground-motion predominant period T_g , the equivalent period (from Equation 6.9) and damping (from Equations 6.10 to 6.12) can be calculated. Subsequently, the approximate peak inelastic displacement can be estimated from the response spectra of the equivalent linear system. Based on such approximate peak inelastic displacement and the exact values of peak deformations, values of E were computed for a range of systems, strength reduction factors and hysteretic models for the 70 ground-motion records on moderately stiff to stiff soils employed in this study.

Figure 6.15 shows the median ratio of approximate to exact peak inelastic displacements for PR systems with different pinching levels and for different strength demands while Figure 6.16 presents the corresponding Coefficients of Variation (COV). Similarly, Figure 6.17 presents median ratios of approximate to exact peak displacement for CB structures of varying slenderness and strength factors while the associated COV are depicted in Figure 6.18. In general, Equations 6.9 to 6.12 were found to provide reasonable estimations of peak displacements with errors within a $\pm 20\%$ for a wide range of periods. It can be seen from Figure 6.15 that the proposed PR equivalent linear models provide reliable estimations for structures with periods greater than 0.20 s

whereas notable under-predictions would be expected for very stiff PR structures. These reliable estimates could be observed irrespectively of the strength demand ratio or the level of pinching in the system (Figure 6.15). In the same way, peak displacement estimations within $\pm 20\%$ would be expected for CB systems with periods greater than 0.20 s within the whole range of brace slenderness studied. Nonetheless, notable over-estimations would be expected for very stiff CB structures, characterized by periods lower than 0.2 s (Figure 6.17). Besides, by using the equivalent models of Equations 6.9 to 6.12, there is a significantly low level of scatter in the peak displacement estimations for PR structures over the whole period range under study (e.g. by comparing the COV of $T/1$ in Figure 6.13 with the corresponding curves in Figure 6.16). However, a less significant improvement in the dispersion of peak deformation estimations is noted for CB systems for intermediate and long periods (e.g. by comparing the COV of $T/1$ in Figure 6.14 with the corresponding curves in Figure 6.18) while the scatter in the short period range is significant.

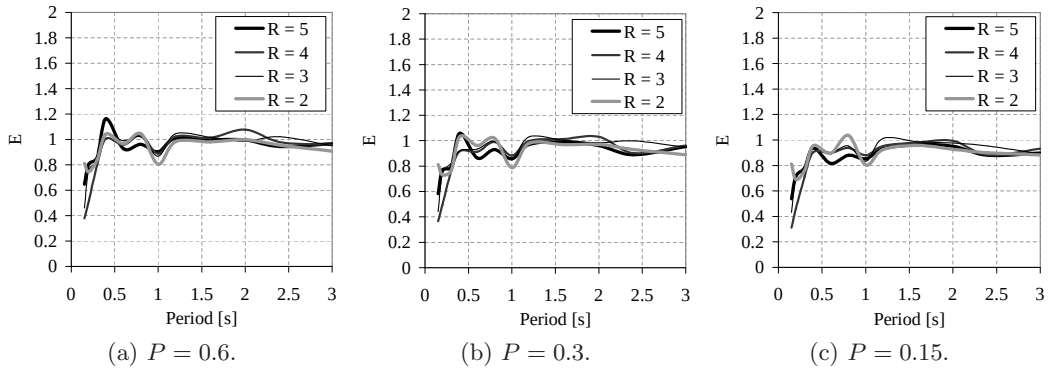


Figure 6.15: Median ratio of approximate to exact (δ_{ap}/δ_{ex}) peak inelastic displacements for PR models.

6. INELASTIC DEMANDS

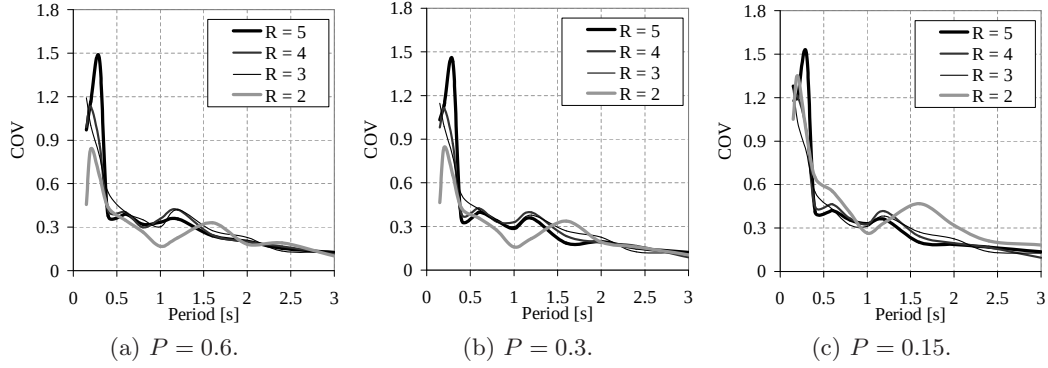


Figure 6.16: COV of approximate to exact (δ_{ap}/δ_{ex}) peak inelastic displacements for PR models.

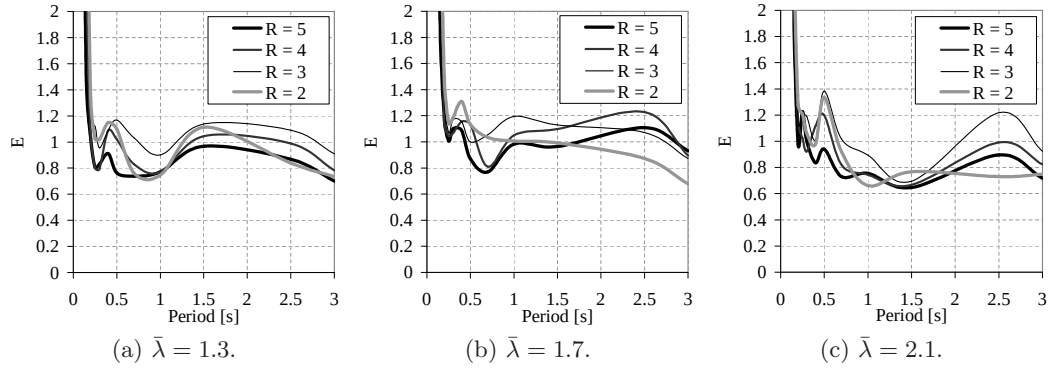


Figure 6.17: Median ratio of approximate to exact peak inelastic displacements for CB models.

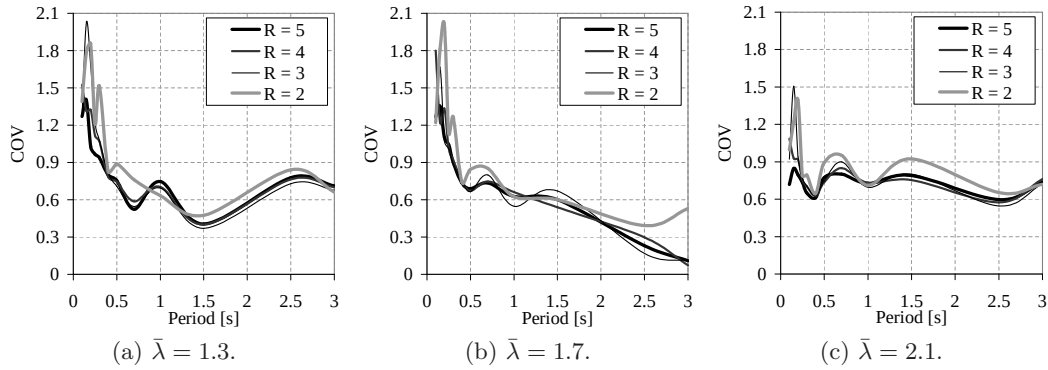


Figure 6.18: COV of approximate to exact (δ_{ap}/δ_{ex}) peak inelastic displacements for CB models.

In order to illustrate the use of the equivalent linearization expressions, the assessment of a 6-storey CB frame building is presented as an example. The building was designed and modelled by Málaga-Chuquitaype et al. [102] to a seismic demand consistent with a 2% probability of exceedance in 50 years for Los Angeles (USA) area. The building

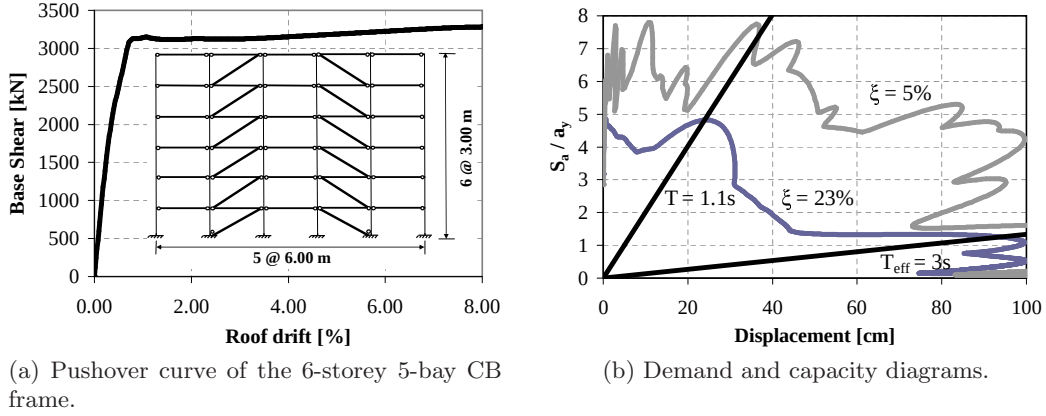


Figure 6.19: Application of the equivalent linearization on the displacement estimation of a structure.

has an initial period of 1.1 s and a base shear resistance of approximately 1000 kN. Figure 6.19a shows a schematic view of the building under consideration together with the base shear versus roof drift curve derived from a monotonic incremental non-linear analysis using an inverted triangular force distribution along the height of the building. Further particular of the non-linear response history analysis and modelling approach can be found in Section 7.5.2. The building is assumed to behave as a SDOF with a constant shape displacement profile which was found to be representative for this particular structure [102]. The response to El Centro record with $T_g = 1.95$ s is considered as an example. The record was scaled to have a *PGA* over acceleration at yield equal to 3, which corresponds to a strength ratio $R = 7.26$ [102].

According to Equation 6.9, the T_{eq}/T ratio can be estimated as 2.48 (i.e. $T_{eq} = 3$ s). By means of Equation 6.12, coefficients a and b are evaluated as -0.27 and 0.2, respectively, and the resulting equivalent viscous damping is $\xi_{eq} = 23\%$. Figure 6.19b presents the demand diagram for the structure and record under consideration. It can be observed that the equivalent linearized model predicts a roof displacement of approximately 95 cm which should be compared with the 102 cm obtained from refined non-linear response history analysis [102]. The good predictions obtained here serve only to exemplify the applicability of the proposed models in estimating peak displacement demands. However, the validity of the proposed linearized model clearly needs to be examined further in future studies covering more buildings and design hypothesis within a comprehensive probabilistic context.

6.6 Concluding remarks

This chapter has examined the inelastic displacement response of steel framed structures of known levels of strength when subjected to a relatively large number of ground-motions. SDOF systems with three hysteretic responses were analysed: (i) bi-linear systems typical of moment resisting structures, (ii) Modified Richard-Abbott models representative of partially restrained (PR) frames, and (iii) fiber-based models simulating concentrically-braced (CB) framed structures. The influence of model characteristics, level of inelastic behaviour and soil conditions on peak displacement ratios has been discussed.

The study revealed key differences in the inelastic deformation demands between bi-linear, PR and CB models, particularly in the relatively short period spectral region. The ratio between the overall yield strength and the strength during pinching intervals was found to be the main factor governing the inelastic deformations of PR models when compared with bi-linear model predictions. PR models can exhibit higher displacement demands that may reach more than double the peak displacements estimated through bi-linear models for relatively stiff structures. Despite their similar cyclic force-displacement relationships, the seismic response of CB and PR models was observed to follow different tendencies, with the rate of displacement amplifications decreasing as the strength ratio increases for CB models when compared against bi-linear predictions. It was found that the effects of local site conditions on displacement ratios are relatively small for moderately stiff to stiff soils, whereas significant displacement amplifications occur in the case of soft soils relative to stiffer sites.

A study on the influence of a number of scalar parameters that characterize the frequency content of the ground-motion on the estimated peak displacement ratios was performed. The predominant period of the ground-motion (T_g) was found to be clearly more effective in reducing the variability and thus enhancing the accuracy of peak inelastic displacement estimations in bi-linear and PR structures when compared with other scalar frequency indicators such as average spectral period T_{aver} , mean period T_m , characteristic period T_c and smoothed spectral predominant period T_o . As for CB structures, the inclusion of T_g reduces the dispersion in peak displacements for $T/T_g > 1$, especially in CB structures with more stocky braces (i.e. $\bar{\lambda} = 1.3$ or 1.7). Nevertheless, the reduction in COV is not as consistent over all periods as in the case of bi-linear or PR models.

Finally, non-iterative equivalent linearization expressions for the estimation of peak deformations on SDOF systems with known strength were proposed. In light of the findings of this study, the equivalent linear system proposed was defined as a function of the strength ratio R , the predominant period of the ground-motion T_g and the structural initial elastic period T . The developed equations were validated by examining the range of estimated over exact peak displacements. These prediction equations appear to be reliable for the assessment of existing steel structures with $T > 0.20$ s. However, the validity of the proposed linearized model needs to be examined further in future studies covering more buildings dimensioned according to different design hypothesis and within a comprehensive probabilistic context.

Chapter 7

Rigid-Plastic Models for Seismic Design and Assessment

7.1 Introduction

The previous chapter highlighted the high levels of variability associated with peak seismic inelastic response in the short period range, in particular for CB structures. Large dispersions are also manifested in the inability of the equivalent linear methods proposed in Section 6.5 to estimate peak displacements for very stiff structures, as described in the previous chapter. Several alternative methods are available for a more precise structural analysis of steel frames subjected to earthquake actions. Among them, non-linear dynamic analysis (also referred to as time-history) is widely recognized as the most reliable technique due to its ability to reflect the time dependent nature of the seismic behaviour. Nevertheless, its application in practical assessment and design is hampered by the high level of expertise it requires as well as the significant time and costs associated with it. This has motivated the studies presented in this chapter that aim to demonstrate the applicability of response history analysis based on simple rigid-plastic idealizations for the seismic assessment and design of steel framed structures.

Modern earthquake resistant design philosophies aim to achieve reliable structural performance whilst maintaining an economic design by ensuring energy dissipation capabilities under strong ground-motions. Such energy dissipation is usually attained through high levels of plastic deformation in certain specially designed elements whilst elastic behaviour is ensured in other structural members. If elastic deformations be-

7. RIGID-PLASTIC MODELS

come relatively insignificant (e.g. at large deformation levels), rigid-plastic models can be adopted to assess the structural response. The relative simplicity of rigid-plastic models, coupled with the rationality of the response history approach, can introduce a great deal of ease and versatility to the analysis. The main assumption adopted in a rigid-plastic model is the disregard for any elastic deformation. This in turn means that the stiffness alternates between infinite and zero levels which may complicate its implementation within conventional stiffness-based computer methods. Nevertheless, the avoidance of elastic stiffness considerations introduces considerable simplification as illustrated in subsequent sections of this chapter. In general, rigid-plastic models also assume that all sources of energy dissipation are due to the plastic behaviour and hence non-hysteretic damping is ignored.

As discussed in Section 2.3 of the literature review, despite its common use in studying the post-elastic behaviour under static loads, rigid-plastic models have not been widely employed in dynamic problems mainly due to the lack of suitable computational procedures. Another perceived problem is related to the inability of the method to deal with resonance effects, although it should be noted that resonant response decays rapidly as soon as non-linear deformations start to occur. One of the first studies dealing with rigid-plastic approximations in the earthquake engineering field was the work of Newmark [118] who devised a method to estimate earthquake induced displacements on slope stability problems where relative displacements start to accumulate whenever the earthquake induced acceleration exceeds certain resistance thresholds. Augusti [6] proposed general formulae for the solution of the dynamic response of framed structures based on rigid-plastic oscillators subjected to simple pulse and periodic loading. Strain hardening and strength degradation effects were partially considered in his study. Paglietti and Porcu [124], by means of the same classic rigid-plastic model, calculated the maximum displacement response of several SDOF systems to seismic loading. Their study also introduced the idea of the so called *rigid-plastic spectrum* defined as the relationship between a structural response parameter (such as peak structural displacement) and the plastic capacity of the rigid-plastic model. More recently, and based on such relationship, Domingues-Costa et al. [37] proposed an earthquake resistant design procedure for reinforced-concrete moment resisting frames modifying the classical rigid-plastic model to take into account pinching effects typical of reinforced-concrete structures. This study highlighted the relative merits of rigid-plastic approaches, which led to renewed interest in exploiting its advantages, particularly in terms of simplified seismic design and assessment procedures.

The principal advantage of using rigid-plastic models is related to their efficiency in terms of modelling effort and computational time. As illustrated in this chapter, the computational procedures are relatively straightforward in comparison with advanced response history analysis and necessitate a much lower degree of specialized knowledge and input. Most importantly, the definition of an appropriate rigid-plastic model requires the specification of a single key parameter: the plastic capacity represented in terms of the strength at yield. This enables simple relationships to be established between the strength provided and a given structural performance parameter, which can be used for design purposes.

The present chapter extends the advantages associated with rigid-plastic models to different typologies of steel buildings (including frames that use tubular members as columns and/or bracings) under earthquake loading. To this end, new hysteretic relationships (as well as their corresponding integration algorithms) are proposed. It is shown that such rigid-plastic models are able to predict global demand response histories with reasonable accuracy. Finally, relationships between peak displacement and the plastic capacity of rigid-plastic oscillators are used to obtain the required structural strength for a given target scenario.

7.2 Moment resisting (MR) frames

7.2.1 Rigid-plastic idealization for SDOF systems

Figure 7.1a presents a classic rigid-plastic oscillator of mass m , height h and plastic moment capacity at the columns M_0 , subjected to a ground acceleration $a_g(t)$. For such a rigid-plastic oscillator, the yield capacity of the structural system defined by $F_{yp} = -F_{yn}$ can be related to the limit moment at the plastic hinges by:

$$F_{yp} = \frac{4M_0}{h} \quad (7.1)$$

similar conventions can be introduced for other, more complicated, rigid-plastic structures.

Figure 7.1b presents the rigid-plastic response of the above described MR oscillator as a function of lateral force and displacement. In this figure, two main behavioural states

7. RIGID-PLASTIC MODELS

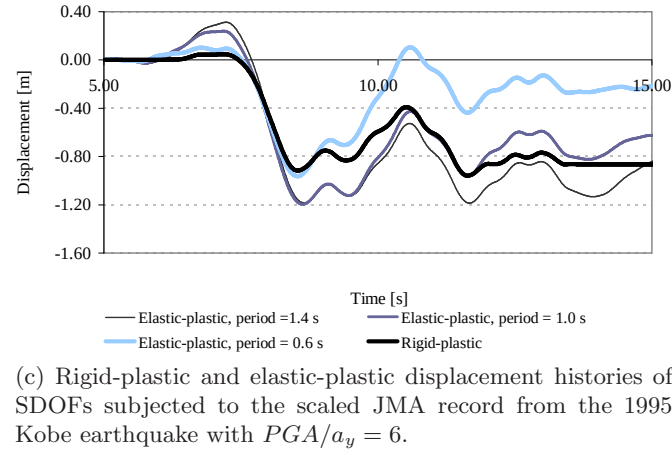
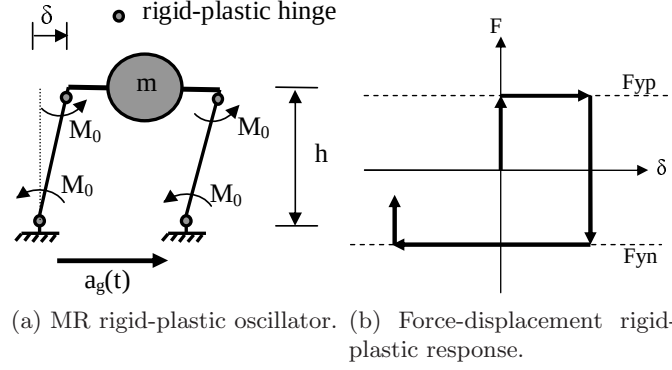


Figure 7.1: Rigid-plastic oscillator subjected to ground-motion excitation.

are evident: (i) a *rigid state* where the yield strength of the member is not reached and hence no displacement takes place and, (ii) a *plastic state* where deformations occur and the strength demand is equal to the yield strength.

Expression of dynamic equilibrium by means of D'Alembert's principle over the rigid-plastic oscillator of Figure 7.1a yields the following expression for the relative acceleration $a_r(t)$ as a function of time t :

$$a_r(t) = \frac{F(t)}{m} - a_g(t) \quad (7.2)$$

where $F(t)$ represents the internal forces opposing the motion. With the previous assumptions and definitions, it can be immediately recognised that Equation 7.2 can be replaced by the following set of relationships, which are more practical for computer-

based numerical integration:

$$\begin{aligned}
 a_r(t) &= \frac{F_{yp}}{m} - a_g(t) & \text{if } v_r(t) > 0 \\
 a_r(t) &= \frac{F_{yn}}{m} - a_g(t) & \text{if } v_r(t) < 0 \\
 a_r(t) &= 0 & \text{if } F(t) \in]F_{yn}, F_{yp}[
 \end{aligned} \tag{7.3}$$

where $v_r(t)$ is the relative velocity of the mass m , and hence a change of state occurs whenever $v_r(t)$ changes from positive or negative to zero. For this purpose, the numerical integration scheme presented in Figure 7.2 was implemented in MATLAB[108] and used in the present study. In this figure $d_r(t)$ is the relative displacement of the mass m , τ is the specific time at which the oscillator changes its behaviour from rigid to plastic or vice versa, Δt is the integration time-step and F_y takes the value of F_{yp} if $v_r(t) > 0$ or F_{yn} if $v_r(t) < 0$. Finally, the value of acceleration is assumed constant over a Δt period of time.

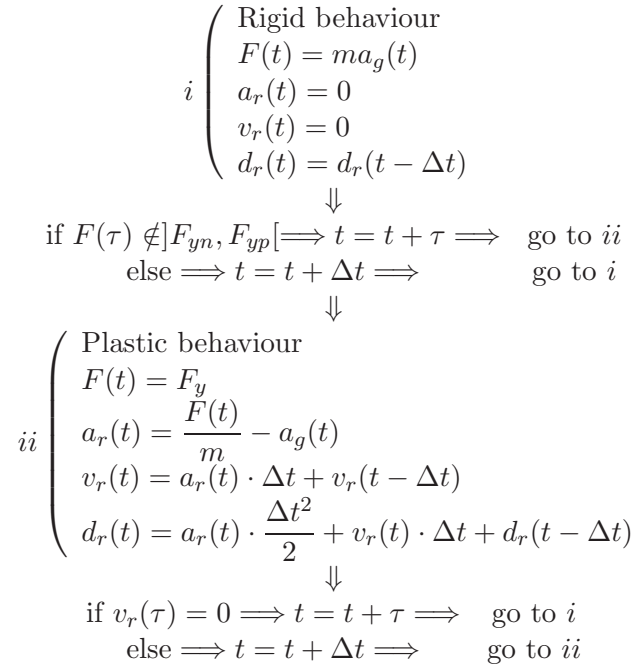


Figure 7.2: Integration algorithm for rigid-plastic MR frames.

Figure 7.1c presents a comparison between the seismic response in terms of displace-

7. RIGID-PLASTIC MODELS

ment history of a rigid-plastic oscillator and a number of elasto-plastic SDOF models with elastic periods varying from 0.6 to 1.4 seconds, to the JMA record from the 1995 Kobe earthquake. This record imposes a peak demand of 6 times the yield strength of the structure or, in other words, a ratio between peak ground acceleration (PGA) and ground acceleration at yield a_y of 6. It should be noted, as pointed out by Paglietti and Porcu [124] that the single most important parameter governing the accuracy of rigid-plastic prediction is the overall contribution of elastic deformation. In other words, as expected for a given structure, the greater the ductility demand the more accurate the rigid-plastic prediction would be, in comparison with a conventional elastic-plastic model that does not account for strength degradation. In this case, the ductility demand is defined as the ratio between the peak displacement and the displacement at yield. This emphasises the suitability of the ratio between the PGA and the acceleration at structural yield for characterizing the accuracy of the rigid-plastic prediction approach. In turn, for the same ductility demand, the rigid-plastic model would give better estimates of peak displacement for stiffer structures.

7.2.2 Response prediction of MDOF systems

In this section, the rigid-plastic model is applied to multi-storey MR frames by considering rigid-plastic behaviour at plastic hinges and rigid behaviour elsewhere. It is assumed that the displacement vector $s(x, t)$ at any location x in the structure and at time t can be represented by:

$$s(x, t) = \delta(t)\Phi(x) \quad (7.4)$$

where $\delta(t)$ represents the variation of the displacement at the effective height of the structure as a function of time and $\Phi(x)$ is the vector representing the shape of the plastic mechanism.

The response of a three-storey building designed by Bruneau et al. [19] and presented in Figure 7.3a is considered. Concentrated dead and superimposed loads of 250 kN and 100 kN, respectively, are applied to the columns at each floor level and distributed loads of 15 kN/m and 10 kN/m, corresponding to dead and live loads respectively, are applied to the beams along their length. The building was designed for drift-controlled behaviour and reduced beam section connections (RBS) were utilised.

7. RIGID-PLASTIC MODELS

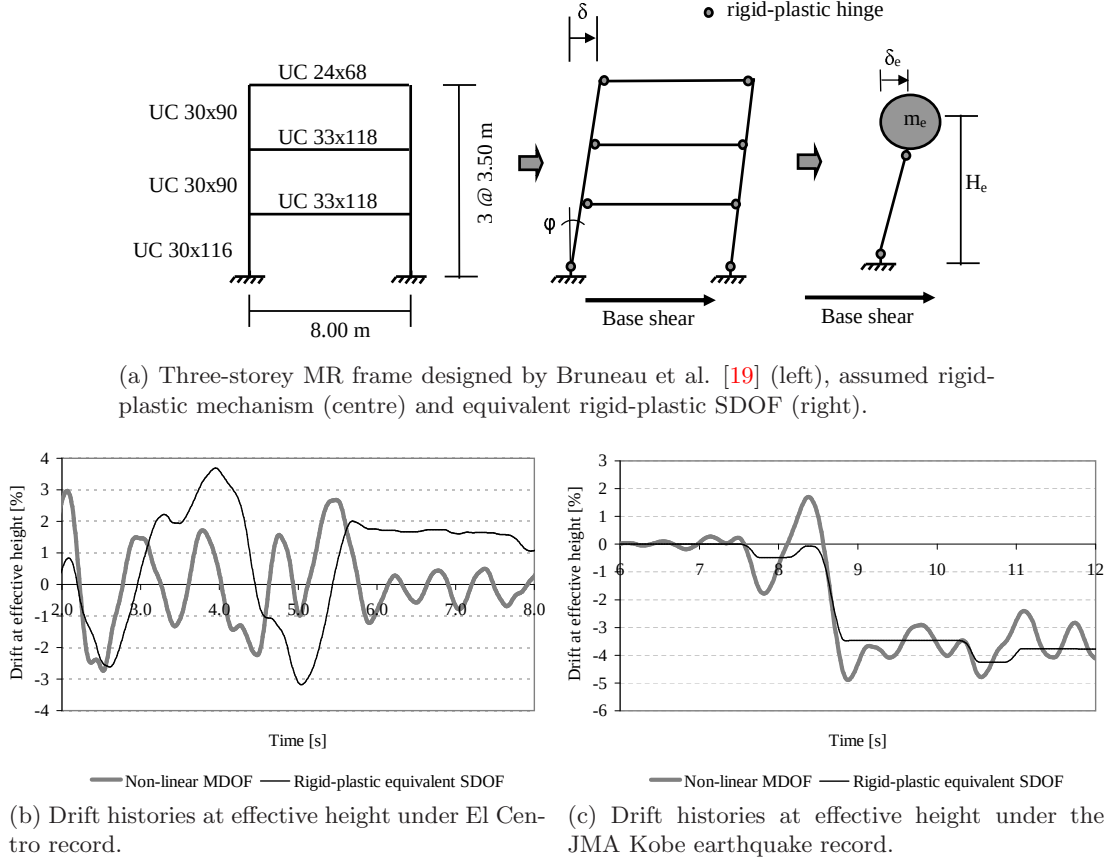


Figure 7.3: Rigid-plastic analysis of a MDOF MR frame.

Several non-linear elasto-plastic analyses were performed using the non-linear finite element program *Adaptic* [75] with due consideration of both material and geometric non-linearities. Cubic elasto-plastic 2D fiber elements were used for the beams and columns including the section variation at the RBS. An equivalent SDOF model was also defined as depicted in Figure 7.3a, with rigid-plastic behaviour in terms of lateral force and displacement at the effective height. As an example, the drift histories at the effective height of the building are compared in Figures 7.3b and 7.3c when both models were subjected to scaled El Centro and JMA Kobe records.

Reasonably good agreement is observed in Figure 7.3 between the detailed elasto-plastic non-linear model and the much simpler rigid-plastic oscillator. Although there are inevitable differences in the shape of the response, including the residual drift, the peak drift is fairly well predicted. This favourable comparison is observed despite the limited inelastic demand imposed with ratios of PGA to acceleration at yield of 3 for El Centro

7. RIGID-PLASTIC MODELS

and 2.21 for the JMA Kobe record. The correlation is expected to improve with the increase in inelastic demand.

7.3 Concentrically-braced (CB) frames

The seismic performance of concentrically-braced (CB) frames depends largely on the inelastic response of their bracing members. Indeed, design approaches in seismic codes typically aim to concentrate inelastic deformation in the bracing members while ensuring elastic behaviour in other parts of the structure by means of the application of capacity design principles and failure mode control [42].

The main behavioural aspects of the cyclic response of braces are: (i) successive elongation that causes yielding in tension to occur at increasing axial deformations and, (ii) gradual degradation of the compression resistance with increasing cycles. Several models have been proposed to estimate the cyclic response of bracing members [18, 61, 74, 132]. Nevertheless such models require detailed input (e.g. non-linear constitutive model for the material, discretization of the cross-section, member imperfections). In this section a simple modelling alternative is proposed through a rigid-plastic representation at the storey level. The model is validated against detailed elasto-plastic single and multi-storey representations, and its accuracy in predicting global peak displacement is examined.

7.3.1 Rigid-plastic idealization for CB frames

The proposed rigid-plastic relationship for CB frames is based initially on the behaviour of a single-storey braced oscillator with pinned joints and rigid elements as shown in Figure 7.4a, where all the inelastic behaviour is concentrated in the bracing members.

Figure 7.4b presents the corresponding rigid-plastic hysteretic response in terms of storey drift and force. It is important to note that the model takes into account the tension elongation by allowing rigid behaviour only if the previous peak deformation is reached. Furthermore, the model assumes that the system is not able to sustain any load reversal until the load carrying capacity is recovered by tension action in

7. RIGID-PLASTIC MODELS

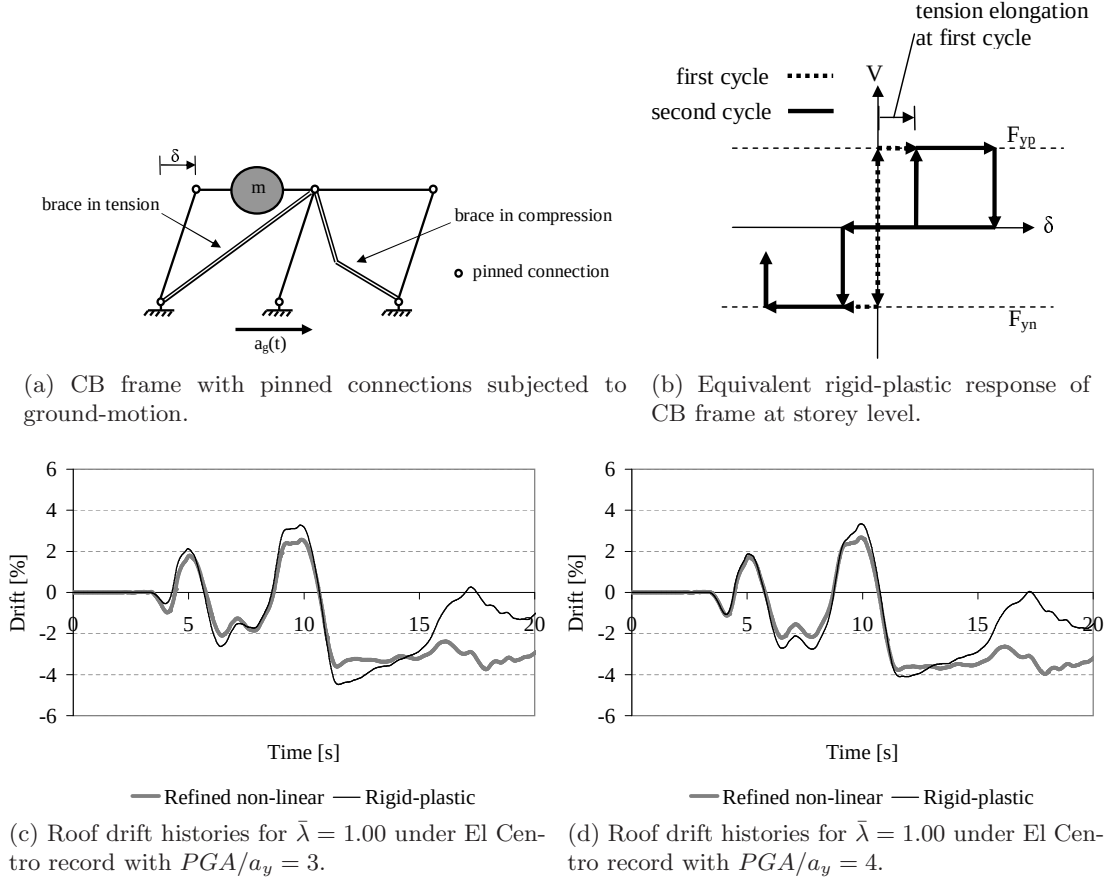


Figure 7.4: Rigid-plastic analysis of a SDOF CB frame.

the alternate brace, hence, introducing effectively a slip-like behaviour. The adopted idealisation ignores the contribution of the braces in compression which in principle is acceptable for slender braces noting that the brace will buckle at a relatively early stage in the response history [18, 44]. Where necessary, for bracing members with relatively low slenderness, an alternative model proposed in the next section can be used in order to take into account the contribution of the compression resistance of the bracing members.

Following the previous discussion, a new set of equations defining the rigid-plastic model

7. RIGID-PLASTIC MODELS

can be devised:

$$\begin{aligned}
 a_r(t) &= \frac{F_{yp}}{m} - a_g(t) & \text{if } v_r(t) > 0 \wedge d_r(t) > d(t_p) \\
 a_r(t) &= \frac{F_{yn}}{m} - a_g(t) & \text{if } v_r(t) < 0 \wedge d_r(t) < d(t_n) \\
 a_r(t) &= -a_g(t) & \text{if } F(t) = 0 \wedge d_r(t) \in]d(t_n), d(t_p)[\\
 a_r(t) &= 0 & \text{if } v_r(t) = 0 \wedge F(t) \in]F_{yn}, F_{yp}[
 \end{aligned} \tag{7.5}$$

where t_n and t_p are the instants of time at which the peak negative or positive displacement is reached, respectively, within a specific interval of time $[0, t]$. As a result, the integration algorithm proposed in Figure 7.5 can be used, where all variables are the same as defined before. Also, as in the previous section, the value of acceleration is considered constant over a Δt period of time.

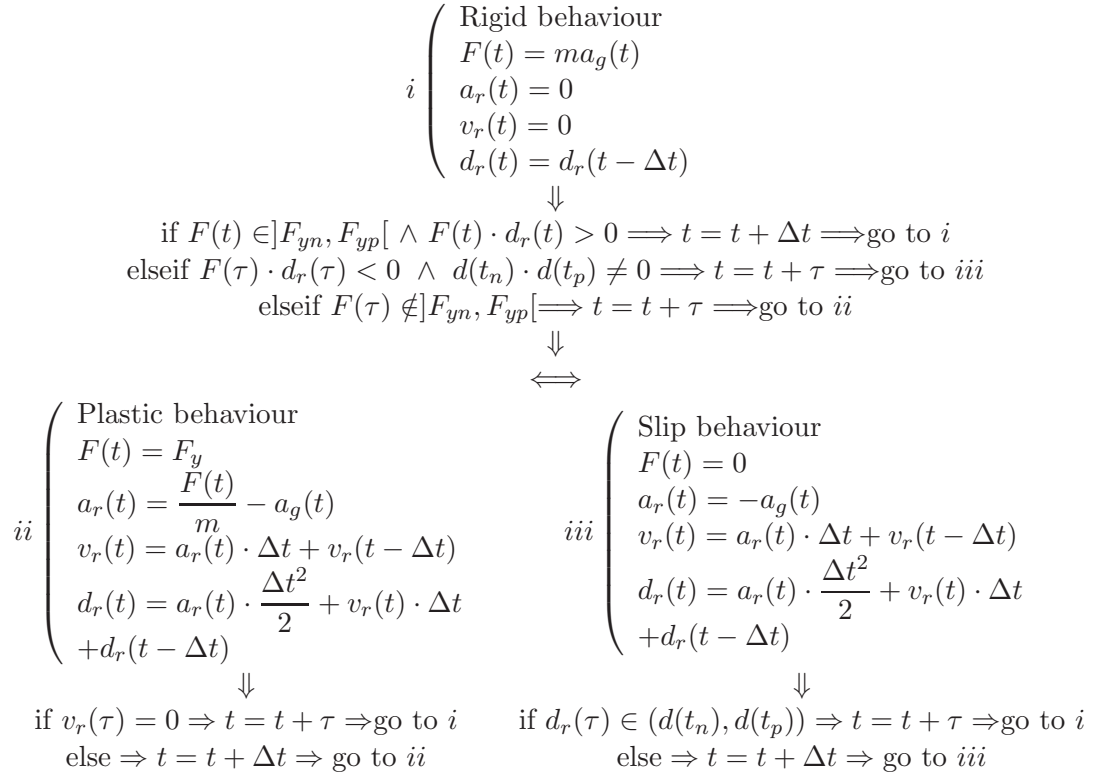


Figure 7.5: Integration algorithm for rigid-plastic concentrically-braced frames.

7.3.2 Response prediction of CB SDOF systems

The response of the rigid-plastic oscillator presented in the previous section is validated herein against refined analyses where attention is placed on the prediction of the displacement history. The single-storey two-bay SDOF CB frame shown in Figure 7.4a is considered with 5 m overall width and 2.9 m height. A refined numerical model was constructed in *Adaptic* using pinned connections and rigid elements for all the structural members except the braces. The elastic modulus of steel is assumed as $200 \times 10^3 \text{ N/mm}^2$ and its yield stress as 300 N/mm^2 with 0.5% of strain hardening. The braces were modelled with pinned ends and are formed of eight cubic elasto-plastic elements with 50 cross-sectional fibres. Several rectangular hollow sections were used which in turn signify a range of different member slenderness values. Notional loads at the midspan were used in order to simulate initial imperfections of the order of $1/200$ of the brace length.

The only parameter defining the rigid-plastic response is the storey shear capacity at yielding of the bracing system. This was calculated considering the plastic section capacity of the brace in tension only. Both models (i.e. detailed non-linear and rigid-plastic) were subjected to a range of ground-motion acceleration histories. As an example, the rigid-plastic prediction is compared with the refined fibre-element model in Figures 7.4c and 7.4d for the case of a 50x2.5 SHS brace (member normalised slenderness $\bar{\lambda} = 1.00$) to the El Centro record and for two different levels of magnification, namely PGA/a_y about 3 and 4. As shown in the figures, very good comparison is observed. The same favourable correlation was confirmed through further analyses with other ground-motion records.

Figure 7.6 explores the variation of the ratio between the peak drift predictions obtained by means of the rigid-plastic and the refined model as a function of slenderness and PGA/a_y for the Beverly Hills Station record of the 1994 Northridge earthquake. As expected, the greater the ductility demand, the more accurate the rigid-plastic approximation for a given value of slenderness is. On the other hand, for stockier braces, the rigid-plastic model tends to overestimate the peak drift due to its lower energy dissipation capacity (narrower hysteresis loops) that ignores the contribution of compression resistance.

In general, good estimates of drift history can be expected for ratios of PGA/a_y greater

7. RIGID-PLASTIC MODELS

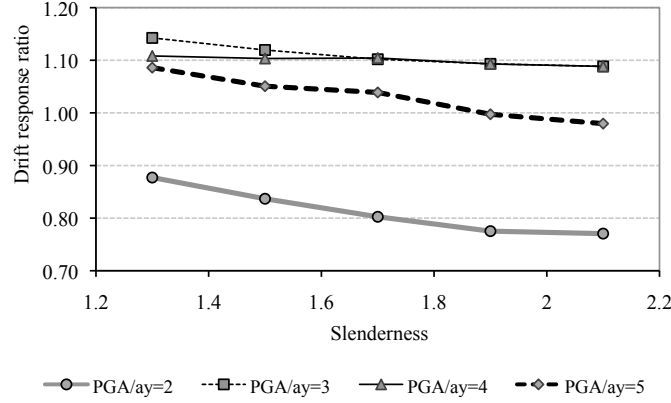


Figure 7.6: Ratio of peak drift predictions (rigid-plastic to non-linear model) as a function of slenderness and PGA/a_y for the Beverly Hills Station 1994 Northridge Earthquake record.

than three and slenderness larger than 1.5. Under these conditions the displacement error is typically less than 10%.

7.3.3 Response prediction of multi-storey CB systems

In this section, the possibility of using the above-described rigid-plastic representation in estimating the seismic response of low-rise CB frames is explored. The four-storey simple-braced structure presented in Figure 7.7, is considered herein. Figure 7.7 also presents the direction of analysis. All columns are assumed to be pinned at the base and simple beam connections are considered.

The building was designed to EC8 [28] employing seismic actions corresponding to Soil C conditions and Spectrum Type 1 together with a PGA of $0.3g$ and a behaviour factor of 4. The dead load included the weight of partitions ($1kN/m^2$), finishing ($0.3kN/m^2$) and a composite flooring system ($2.88 kN/m^2$ on average), while $2.5 kN/m^2$ was considered as superimposed load.

As suggested in the code, the design assumed that all the lateral resistance is provided by the tension braces. Code limits related to second-order stability checks and inter-storey drift limits were satisfied in the design. Figure 7.8a presents the relationships between base shear and roof drift for the four-storey building obtained from a non-linear static analysis with a vertical distribution of loads following the fundamental modal shape. In the figure, the design base shear value is also represented and hence the overstrength due to the contribution of braces in compression is evident. It

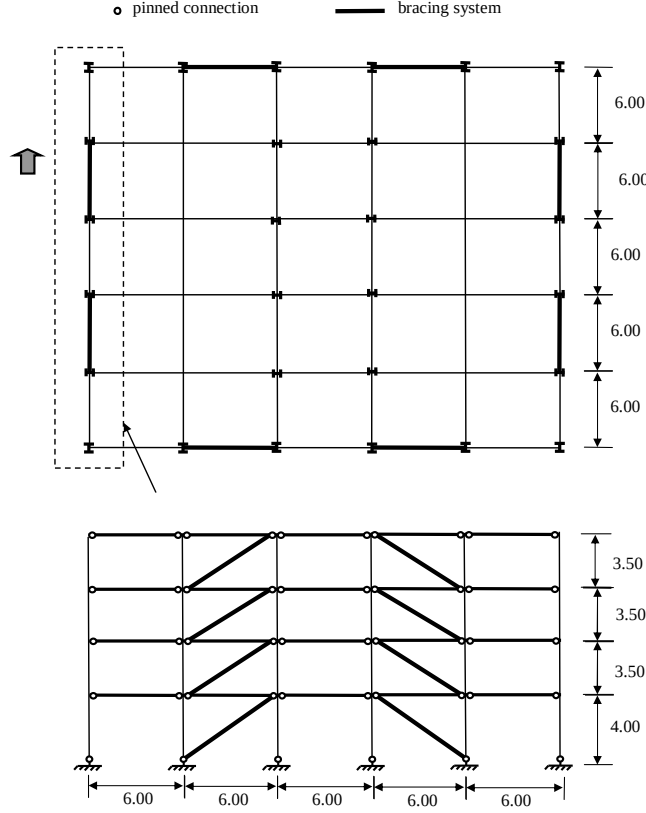


Figure 7.7: Plan layout and elevation of the four-storey CB frame under study.

should be noted that the equivalent SDOF model employed in this study is based on the assumption that the response of the structure is dominated by the first mode of vibration, and that the design is based on achieving a plastic mechanism that avoids storey localisation.

The building was modelled in Adaptic with due account of material and geometric non-linearities. Columns and braces were discretized in a number of finite elements with cubic shape functions. Initial imperfections at midlength of the braces were induced by means of the notional load method as described previously. The elastic modulus of steel was taken as $200 \times 10^3 \text{ N/mm}^2$ and its yield stress as 300 N/mm^2 with 0.5% of strain hardening. This value of material strain hardening was assumed based on typical modelling practice. It is worth noting however that use of a value between 0% and 0.5% has no notable influence in the response within the level of displacement demand considered.

7. RIGID-PLASTIC MODELS

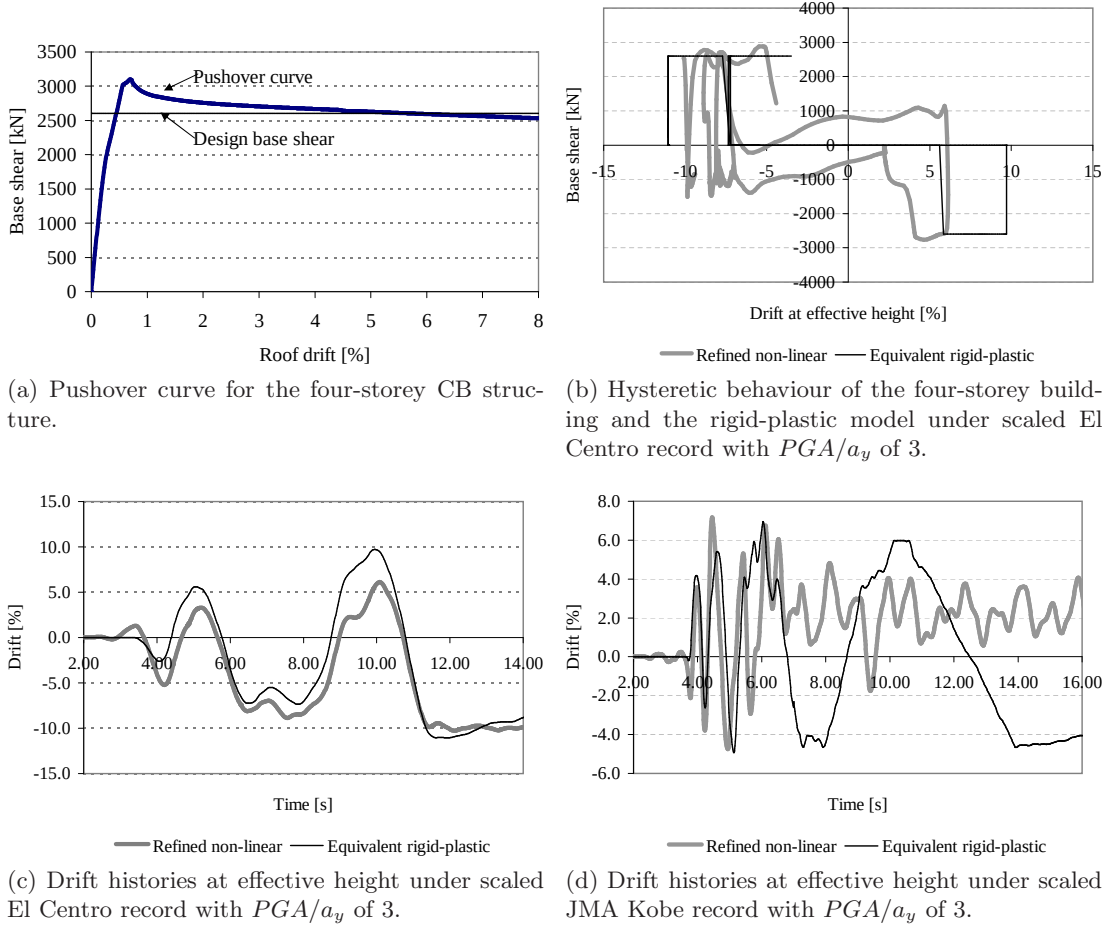


Figure 7.8: Rigid-plastic analysis of a four-storey CB frame.

The equivalent rigid-plastic oscillator was defined in terms of the effective mass and height considering an inverted triangular distribution of displacements and a plastic resistance calculated from the nominal brace resistance in tension only. Both models were subjected to the El Centro and JMA Kobe records scaled so as to induce a peak force (computed from the PGA and the effective mass) of 3 times the base shear capacity of the building, respectively. The results in terms of drift at the effective height are presented in Figures 7.8c and 7.8d.

It is evident from Figures 7.8c and 7.8d that very good estimates of peak drift and close match in the drift response history are achieved over the strong part of the record. Nonetheless, after the strong demands cease to act, (particularly after 6 s in the response shown in Figure 7.8d) the actual structure responds more in an elastic fashion while

the rigid-plastic model slips between the two rigid borders determined by the previous brace tension elongation limits (e.g. $d(t_n)$ and $d(t_p)$ in Equation 7.5). Such difference in response at the later stages of the drift history is caused by the absence of resistance in the rigid-plastic model during the slipping state whereas the actual building is able to dissipate some energy through compression resistance of the braces as indicated in Figure 7.8b. This in turn means that rigid-plastic approaches may perform better under pulse-like demands typical of near-field ground-motions. A procedure to overcome such limitations is proposed in the next section where a representation of compression resistance is introduced in the rigid-plastic model.

7.4 Dual frames

The asymmetric behaviour, member elongation and loss of compressive resistance associated with brace behaviour require considerable attention in design to avoid undesirable response mechanisms. In this respect, dual systems that combine moment resisting frames with concentric bracings, are often preferred in practice, owing to an enhanced seismic performance obtained by coupling the high ductility capacity of the moment frame with the improved lateral stiffness and capacity of the bracing system. Hence, an adequate rigid-plastic characterization of the seismic response of such dual frames is desirable.

7.4.1 Rigid-plastic idealization of dual systems

The single-storey oscillator presented in Figure 7.9a is used here in order to derive a simple rigid-plastic hysteretic approximation for dual structures. The seismic response of such oscillator combines the dissipative behaviour of bracing members at the storey level addressed in the previous section with flexural resistance in the columns where classic rigid-plastic hinges are formed. The related hysteretic behaviour in terms of force and storey drift is presented in Figure 7.9b.

Typically, the lateral resistance of the bracing system would exceed the lateral resistance of the moment frame. Consequently, the effects of the rigid-plastic hinges formed in the beams or columns will be to eliminate the slip behaviour periods where no resistance is present (i.e. as in the rigid-plastic model for CB structures shown in Figure 7.4b). This allows the structure to mobilize actions in reverse loading but with a net flexural resistance F_{sn} or F_{sp} provided by hinges in the MRF. In this idealisation, the possibility of internal classic rigid-plastic hysteresis loops is introduced whenever

7. RIGID-PLASTIC MODELS

the system oscillates within the limits imposed by the elongated bracing members. The full lateral resistance is not recovered until the previous peak displacement is reached again. Hence, the set of equations defining the dual rigid-plastic response are derived as:

$$\begin{aligned}
 a_r(t) &= \frac{F_{yp}}{m} - a_g(t) \quad \text{if } v_r(t) > 0 \wedge d_r(t) > d(t_p) \\
 a_r(t) &= \frac{F_{yn}}{m} - a_g(t) \quad \text{if } v_r(t) < 0 \wedge d_r(t) < d(t_n) \\
 a_r(t) &= \frac{F_{sp}}{m} - a_g(t) \quad \text{if } F(t) = F_{sp} \wedge v_r(t) > 0 \wedge d_r(t) < d(t_p) \\
 a_r(t) &= \frac{F_{sn}}{m} - a_g(t) \quad \text{if } F(t) = F_{sn} \wedge v_r(t) < 0 \wedge d_r(t) > d(t_n) \\
 a_r(t) &= 0 \quad \text{if } v_r(t) = 0 \wedge \left(\begin{array}{c} F(t) \in]F_{sn}, F_{sp}[\wedge d_r(t) \in]d(t_n), d(t_p)[\\ \vee \\ F(t) \in]F_{yn}, F_{yp}[\wedge d_r(t) \in (d(t_n), d(t_p)) \end{array} \right)
 \end{aligned} \tag{7.6}$$

where $F_{sp} = -F_{sn}$ is the lateral resistance of the flexural plastic hinges formed in the MR frame and $F_{yp} = -F_{yn}$ is the overall lateral strength of the system considering the strength contribution of the bracing system and other variables as previously defined.

7.4.2 Response prediction of dual SDOF systems

The proposed rigid-plastic model was validated against the results of detailed non-linear response history analyses for single-storey frames with fixed bases. Cubic elasto-plastic elements with distributed plasticity were employed for the braces and columns. The elastic modulus of steel was taken as $200 \times 10^3 \text{ N/mm}^2$ and the yield stress as 300 N/mm^2 with 0.5% strain hardening.

Rectangular Hollow Sections (RHS) were used for the braces and several cross-sections of braces and columns were employed in order to explore the variation of the accuracy of prediction with slenderness and two levels of flexural strength contribution. Figures 7.9c and 7.9d present a typical comparison of drift histories. Figure 7.10 depicts the ratio of peak drift predictions obtained by means of the rigid-plastic and the refined non-linear model as a function of slenderness and PGA/a_y subjected to the Beverly Hills Station record of the 1994 Northridge earthquake when 15% and 30% of the total lateral strength is contributed by flexural plastic hinges.

7. RIGID-PLASTIC MODELS

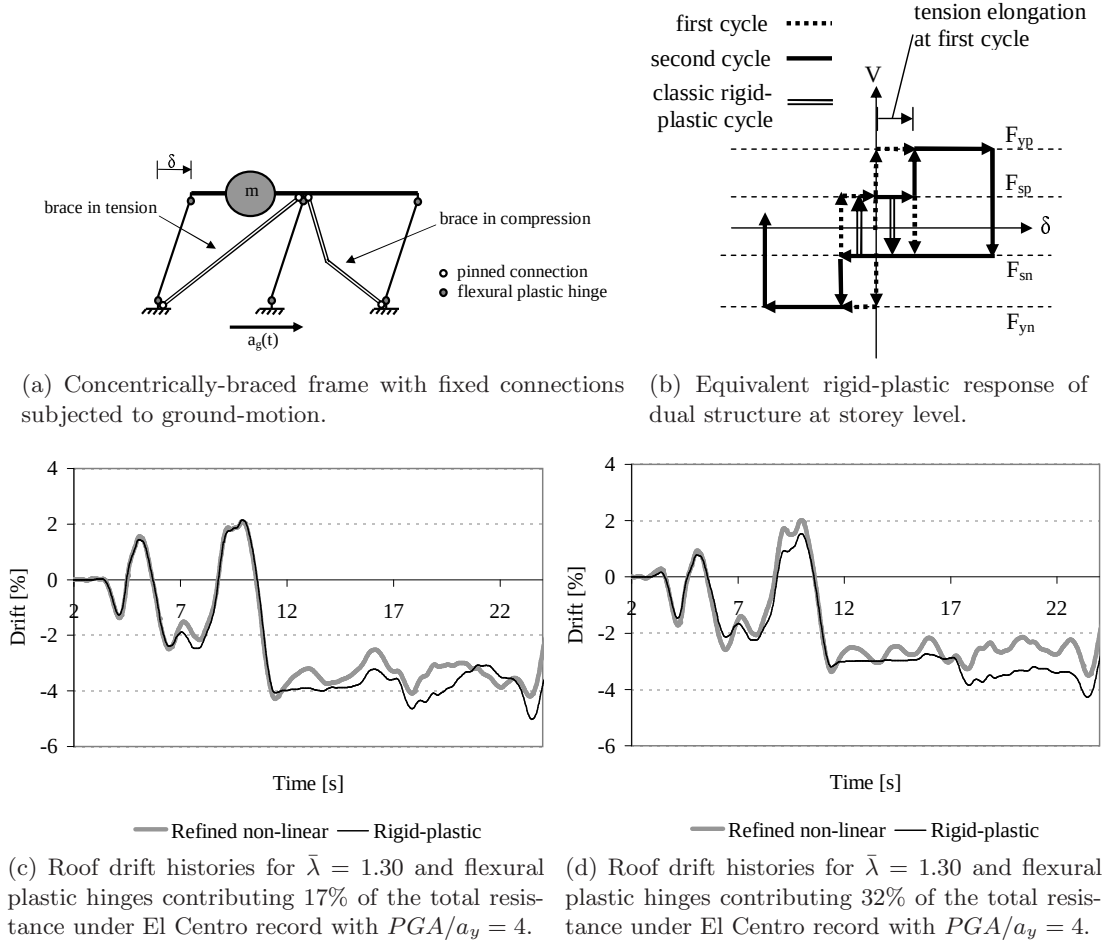


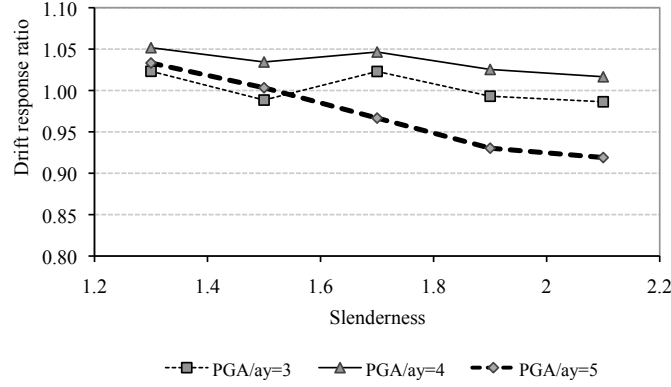
Figure 7.9: Rigid-plastic analysis of dual SDOF structures.

For records with very large PGA and more slender braces, the rigid-plastic approximation indicates a slight tendency to underestimate the peak drift predictions compared to the refined non-linear model. Nevertheless, in all cases the estimates are within the $\pm 10\%$ band. In general, for the full range of slenderness studied here (from 1.3 to 2.1) the rigid-plastic procedure provided good predictions of peak displacements when the frame was subjected to acceleration time histories with PGA of 3 or more times the acceleration at yield.

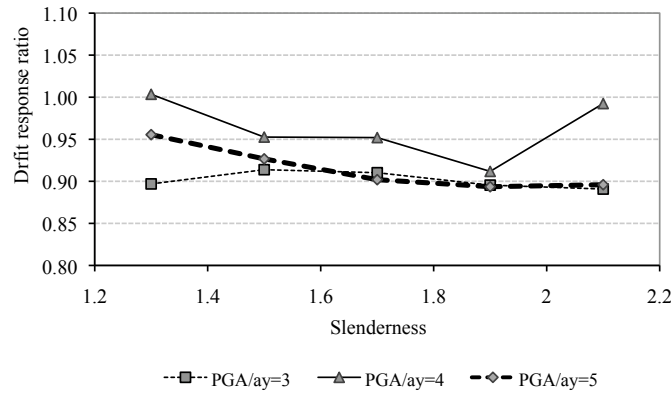
7.4.3 Response prediction of dual multi-storey frames

In this section, the ability of the proposed rigid-plastic model is assessed against the prediction of displacement histories obtained by means of a detailed non-linear model

7. RIGID-PLASTIC MODELS



(a) Ratio of peak drift predictions (rigid-plastic to non-linear model) when 15% of the total lateral strength is contributed by flexural plastic hinges for the Beverly Hills Station 1994 Northridge earthquake record.



(b) Ratio of peak drift predictions (rigid-plastic to non-linear model) when 30% of the total lateral strength is contributed by flexural plastic hinges for the Beverly Hills Station 1994 Northridge earthquake record.

Figure 7.10: Effect of brace slenderness and ratio PGA/a_y in the accuracy of rigid-plastic predictions.

of a four-storey building. The structure under consideration has the same plan configuration and elevation as the building presented in the previous section (Figure 7.7). However, in this case, fixed bases were assumed at the bottom of columns, hence a new design in accordance with EC8 [28] was performed. The same dead and superimposed loads used previously were utilized here. The pushover curve in terms of base shear and roof displacement of the modified building using a first-mode vertical distribution of loads is presented in Figure 7.11a. In this figure, the total design base shear value is also indicated.

The same modelling considerations described in Section 7.3.3 were adopted and both

7. RIGID-PLASTIC MODELS

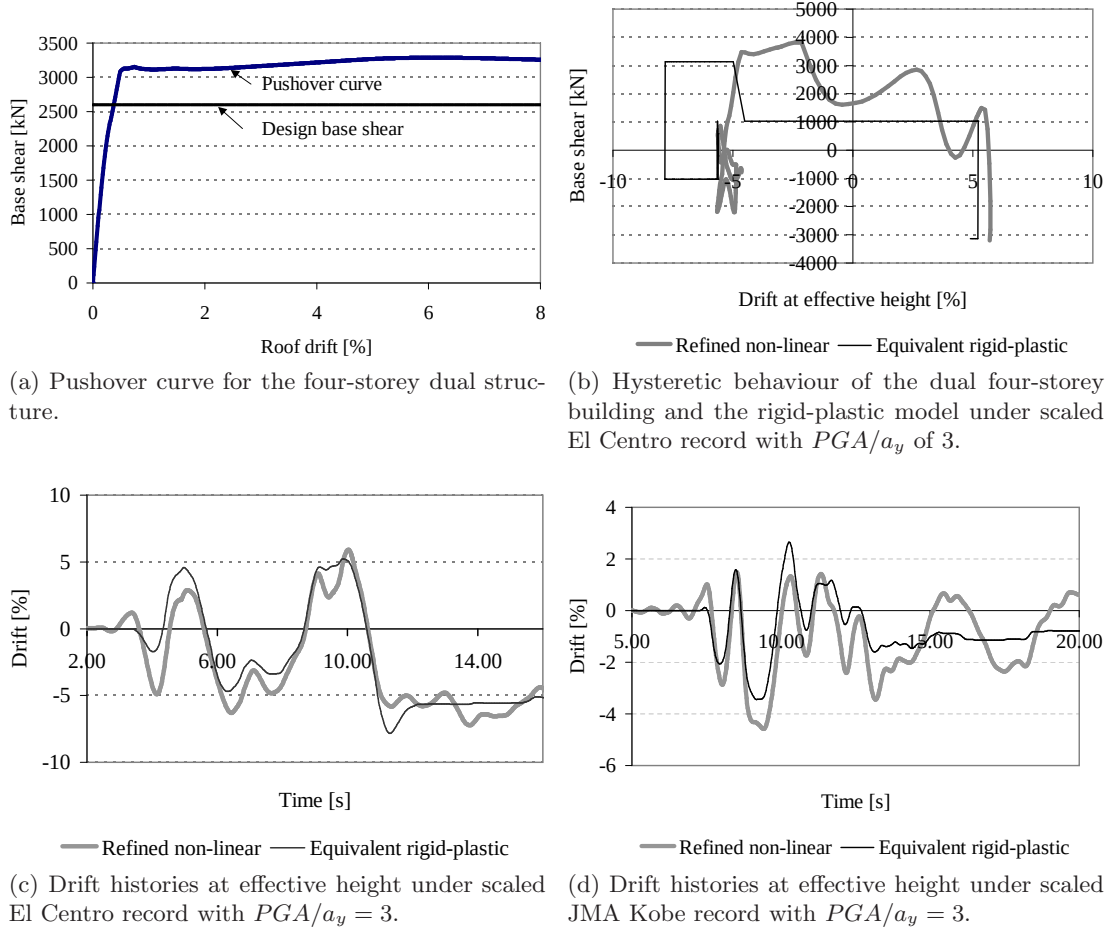


Figure 7.11: Rigid-plastic analysis of a four-storey dual frame.

the detailed MDOF non-linear model and its equivalent rigid-plastic representation were used. The corresponding results in terms of drift at effective height are presented in Figures 7.11c and 7.11d, where good agreement over the whole response history can be observed. The El Centro record as well as the JMA Kobe earthquake record were scaled so as to induce a peak force (computed from the PGA and the effective mass) of 3 times the actual base shear capacity of the buildings.

Figure 7.11b presents the hysteretic response of the four-storey building when subjected to the El Centro record inducing a PGA of 3 times the yield acceleration of the equivalent rigid-plastic model. It is evident that the actual building develops greater lateral forces while oscillating between the rigid states, which is largely attributed to the contribution of compression forces in the braces that were not taken into account

7. RIGID-PLASTIC MODELS

when calculating the oscillator's capacity. Nevertheless, this occurs over a relatively small period of time and hence favourable predictions are still achieved.

7.5 Rigid-plastic design procedures

7.5.1 Relationship between strength and structural response

As noted previously, the only parameter defining rigid-plastic models, such as the ones proposed in earlier sections, is the plastic capacity either in terms of yield force or moment. Even for dual systems, the contribution of the flexural plastic hinges to the overall lateral capacity of the structure can be expressed as a percentage of the total lateral strength. This makes response history analysis a relatively straightforward task once the lateral strength of the structure has been evaluated.

From another viewpoint, the single-parameter characteristic of rigid-plastic dynamics enables the evaluation of the required strength given a certain response target. In this context, plots of peak displacement as a function of the lateral capacity of the rigid-plastic oscillator have been proposed by Paglietti and Porcu [124] and referred to as rigid-plastic "spectra" to characterize the earthquake demand. This approach has been employed by Domingues-Costa et al. [36] to design reinforced concrete MR frame structures. In this section, the principal characteristics which were already pointed out by previous researchers are summarized and the concept is further extended to steel structures, with particular emphasis on braced and dual frames.

Figure 7.12 presents a plot of the relationship between peak displacement and ultimate lateral capacity for a dual rigid-plastic oscillator such as the one presented in Figure 7.9. In this case the lateral resistance due to flexural action represents about 20% of the total lateral capacity of the structure. The curve was obtained by running a series of response history analyses for several values of lateral resistance and employing the Sylmar Station record of the 1994 Northridge Mw=6.5 earthquake with 7.5 km source to site distance. The lateral resistance is expressed as a function of force divided by acceleration at the onset of plastic behaviour and the system mass, thus giving a non-dimensional value and facilitating the scaling of the curve for different values of yield capacity. It is worth noting that linear scaling of the mass or the acceleration time series will simply imply multiplying the ordinates of the curve by the

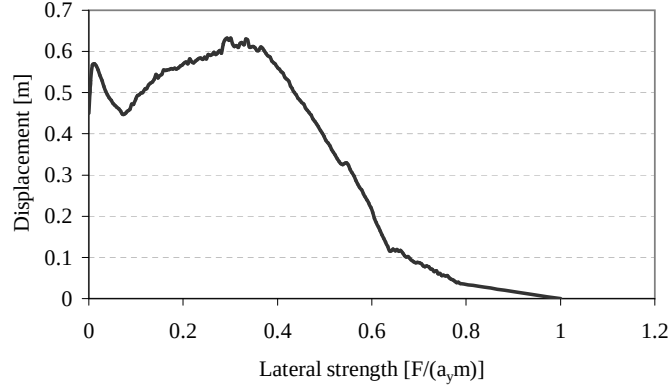


Figure 7.12: Rigid-plastic relationship between lateral strength and peak displacement for a dual system with flexural strength of 20% of the total capacity when subjected to the 1994 Northridge earthquake Sylmar Station record.

corresponding scaling factor, which offers a great deal of versatility to plots of this form.

The point at which the curve crosses the horizontal axis corresponds to a lateral strength equal to the PGA times the mass of the oscillator. In turn, the point at which the curve crosses the vertical axis represents the peak ground displacement (PGD) since, at this point, no structural resistance is present and the equation of motion reduces to the double integration of the input acceleration.

Another feature of this type of relationship is the fact that the curve is not entirely smooth and some peaks and valleys do take place (for example at 0.1 of F/ma_y in Figure 7.12). This reflects the dependence on the earthquake acceleration waveform signifying that different levels of strength may still give the same value of peak displacement (e.g. 0.5m).

7.5.2 Rigid-plastic design of steel buildings

In this section, the use of rigid-plastic relationships between yield capacity of the structure and response parameters for design purposes [36] is illustrated for dual steel frames. This is carried out through a design example of a six-storey braced building with fixed columns and constant storey height of 3.0 m which plan and elevation configurations are as presented in Figure 7.13.

The gravity loads and mass were calculated in the same way as described previously in Sections 7.3.3 and 7.4.3. The equivalent rigid-plastic oscillator was considered assuming

7. RIGID-PLASTIC MODELS

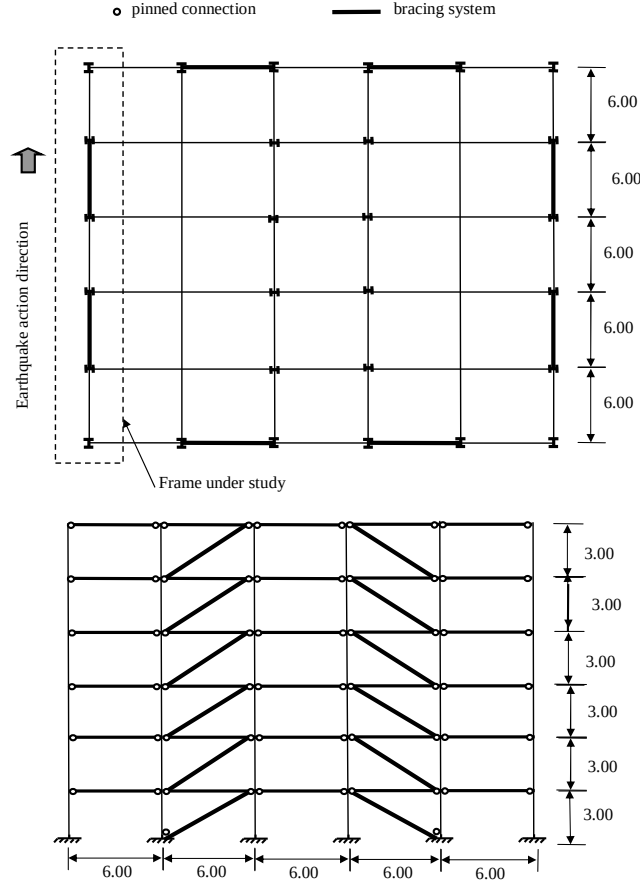


Figure 7.13: Plan layout and elevation of the six-storey steel frame with fixed bases used for design.

a mechanism with yielding of the bracing members and formation of plastic hinges at the base of columns. Hence, the dual oscillator of Section 7.4 is used with an effective mass of 411 505 kg and an effective height of 12.62 meters. An inverted triangular displacement shape was assumed and a target drift of 3.5% was used. Furthermore, a 20% contribution of the moment resisting system at the base of the columns to the overall system resistance is considered as a design choice.

The seismic demand is represented by 8 records from the dataset developed for the SAC project [142] which are scaled in order to be consistent with a 2% probability of exceedance in 50 years for Los Angeles City, USA. The characteristics of these records are presented in Table 7.1. The first five records (1 to 5) were used to generate the rigid-plastic relationship between peak drift at effective height and base shear demand

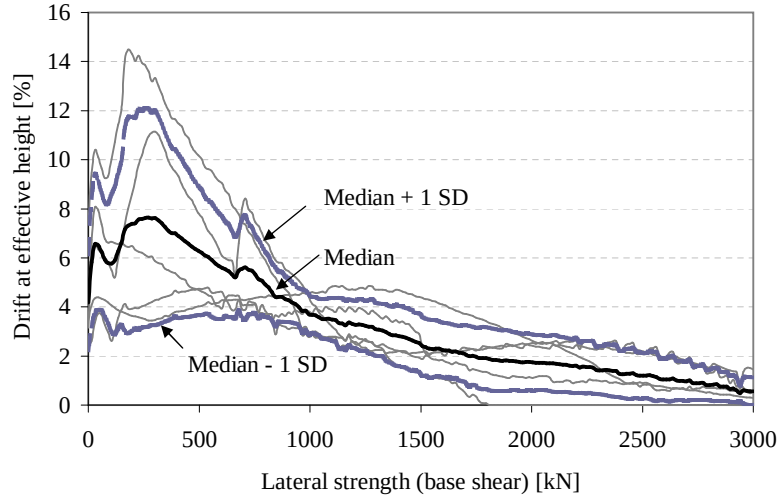


Figure 7.14: Drift at effective height vs. base shear relationship for dual rigid-plastic oscillators with flexural resistance of 20% of the total building capacity. Thin lines represent relationships for individual records. The median values and median ± 1 standard deviation are highlighted. The standard deviation corresponding to 3.5% design drift is $\pm 0.8\%$.

(presented in Figure 7.14), while the last three records (6 to 8) were used for validation purposes.

Table 7.1: Characteristics of Los Angeles ground-motions [142] with a 2% probability of exceedance in 50 years, used for the design example.

ID	Earthquake name	Magnitude Mw	Distance [km]	Scale factor	Duration [s]	PGA [m/s ²]
1	1995 Kobe	6.9	3.4	1.15	60	9.03
2	1989 Loma Prieta	7	3.5	0.82	25	4.64
3	1994 Northridge	6.7	6.4	1.61	60	9.09
4	1994 Northridge	6.7	6.4	1.61	60	1.3
5	1974 Tabas	7.4	1.2	1.08	50	9.73
6	1994 Northridge	6.7	7.5	1.29	15	9.25
7	1974 Tabas	7.4	1.2	1.08	50	7.93
8	Elysian Park (simulated)	7.1	17.5	1.43	30	1.27

From Figure 7.14, a lateral resistance of at least 1000 kN seems necessary to achieve the target drift of 3.5% (which corresponds to a displacement of 64 cm at the roof level). The average response of the whole set was used in recognition of the common engineering practice of focusing on average values. Other storey forces were calculated in accordance with the shape of the plastic mechanism assumed.

The design force for bracing members at the bottom storey was defined as 800 kN,

7. RIGID-PLASTIC MODELS

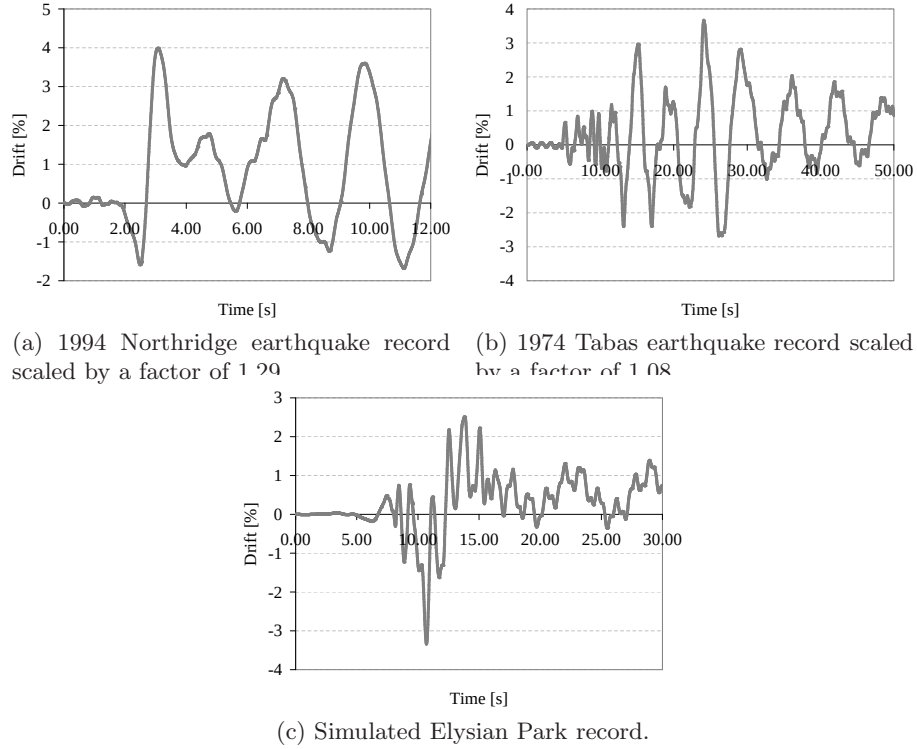


Figure 7.15: Drift at equivalent height of the 6-storey steel braced building with fixed columns when subjected to records corresponding to a 2% probability of exceedance in Los Angeles area.

while the force resisted by the flexural capacity of the plastic hinges assumed to form at the bottom of the columns was considered as 200 kN (20% as assumed previously). In fact, a UC203x86 section at the bottom columns would have been enough to resist the gravity loads with due consideration of $P-\Delta$ effects if the whole lateral load was assigned to the braces. Nevertheless, a UC305x97 was used here to provide the 20% of flexural capacity assumed in the first step of the design process. The design of the braces was then performed but allowing for non-dimensional slenderness of up to 2.5 in order to achieve a close match between demand and capacity throughout the height of the building. Capacity design principles were followed for the seismic design of other members. The building was then subjected to the last three ground-motions of Table 7.1, which are consistent with the same probability of exceedance of the target scenario, and the results are presented in Figure 7.15.

In general, good agreement is observed with an average peak drift of 3.7% compared to the target of 3.5% and a standard deviation in the peak roof drift of $\pm 0.3\%$. In effect, the methodology succeed in combining the reliability of response history analysis tech-

niques with the simplicity of rigid-plastic models with the aim of providing a reliable design based on structural performance at a given level. Indeed, it has been recognized that spectra in terms of the maximum plastic displacement are more convenient for design purposes than spectra in terms of maximum seismic acceleration and ductility factors [51, 128].

It is worth noting that previously proposed displacement-strength design relationships based on rigid-plastic response history used the so called "envelope" [37] with little or no correspondance with a specific earthquake scenario. In the present study the median value was considered as a more meaningful representation of such relationships. However, the median plus a number of standard deviations could also have been used but this would perhaps require a larger number of records in order to capture with confidence the variability associated with the process. Also, the displacement-plastic resistance curve expressed in terms of the median does not imply that such required strength will be associated with the median probability as the ability of the rigid-plastic oscillator varies as a function of the plastic resistance itself. Further studies are however required to provide additional validation, covering other structural configurations and earthquake records, for the methodology illustrated above.

In theory, a designer can follow an approach through which serviceability checks, associated with frequent events, can be carried out using conventional elastic models. On the other hand, validation of the structural response under extreme events can be performed using rigid-plastic approaches. This can be implemented in a performance-based design framework of the type presented in Figure 7.16. Such a design framework would allow reliability levels to be estimated with due account of the real earthquake variability. In addition, such framework would facilitate the understanding and selection of performance levels and enable different levels of structural optimization without the computational demand and inherent complexity associated with non-linear finite element response history analysis. Furthermore, the direct consideration of the time dependent nature of the seismic action can be further exploited to obtain preliminary estimates of local damage indexes as illustrated in Appendix C.

7. RIGID-PLASTIC MODELS

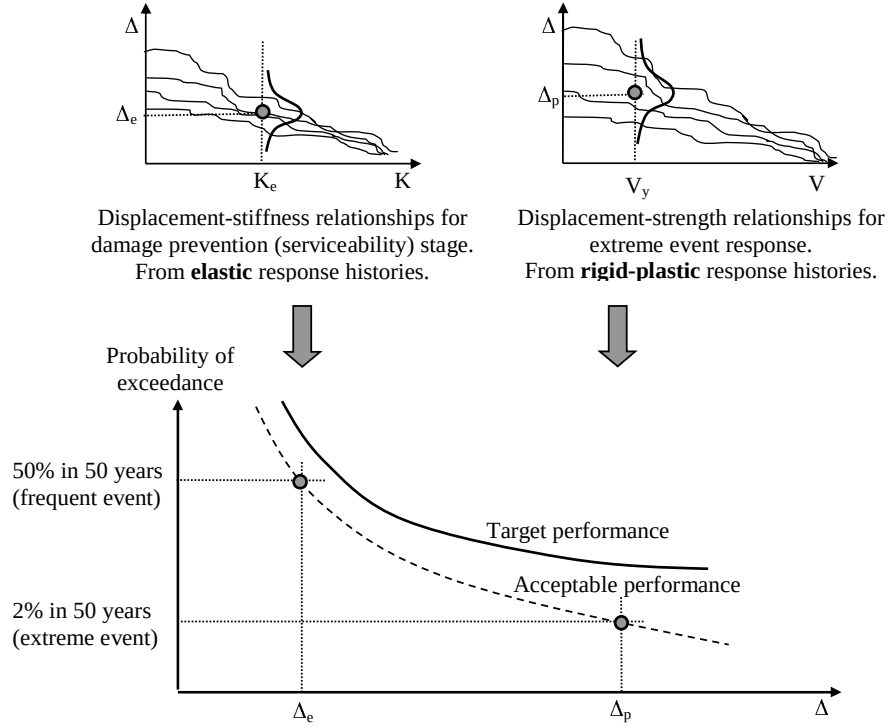


Figure 7.16: Seismic design framework with de-coupled assignment of stiffness and capacity based on elastic and rigid-plastic models.

7.6 Concluding remarks

The applicability of rigid-plastic models in predicting the seismic behaviour of typical steel framed buildings has been investigated with particular emphasis on moment resisting (MR), concentrically-braced (CB) and dual configurations. The principal advantage of using response history analysis based on rigid-plastic models has been highlighted as their ability to predict with good levels of accuracy, and for well-defined conditions, the structural response while keeping the computational costs and the detailed knowledge required at a relatively low level.

For MR frames, the classical rigid-plastic model was used in conjunction with the transformation of a MDOF into an equivalent SDOF representation based on a specified plastic mechanism. It has been shown that such rigid-plastic model can give good estimates of global deformations when relatively high levels of non-linear response (typically for $PGA/a_y > 3$) are expected. On the other hand, for CB and dual systems, this chapter proposed new rigid-plastic hysteretic models that take into consideration the

specific behavioural characteristics of such structural configurations. In addition, for the newly developed models, this study has shown that in general, good estimates can be achieved if the ground-motion pushes the structure well into the inelastic range with the ratio of PGA over acceleration at yield serving as a good parameter to quantify such ability.

It was shown in this chapter that the more slender the braces in CB systems, the better the response prediction obtained from the idealised rigid-plastic model suggested in this chapter, particularly when braces with a non-dimensional slenderness greater than 1.5 are employed. Nevertheless, when the contribution of flexural strength is taken into account, such dependence on the slenderness value is reduced and the response is adequately predicted for the whole range of slenderness studied herein (from 1.3 to 2.1). An additional parameter affecting the accuracy of rigid-plastic models for dual systems was shown to be the extent of flexural strength contribution to the total lateral capacity of the system. In general, when the flexural plastic hinge contribution increases, the rigid-plastic model tends to underestimate the displacement response, but such underestimation was typically less than 10% for all the cases considered. Besides, the rigid-plastic idealizations employed in this study also share the limitations of equivalent SDOF representations, and require the selection of a pre-defined plastic mechanism. The approach is therefore suitable for regular low and medium-rise structures which are dominated by the first mode of vibration, and in which a pre-defined plastic mechanism is ensured. In addition, the rigid-plastic model does not account for strength degradation which may be present under severe cyclic conditions.

Finally, the potential use of design methodologies based on rigid-plastic relationships between plastic capacity and demand parameters was illustrated, although this needs to be examined further in future studies covering more buildings and a larger data set of records. Overall, the results presented in this chapter suggest that rigid-plastic models can be used in some situations as a simple approach for predicting seismic response.

Part IV

Building Systems

Chapter 8

Seismic Response of Steel Framed Buildings

8.1 Introduction

As discussed in earlier chapters, the main aim of this thesis is to investigate key behavioural and design aspects of steel structures incorporating tubular columns and/or braces. To this end, the lack of investigations on cost-effective semi-rigid connections between open beams and tubular columns (as compared with the extensive research available on fully-rigid details) motivated the experimental and analytical assessments of the monotonic and cyclic response of practical bolted angle connections reported in Chapters 3 to 5. In turn, Chapters 6 and 7 of this thesis studied the seismic demands on steel structures incorporating those previously proposed semi-rigid connections, as well as frames with fully-rigid connections and others employing tubular braces. Extensive response history analyses on pinching, bi-linear, braced and rigid-plastic single-degree-of-freedom (SDOF) systems were carried out and the results were subsequently validated for multi-degree-of-freedom (MDOF) planar 2D frames. Finally, this Fourth Part of the thesis is concerned with assessing the effects of different framing layouts on the seismic performance of complete structural building systems with tubular members. Therefore, the main objective of the present chapter is to identify general trends in the response of a number of framing systems involving different structural arrangements and connection details subjected to bi-directional earthquake loading. Particular focus is given to examining the influence of secondary/gravity frames on the reduction of peak response both in the small to moderate and the large displacement demand ranges.

8. FRAMING SYSTEMS

To this end, this chapter presents results of peak response in one-way, two-way and mixed framing configurations incorporating concentrically-braced (CB) frames and moment resisting (MR) frames as primary lateral resisting systems. Extensive non-linear response history analyses over a number of idealized three-storey 3D structures are performed under a suite of earthquake records. The performance level of interest in this evaluation is related to building damage and therefore, severe structural deterioration is not considered. It is demonstrated that secondary systems, which are normally designed to carry gravity loads only, can play a significant role in reducing peak inter-storey drift demands if the primary system is formed by stiff CB frames, whereas no significant improvement is observed for buildings with primary MR frames. It is also shown that by modifying the two-way configuration through the introduction of variable degrees of moment release at selected beam-to-column connections, significant reductions in drift demands can be obtained. This chapter then continues to examine the effects of secondary/gravity systems in the near-collapse performance region by means of advanced 2D models incorporating a number of structural degradation sources. This enables a detailed quantification of the contribution of gravity frames to the reduction of financial risks and highlights the benefits of proper secondary frame design in mitigating the probabilities of dynamic instability.

8.2 Steel framing systems incorporating tubular members

Figure 8.1 presents typical plan views of the four framing systems considered in this study, including:

- Two-way framing system in which all beams and columns are proportioned to resist the simultaneous action of seismic and gravity loads (Figure 8.1a).
- One-way systems that distinguish between a few primary resisting frames considered to carry lateral loads and other secondary/gravity frames designed to carry gravity loads only. One-way framing systems with either: (i) MR frames (Figure 8.1b) and (ii) CB frames (Figure 8.1c) acting as primary lateral resisting systems are considered.
- A fourth type of framing configuration (indicated as mixed framing in Figure 8.1d) is formed by modifying the two-way frame through the introduction of variable degrees of moment release at selected beam-to-column connections. (e.g. by introducing connection details similar to those proposed in Chapters 3 to 5).

Tubular columns are considered in all cases with symmetric four bay-by-four bay plan layout and 6.00 m width per bay. Only three-storey buildings with constant 3.00 m storey height are studied. It is believed that this offers a comparative insight into the general behavioural trends and provides a good basis for future studies incorporating a wider range of geometric variations.

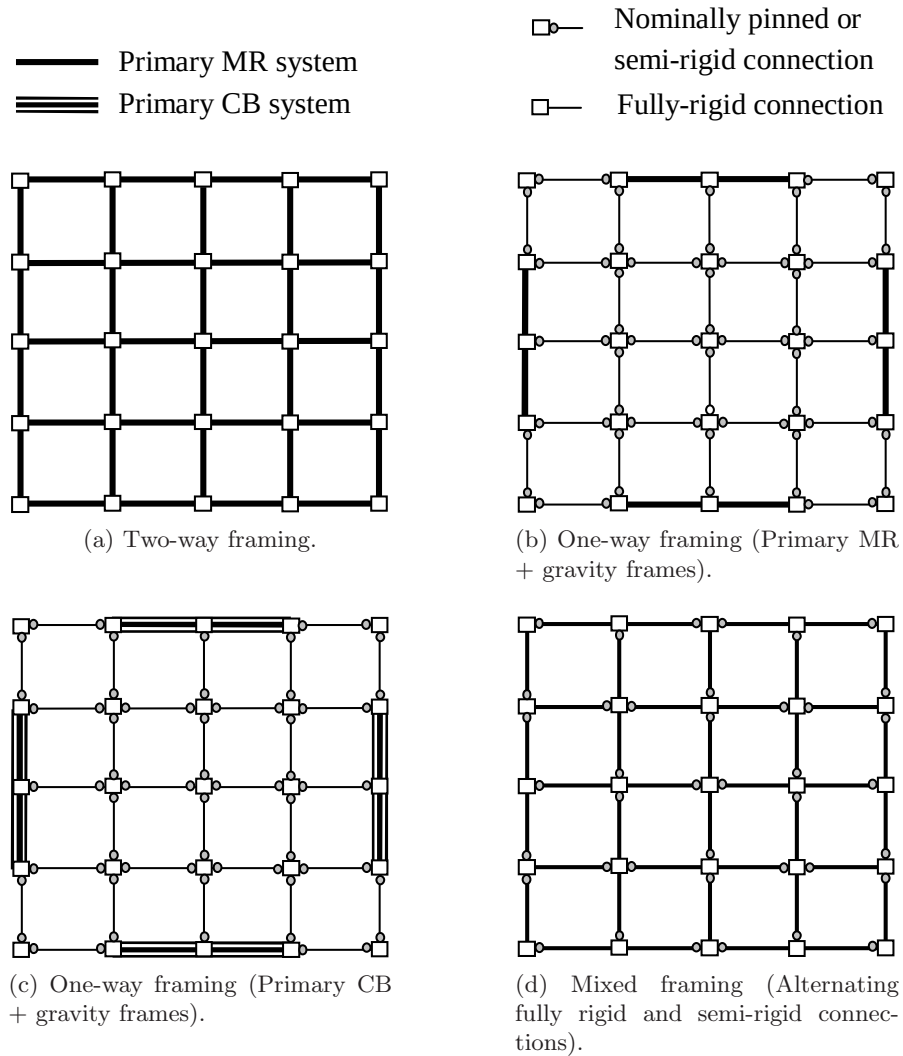


Figure 8.1: Framing systems.

8.3 Seismic drift demands in building configurations

8.3.1 3D frame idealizations

In order to identify general response trends, extensive non-linear analyses are performed using idealized 3D models aimed at representing the main behavioural characteristics of the buildings under consideration. Such simplified 3D models were first proposed by Tawaga et al. [146] in the context of comparing annual probabilities of connection failure between one-way and two-way framing systems. The ability of the idealized 3D structures to represent the behaviour of complete building systems is illustrated in Figure 8.2, where a typical comparison of drift histories obtained by means of simplified and full building models by Tawaga et al. [146] is presented. Nevertheless, the afore-mentioned study focused on buildings with constant steel volumes (rather than a range of comparable periods/stiffnesses) and the contribution of secondary frames was neglected. Besides, only MR were considered and neither CB nor mixed framing systems were studied. In contrast, the present investigation takes into account varying levels of primary/secondary frame interaction and includes the response of CB and mixed framing systems.

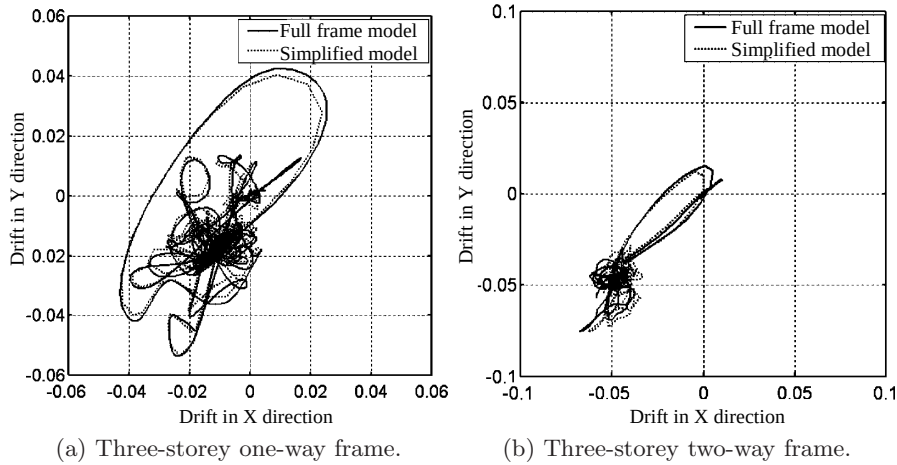


Figure 8.2: Comparison of histories of first storey drift in complete frame and simplified models by Tagawa et al. [146].

Figure 8.3 depicts the steps involved in establishing an equivalent 2D fishbone model of a one-way building. Due to symmetry, in-plane interaction can be assumed and thus primary and secondary frames are considered to act in series. Also, the inflection point in the primary beams is assumed to be located at midspan, and the gravity frame is

represented by a continuous column with various degrees of moment transfer at each floor level.

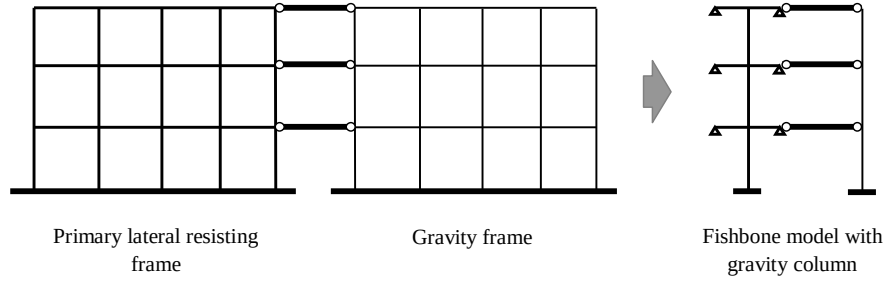


Figure 8.3: Planar simplification to a fishbone model of a one-way building.

The number of degrees of freedom of 3D building models can be significantly reduced by following the same approach used in defining fishbone 2D frame representations (Figure 8.3). Figure 8.4 presents the idealized 3D simplifications of the different building configurations presented previously (in Figure 8.1). In the case of one-way buildings (Figures 8.4a and 8.4b), the lateral resisting frames acting primarily in-plane are, as in the case of the fishbone idealization, turned into four primary frames with beam-ends supported by pin-ended rigid bars. In turn, all the gravity frames are summed-up into a single continuous column connected at each floor level (by means of pinned or semi-rigid connections) to each of the four primary frames. On the other hand, the two-way and mixed framed buildings (Figures 8.4c and 8.4d) are modelled with four columns and varying degrees of moment release. The models are developed in OpenSees [109] with structural members formed by a combination of elastic and fibre based buckling elements accounting for material and geometric non-linearities. Masses are concentrated at each floor level with torsional components equivalent to an accidental eccentricity of 5% of the building width. A live load of 3.00 kN/m^2 is employed. One directional zero-length elements with Hysteretic Material models are used to represent the pinching and hardening characteristics of semi-rigid connections.

Table 8.1 summarizes the main structural characteristics of the 7 groups of 3D idealizations considered. A total of 28 models were developed with four fundamental periods ($T_1 = 0.2, 0.4, 0.6$ and 0.8 seconds) for each of the 7 structural configurations described in Table 8.1. Three different levels of secondary frame contribution are studied in the case of one-way systems with primary MR frames (0, 10 and 15% contribution in terms of initial stiffness). These different levels of secondary frame contribution are

8. FRAMING SYSTEMS

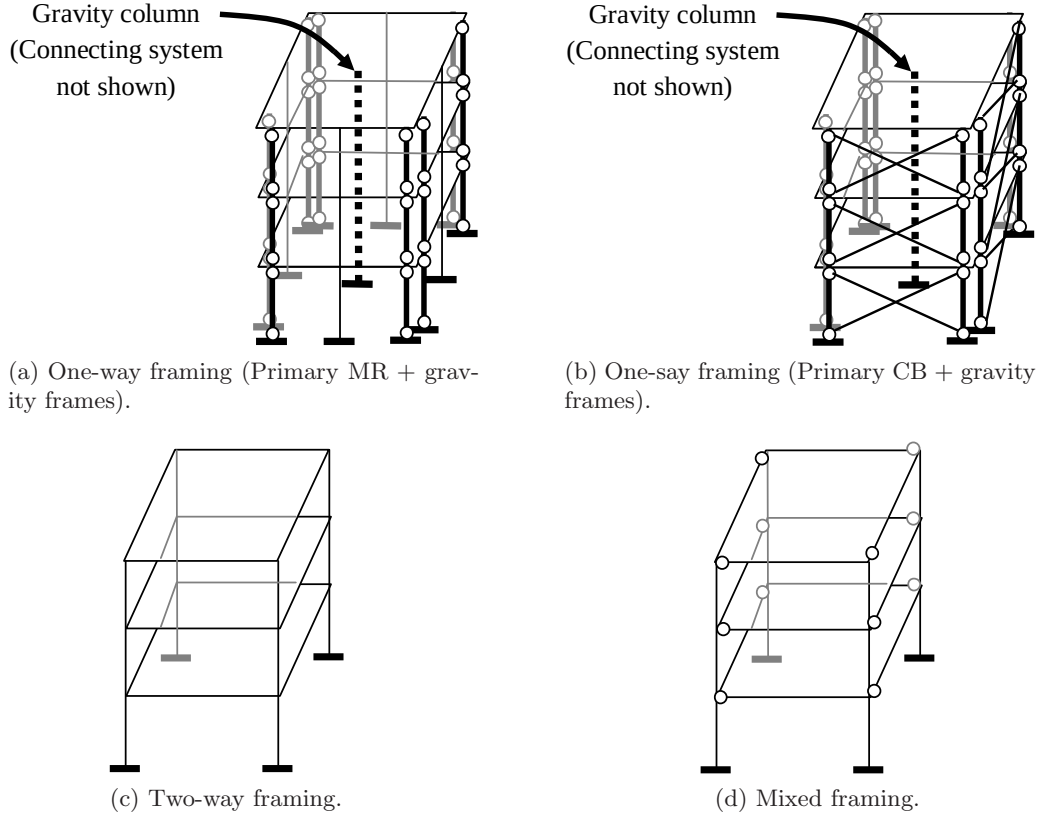


Figure 8.4: Simplified 3D models.

achieved by modifying the connection details in the gravity frame from *web cleats* to *top, seat and web angle* connections and, in the case of MR-15, by slightly varying the dimensions of the gravity columns so as to obtain a 1.25 ratio between the plastic capacity of the column and that of the connection. In the case of one-way buildings with CB frames, it was not possible to increase the secondary frame contribution above 10% without significant enlargement of the gravity column dimensions and thus only a 10% contribution level is considered herein. It is worth noting that the capacities of the adopted semi-rigid angle connections represent about 50% of the plastic beam capacities (M_{pb}), which are typically achievable by means of the details studied in Chapters 3 to 6.

8.3.2 Building response under static loads

Figures 8.5 to 8.7 present the pushover curves (in terms of normalized total lateral force or base shear F/F_y to normalized roof displacement δ/δ_y) for the simplified three-storey 3D models described previously. A constant first-mode lateral force distribution along

Table 8.1: Characteristics of the simplified 3D models.

Model	Framing System	Primary System	Secondary System Contribution	Gravity frame / connection detail
MR-00	One-way	MR frame	Ignored	Web-cleat (Nominally pinned)
MR-10	One-way	MR frame	10 % of the initial stiffness of the primary	Top, seat and web angle (Semi-rigid 50% of M_{pb})
MR-15	One-way	MR frame	15 % of the initial stiffness of the primary system	Top, seat and web angle (Semi-rigid 50% of M_{pb}). Capacity design of column to connection
CB-00	One-way	CB frame	Ignored	Web-cleat (Nominally pinned)
CB-10	One-way	CB frame	10 % of the initial stiffness of the primary system	Top, seat and web angle (Semi-rigid 50% of M_{pb}). Column to connection capacity design
TW	Two-way	N/A	N/A	
MF	Mixed	N/A	N/A	Top, seat and web angle (Semi-rigid 50% of M_{pb})

the height of the building was employed during the non-linear static analyses. The drift concentration factor (DCF), defined here as the ratio of the first storey drift to the roof drift is also presented in Figures 8.5 to 8.7. It is evident from Figure 8.5a that in the case of one-way buildings using MR as primary lateral resisting systems, an increment in secondary frame contribution (from 0 to 15% in terms of elastic stiffness) leads to proportional gains in building over-strength (from 30% to 50% approximately). This increase in over-strength is attributed to the delay of the onset of P-delta instability gained by the inclusion of stiffer gravity frames, with a negative stiffness appearing at $\delta/\delta_y \approx 2$ for MR-00 and at $\delta/\delta_y \approx 3$ for MR-15. Nevertheless, only minor differences on the DCF can be noted (Figure 8.5b).

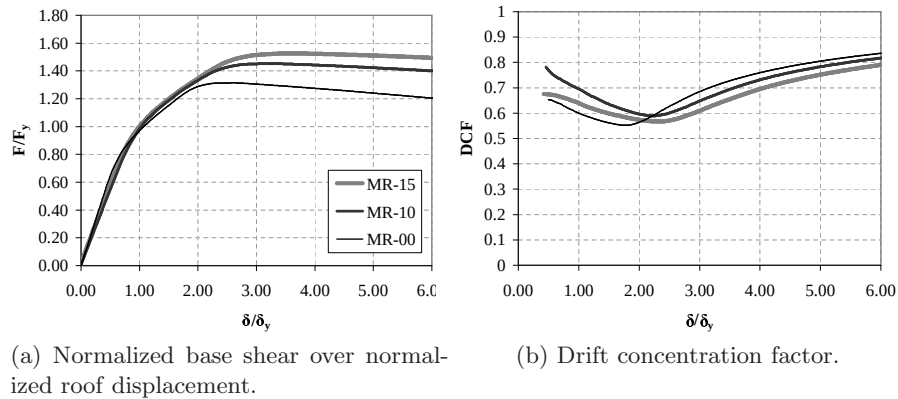


Figure 8.5: Pushover and drift concentration curves for the three-storey one-way framing system with MR frames.

8. FRAMING SYSTEMS

In contrast, the stiffer response of CB systems means that, unlike MR frame for which the design is usually governed by drift limitations, and which exhibit significant over-strength ratios [137], CB systems have yield lateral capacities closer to the ultimate base shear. Furthermore, the lateral capacities of CB buildings can quickly deteriorate upon buckling of the braces if large enough P-delta effects are present. Also, an increment of 10% in gravity frame contribution causes a directly proportional increment in the maximum base shear of CBF one-way buildings. Besides, the stiffer gravity frames significantly reduce the first storey drift concentrations as can be observed from the evolution of DCF with normalized roof drift (Figure 8.6b).

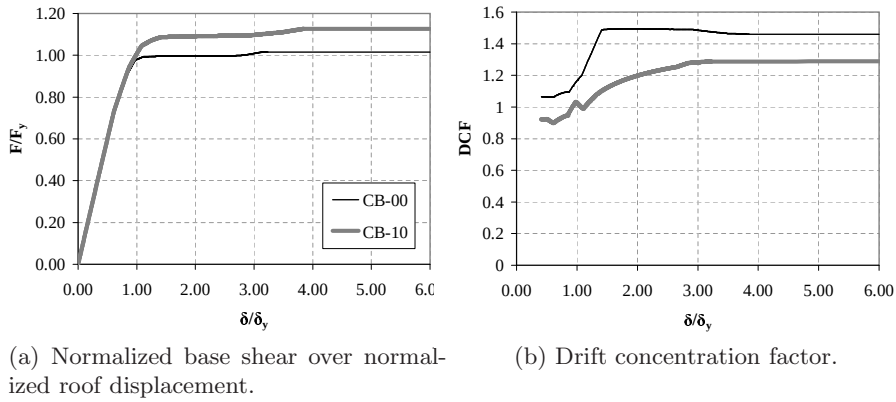


Figure 8.6: Pushover and drift concentration curves for the three-storey one-way framing system with CB frames.

Finally, Figure 8.7 compares the non-linear static response of two-way (TW) and mixed frame (MF) models. As indicated in Figure 8.7a the introduction of moment releases at given column locations increases the over-strength of the building from below 40% in the TW to over 60% in the MF model. The MF model also presents consistently lower drift concentrations in the first storey when compared with the equivalent TW building (Figure 8.7b), in particular for large drift demands. These benefits can be attributed to: (i) the limited transmission of moments to certain columns that would enable them to act as equivalent continuous elements, thus aiding in the redistribution of plasticity along the height, and (ii) the capping of bi-axial moment action imposed on bottom storey columns.

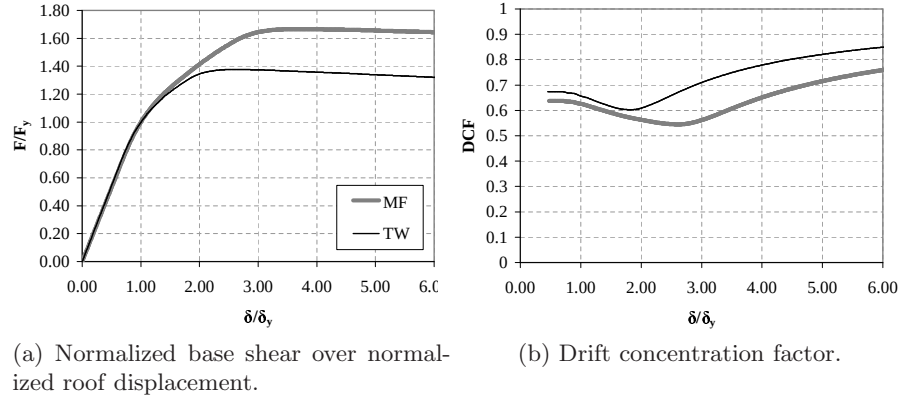


Figure 8.7: Pushover and drift concentration curves for the three-storey two-way and mixed framing systems.

8.3.3 Building response under dynamic loads

Non-linear response history analyses were performed using the sub-set of 30 records on Soil Types A and B from the suite previously described in Section 6.2.3. Issues related to the selection and scaling of ground-motions for bi-directional loading are beyond the scope of the present thesis and represent a subject matter of current debate. Therefore, the recommendations of Beyer and Bommer [16] are considered as guidelines, and the acceleration histories are randomly rotated so as to obtain average response values. The records are scaled according to their relative intensities defined as $[GMS_a(T_1)/g]/\Gamma$, where Γ is the base shear coefficient (ratio between the base shear at yield and the weight of the structure), g is the acceleration of gravity, and $GMS_a(T_1)$ is the 5% geometric mean spectral acceleration at the initial period of the structure [16].

Figures 8.8, 8.10 and 8.12 present average maximum storey ductility demands, μ , over different periods and relative intensities for one-way MR, one-way CB and two-way building systems, respectively. The maximum storey ductility is defined as the maximum of the ratios of storey drift to the storey yield drift over the full height of the building. In turn, the storey yield drift is defined as the drift at which the first storey yielding is observed under a first-mode lateral load distribution during pushover analysis. Graphs 8.8, 8.10 and 8.12 can be interpreted as MDOF strength-ductility(μ) relationships [110]. On the other hand, Figures 8.9, 8.11 and 8.13 depict Incremental Dynamic Analysis (IDA) curves and the associated Coefficients of Variation (COV) as a function of normalized maximum storey drift, $\Theta_{s,max}/\Theta_{s,y}$, for given structural periods. $\Theta_{s,max}$ is the maximum storey drift along the height of the building for any

8. FRAMING SYSTEMS

column and $\Theta_{s,y}$ is the respective yield drift.

It can be observed from Figure 8.8 that for first-mode driven one-way MR buildings, the equal displacement rule provides a reasonable approximation, except for longer periods and greater intensities (i.e. $T \geq 0.6$ and $[GMS_a(T_1)/g]/\Gamma \geq 4$). Deviations from the equal-displacement rule are expected to arise for long period structures due to higher mode and P-delta effects. It is worth noting that the 15% increase in secondary frame contribution does not seem to have noticeable influence on the average ductility demands for MR buildings (Figure 8.8). This is further illustrated in Figure 8.9 in which virtually equal IDA responses are obtained regardless of the secondary frame contribution, with only minor differences for very large drift demand levels (i.e. $\Theta_{s,max}/\Theta_{s,y} \geq 6$). Similarly, the COV remains low irrespective of the gravity frame stiffness (Figure 8.9b).

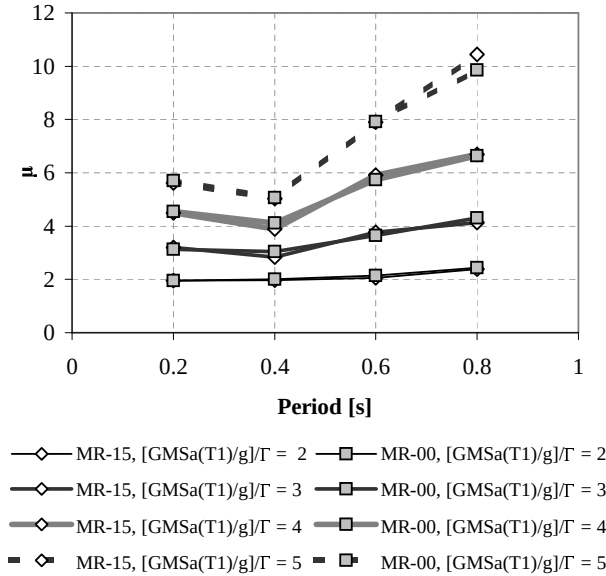


Figure 8.8: Ductility versus strength demand curves. One-way framing system with MR frames.

In the case of one-way CB buildings, it is evident from Figure 8.10 that the $[GMS_a(T_1)/g]/\Gamma = \mu$ relationship is restricted to $T \geq 0.4$ and $[GMS_a(T_1)/g]/\Gamma \leq 2.5$ while increased average ductility demands are observed for shorter periods, notably $T = 0.2$ seconds. This limited range of applicability of the equal displacement rule in CBF buildings may be attributed to the early onset of dynamic instabilities upon buckling of the first-storey braces. This can be further studied with reference to Figure 8.11 where the IDA curves

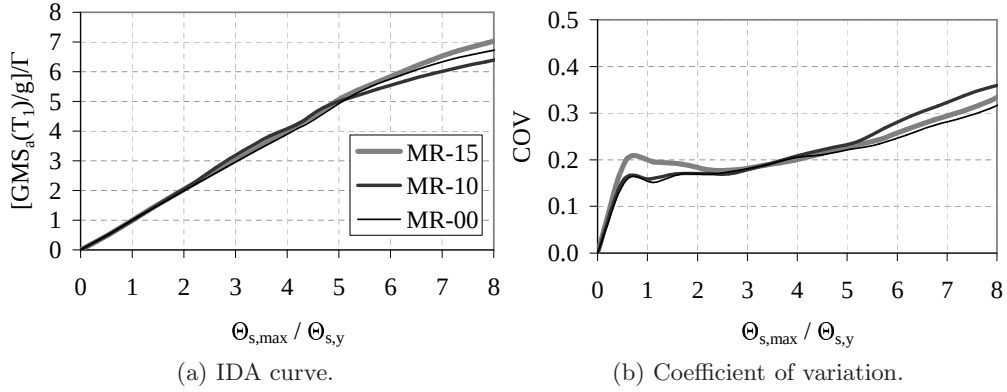


Figure 8.9: IDA curve and COV for one-way framing systems with MR frames. $T=0.4$ seconds.

for CB-00 and CB-10 models with $T = 0.2$ seconds are presented. It can be observed in Figure 8.11 that once a ground motion intensity of approximately 1.6 times that required to yield the structure is reached, the lateral displacements in model CB-00 becomes disproportionally large. An analogous behaviour is observed for the CB-10 model, although at an increased ground motion intensity (i.e. $[GMS_a(T_1)/g]/\Gamma \approx 1.9$) due to the enhanced stiffness of the gravity frames. Furthermore, the advantage of stiffer gravity frames is also reflected in the reduction of the variability of the structural response as depicted in the low COV values of Figure 8.11b. It is worth noting that estimations made with the CB-10 model exhibit substantially lower dispersion levels than those associated with the CB-00 model. In turn, the variability in drift estimations obtained by means of the CB-00 model are shown to increase drastically once non-linear behaviour is attained (at $\Theta_{s,max}/\Theta_{s,y} \geq 1$ in Figure 8.11b).

Figure 8.12 presents the results of average ductility demands for MF and TW models. Again, the equal displacement rule seems to provide a conservative estimate of ductility demands along the period range of study. As expected, notable reductions in peak drifts can be achieved by introducing semi-rigid connections at given frame locations. These drift reductions seem to be more pronounced for higher ground-motion intensity levels. This is more clearly observed by comparing the IDA curves of the TW and MF models with $T = 0.4$ seconds presented in Figure 8.13a. Smaller levels of normalized drifts are evident in the MF model than in the TW for relative intensities of $[GMS_a(T_1)/g]/\Gamma \geq 2.5$. However, the replacement of some fully-rigid connections by semi-rigid details also seem to increase the variability in the response estimation although relatively low values of COV are still present (Figure 8.13b).

8. FRAMING SYSTEMS

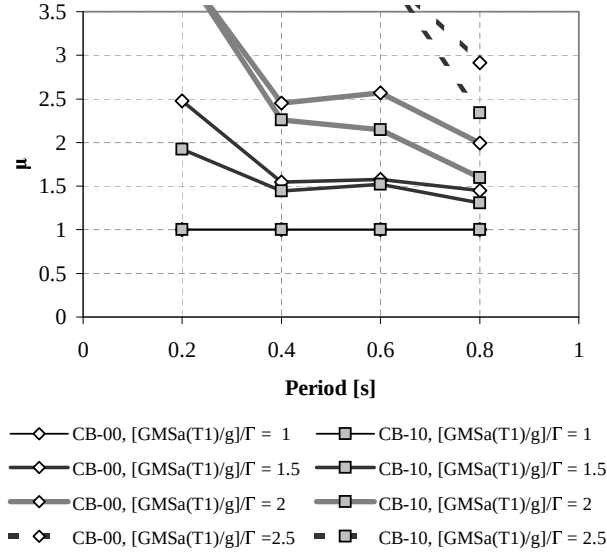


Figure 8.10: Ductility versus strength demand curves. One-way framing system with CB frames.

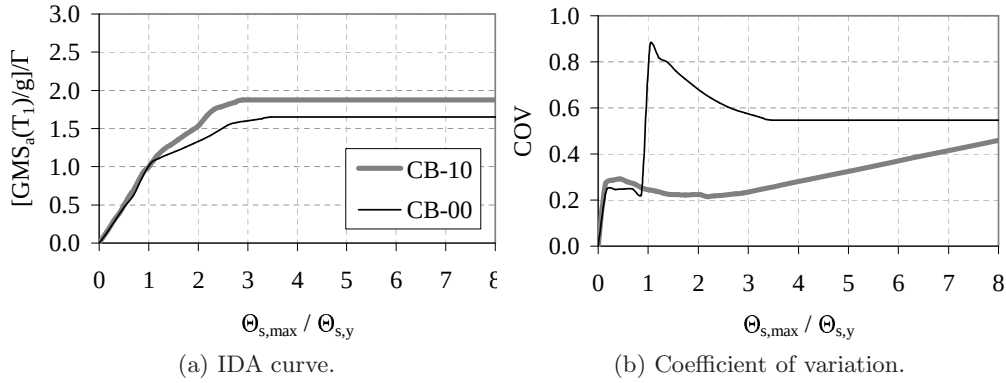


Figure 8.11: IDA curve and COV for one-way framing systems with CB frame. $T=0.2$ seconds.

8.4 Contribution of secondary systems to the seismic performance of CB buildings in near-collapse stages

8.4.1 Generalities and design approach

The previous section has highlighted the significant improvements in seismic response brought about by an adequate consideration of the primary/secondary frame interaction. This response enhancements were more evident for mixed framing systems starting at low and moderate displacement demand ranges and also for CB structures in the near-collapse stage. On the other hand, P-delta and higher mode effects in MR buildings seem to counter-balance the benefits of an adequate consideration of secondary

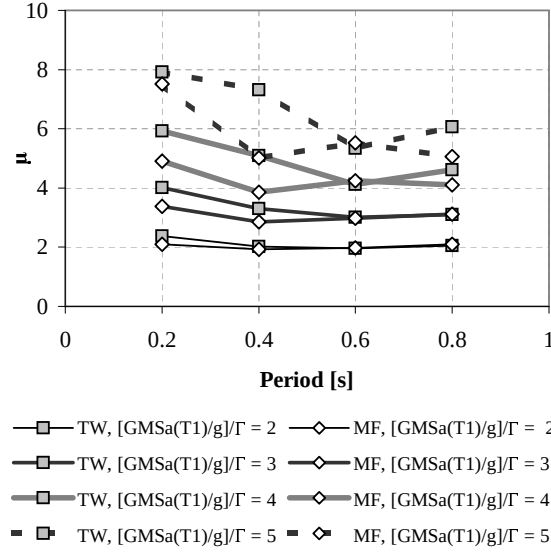


Figure 8.12: Ductility versus strength demand curves. Two-way and mixed framing systems.

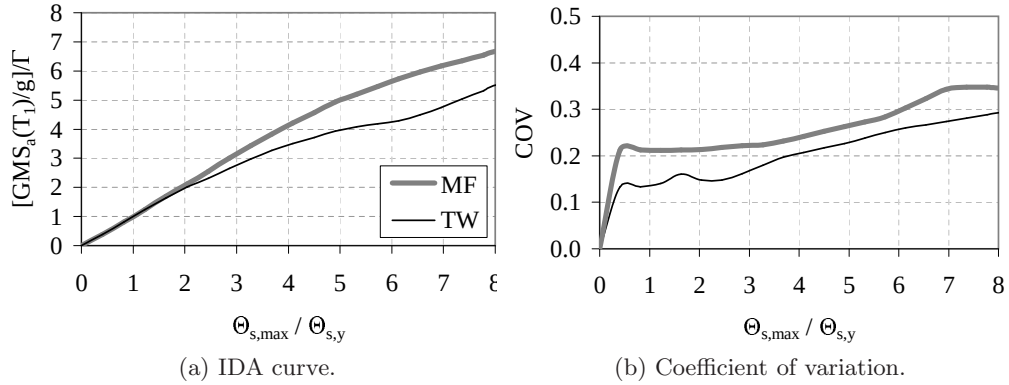


Figure 8.13: IDA curve and COV for two-way and mixed framing systems. $T=0.4$ seconds.

frames and no significant improvements were observed. Nevertheless, an accurate evaluation of the benefits of primary/secondary frame interaction in the near-collapse stage necessitates the consideration of several sources of structural deterioration [73] and their representation through refined non-linear modelling techniques that were not part of the assessment covered in the previous section. It is therefore the aim of this section to quantify the influence of secondary lateral resisting systems on the collapse risk and seismic loss mitigation of steel CB buildings well into the non-linear range. Due to symmetry assumptions, the response of one-way CB frame in the near-collapse range is herein examined in two dimensions, for which adequate information on structural degradation is already available [95]. Mixed framing configurations that also exhibited

8. FRAMING SYSTEMS

response improvements are not considered in this section owing to the lack of experimental information on the effects of bi-axial deterioration and associated complexities that are beyond the scope of the present thesis.

Figure 8.14 depicts a general idealized force-displacement relationship of a primary CB frame acting in parallel with a secondary Partially-Restrained Moment Resisting Frame (PR-MR). Assuming the presence of diaphragm action through the floor slab, both systems are considered to displace laterally by the same amount and hence their simplified force-displacement responses can be combined. In the figure, K_P , K_S and K_T are the stiffness of the primary, secondary and overall structural systems, respectively; h_P and h_T are the post-elastic hardening coefficients of the primary system and overall structure, respectively; and θ is the stability factor representing the ratio between second and first order action on the structure (which quantifies the reduction in lateral stiffness due to gravity loads).

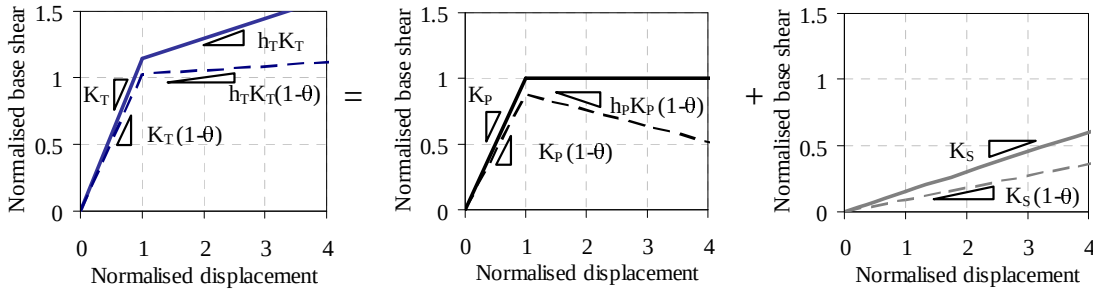


Figure 8.14: Force-displacement relationships for the overall structure (left), primary (middle) and secondary (right) systems normalized to yield of primary system.

The main objective in designing the stiffness contribution of secondary frames becomes apparent from Figure 8.14. Owing to its relatively large elastic deformation (compared with the buckling deformation of CB frames), semi-rigid secondary systems can be used to modify the post-yield branch of the overall structural response. It has been shown that the post-elastic stiffness is directly related to the onset of dynamic instability [115]. In this respect, assuming that K_P and h_P are known or can be estimated, and given a target post-elastic hardening ratio h_T , the secondary frame stiffness can be determined from:

$$K_S \geq K_P(h_T - h_P)/(1 - h_T) \quad (8.1)$$

Once the target post-elastic stiffness and the primary system stiffness are determined, by means of elastic analysis or simplified expressions such as those proposed by Englekirk [47], the required stiffness from a PR-MR secondary frame (K_S) can be estimated from 8.1. In turn, the PR-MR stiffness can be assumed to be the sum of the stiffness contributions associated with the columns (k_{col}), beams (k_{beam}) and connections (k_θ). Therefore, the required rotational stiffness of the PR-connection, S_θ , can be estimated from:

$$S_\theta = h^2 \frac{l_c}{l} k_\theta = h^2 \frac{l_c}{l} \frac{K_S k_{col} k_{beam}}{k_{beam} k_{col} - (k_{beam} + k_{col}) K_S} \quad (8.2)$$

where l is the bay length, l_c is the distance between the bay mid-span and the column face, and k_{beam} and k_{col} are the stiffnesses of the beam and column - for which closed form solutions can be found elsewhere [47].

8.4.2 Evaluation of design scenarios

In order to identify optimum design scenarios (i.e. target h_T in Figure 8.14) a parametric study is carried out using the simplified SDOF model shown in Figure 8.15. The idealised SDOF consists of pin-connected rigid members forming a three-metre high two-bay CB that incorporates two semi-rigid partially-restrained connections between beams and columns. The structure is modelled in OpenSees [109] which accounts for geometric and material nonlinearities. The fatigue damage model implemented in OpenSees by Uriz and Mahin [149] is used for the braces and a hysteretic model is employed for the partially-restrained connections.

Structural collapse is defined as the state at which a small increment in the ground motion intensity produces a disproportionately high increment in the structural demand parameter (peak roof drift herein). A number of structures with different stability coefficients (Θ) are considered, as well as PR stiffness contributions ranging from 0% to 25% of the initial stiffness of the overall system. The SDOF structure is characterized by its normalized slenderness value ($\bar{\lambda}$) defined as: $\bar{\lambda} = \sqrt{AF_y/N_{cr}}$, where A is the brace area, F_y is its yield strength and N_{cr} the Euler buckling load. In total, more than 25000 analyses were performed. Only the comparisons in terms of ground-motion

8. FRAMING SYSTEMS

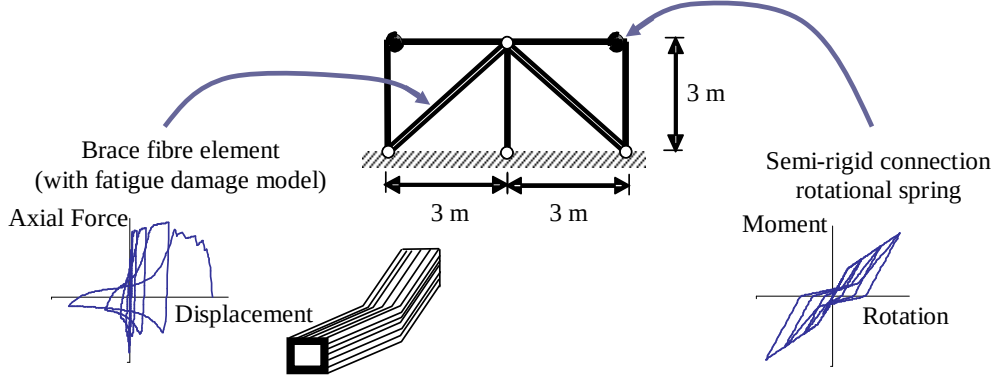


Figure 8.15: SDOF model of primary CB and secondary PR-MR for collapse parametric studies.

intensity (PGA) at collapse are summarised below.

Figure 8.16 present the comparisons of the median PGA values at collapse for the hybrid CB with different PR connections as compared to the original all-pinned CB system. It is apparent from Figure 8.16 that the greater the PR contribution, the higher the ground motion intensity at collapse, irrespective of the brace slenderness. On the other hand, the improvement in collapse capacity is strongly dependent on the stability coefficient Θ , with higher values of PR contributions needed for systems exhibiting larger stability coefficients in order to achieve comparable collapse capacities. In general, it appears that up to a three-fold increase in collapse intensities can be expected if the PR is designed to provide adequate stiffness and satisfy a maximum stability coefficient (e.g. systems with PR stiffness contribution 20% and $\theta \leq 0.2$).

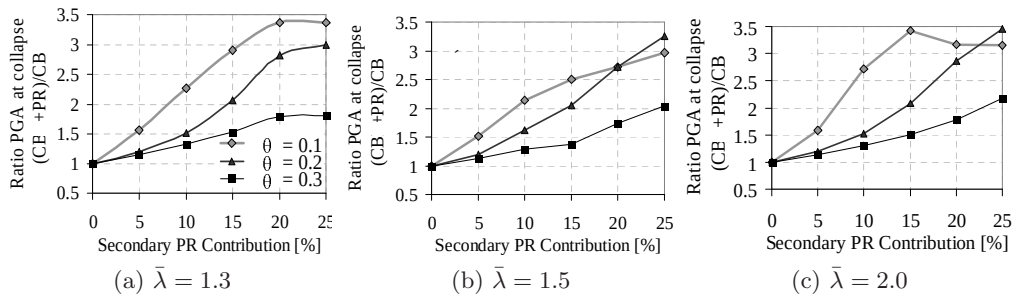


Figure 8.16: Median collapse capacities for hybrid CB + PR-MR SDOF systems.

8.4.3 Case study

Figure 8.17 presents a case study building used to illustrate the influence of secondary lateral resisting systems on the inelastic behaviour of a typical steel frame configuration. The four-bay five-storey building under consideration has external CB frames acting as primary lateral resisting system and internal MR frames representing a secondary system. Only the 2D behaviour is considered herein due to the considerations discussed before. Two designs were performed: (i) firstly, the internal frames were assumed to resist gravity loads only and hence web-cleat (nominally pinned) connections were designed for the internal MR; (ii) a second design was performed following the procedure outlined above with a target post-elastic stiffness for the overall structure of 5% - in this case blind-bolted semi-rigid (top, seat and web angle) connections were designed to achieve the required PR-MR stiffness. Figure 8.17b depicts the push-over response obtained for both designs in terms of base shear (normalized to the effective building weight) against roof drift.

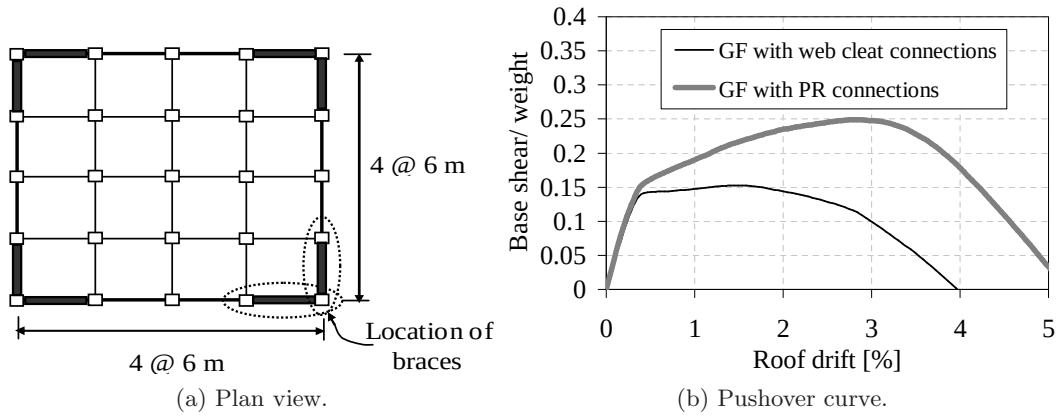


Figure 8.17: One-way CB case study building for collapse stage verification.

The seismic risk assessment performed herein is based on the results of IDA under uniform hazard ground-motions. The suite of 20 records described by Vamvatsikos and Cornell [151], with Richter magnitudes between 6.5 and 6.9 and moderate epicentral distances (16 to 32 km), was used for these purposes. The hazard recurrence relationship, between the intensity measure IM and the annual frequency (f_a) for this set of records, is simplified as:

$$f_a(IM) = a(IM)^{-b} \quad (8.3)$$

8. FRAMING SYSTEMS

where a and b are empirical parameters. The 5%-damped spectral acceleration at the initial period of the structure is considered here as the intensity measure while the peak inter-storey drift is used as the structural demand parameter. The structure is modelled in the non-linear analysis framework OpenSees [109]. The models developed consider a number of advanced features so as to be able to trace accurately the structural behaviour up to collapse stages. The gusset plate effects are modelled by a combination of rigid and fibre elements buckling out of plane. Fatigue damage in bracing members is included by means of cumulative fatigue damage model developed and implemented by Uriz and Mahin [149]. Cyclic degradation effects in the column and beam elements are taken into account by tailoring hysteretic degrading models to empirically calibrated degradation factors [95]. Pinned joint frame models with tri-linear rotational springs are used to describe the panel zone effects as suggested by Krawinkler [84]. Also, the typical pinching and stiffness degradation effects in the blind-bolted connections are considered by means of zero-length elements with hysteretic materials.

Figure 8.18 presents the comparison of collapse analysis results for the building under study. From the IDA curves in Figures 8.18a and 8.18b, it can be observed that an adequate design of the secondary frames improves its median collapse capacity (IM at collapse) by 40 % (from 0.52g to 0.72g). This is further illustrated by comparing the collapse fragility curves presented in Figure 8.18c. A reduction in the annual frequencies of collapse of up to one-third can be observed for this building when adequate secondary PR-MR design is performed in comparison with a gravity-only design (Figure 8.18c). This reduction in the collapse probabilities occurs, at least partially, through a modification of the plastic mechanism at ultimate. As indicated in Figure 8.19, 15% of the soft first-storey collapse mechanism cases are turned into another sideways collapse pattern involving the bottom two storeys.

The original set of median IDA and their respective percentiles were modified to include the effects of uncertainty following the recommendations given by FEMA [55]. In order to estimate direct losses, the probability of reaching or exceeding a certain damage state is represented against the peak roof displacement obtained from the modified IDA, considering four damage states as determined by HAZUS [70]. Figure 8.20 presents the fragility functions for different performance levels obtained for the two CB designs. The financial implications of damage accumulation were then compared by means of the Expected Annual Loss (EAL) considering the integration of losses against all possible earthquake scenarios [103]. In general, it was found that a 22% reduction in the

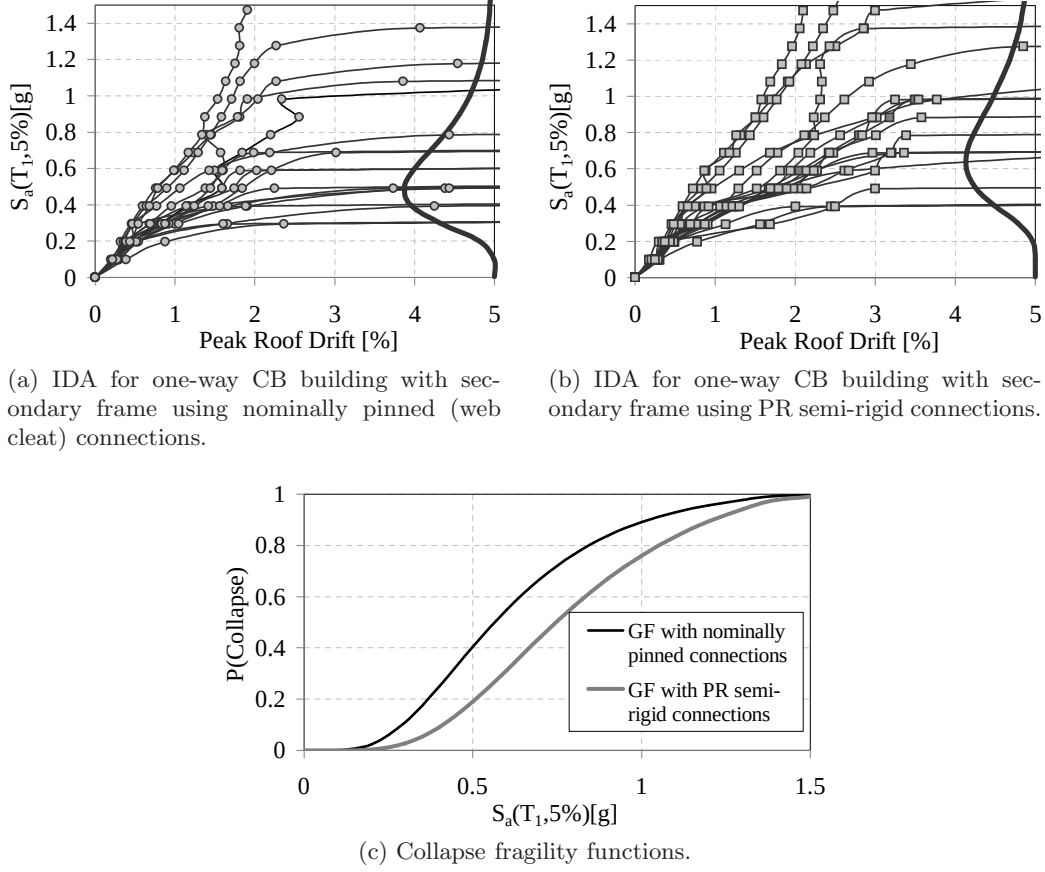


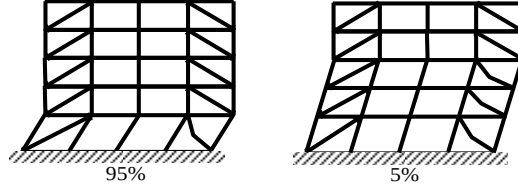
Figure 8.18: Collapse probability assessment of case study building.

estimated EAL would be achieved through a proper consideration and design of the secondary lateral resisting system in this case study building.

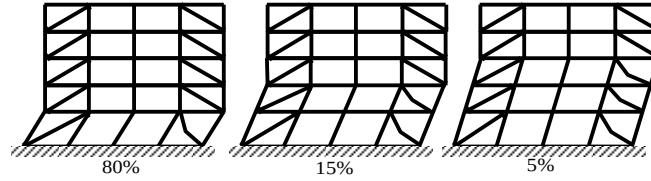
8.5 Concluding remarks

This chapter has studied the peak response of one-way, two-way and mixed framing configurations incorporating moment resisting (MR) and concentrically-braced (CB) lateral resisting systems. Results of peak responses, related to building and content damage performance levels, from idealized 3D models under a suite of earthquake records, have been presented. It was demonstrated that secondary systems, which are normally designed to carry gravity loads only, can play a significant role in reducing both peak inter-storey drifts and the probability of dynamic instability of the overall structure if the primary systems consists of CB frames. On the other hand, no significant improve-

8. FRAMING SYSTEMS

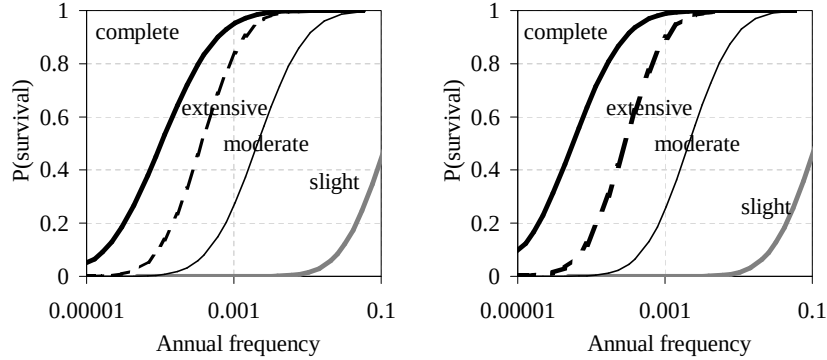


(a) Collapse mechanisms for one-way CB building with secondary frame using nominally pinned (web cleat) connections.



(b) Collapse mechanisms for one-way CB building with secondary frame using PR semi-rigid connections.

Figure 8.19: Collapse mechanisms.



(a) Fragility functions for secondary frame with nominally pinned connections.

(b) Fragility functions for secondary frame with PR semi-rigid connections.

Figure 8.20: One-way CB case study building for collapse stage verification.

ment was observed for buildings with primary MR frames, which is largely attributed to the overriding effects of secondary moments and higher modes. It was also shown that by modifying the two-way configuration through the introduction of variable degrees of moment release at selected beam-to-column connections, significant reduction in the peak seismic displacement demands can be obtained. Additionally, the results validated the applicability of the equal-displacement rule in providing upper-bound estimates for low to moderate relative ground-motion intensities and for the range of periods studied here under bi-directional earthquake action.

The effects of secondary/gravity system interaction at large displacements (near-collapse performance region) have also been studied by means of advanced 2D models incorporating a number of structural degradation sources. It was demonstrated that an appropriate detailing and use of secondary systems in seismic design can lead to considerable benefits in terms of the reduction of seismic losses and collapse probabilities. The results of parametric analyses on SDOF CB models coupled with PR-connections suggest that such beneficial effects can be achieved regardless of the brace slenderness. However, greater PR stiffness contributions would be required for structures with higher second order effects (represented by larger values of the stability coefficient). It was shown in this chapter that such post-yield hardening effects bring notable improvements to the collapse capacity of structures. General approaches and design procedures to achieve this aim have been outlined and demonstrated through a building case study.

Part V

Closure

Chapter 9

Conclusions

9.1 Summary of main findings

The main objective of this thesis, as stated in Section 1.2 is twofold: (i) to contribute to the fundamental understanding of the seismic behaviour of steel structures incorporating tubular members, and (ii) to develop reliable analysis and assessment approaches with a view to improving the design guidance from a local, global and system interaction perspective. To this end, this thesis employed experimental, analytical and numerical methods to investigate a number of issues pertaining to frames with tubular columns and/or tubular braces at three levels of structural discretization, namely: connection, frame and complete framing system interaction. The main findings can be summarized as follows:

- *Experimental behaviour of Blind-bolted angle connections:* Three main inelastic mechanisms were identified in Holo-bolted connections between open beams and tubular columns (Figure 3.8). These inelastic mechanisms primarily originate from the interaction between the angle components and the Holo-bolt/column face assemblage. Also, the grade of the Holo-bolt, coupled with the gauge distance between the Holo-bolt and the beam flange, was shown to have the most notable effect on the connection performance. As illustrated in direct tension tests (Figure 3.6), the higher torque permitted in Grade 10.9 Holo-bolts improves significantly its axial stiffness and capacity in comparison with Grade 8.8, albeit with some reduction in local ductility. The tests performed also showed the necessity to limit the extension induced in the Holo-bolts as well as the bending deformations within the column face, in order to satisfy serviceability requirements. Besides,

9. CONCLUSIONS

the presence of an adequate mechanism for transferring the compression forces to the column is important for achieving satisfactory connection performance, especially when web angles are employed due to the higher loading capacity. In terms of ductility, Holo-bolted angle connections can provide rotational capacities in excess of 120 mrad under monotonic loading and more than 60 mrad under cyclic conditions, which are well beyond those required under typical design scenarios. This type of connection can exhibit stable hysteretic response and provide reasonable energy dissipation capabilities despite the notable pinching observed in some cases.

- *Experimental behaviour of combined channel/angle connections:* The inelastic mechanisms exhibited by open beam-to-tubular column angle connections with channel components were identified (Figure 4.2). These inelastic deformation patterns stem directly from the interaction between the different connection components. To this end, the angle gauge distance is inversely proportional to the connection capacity with less significant effect on the stiffness. Conversely, the flexibility of the reverse channel component has a direct influence on both the initial stiffness and capacity of the connection. Besides, the addition of web angles can significantly enhance the overall connection resistance. As with blind-bolted connections, the importance of ensuring an adequate mechanism for transferring the compression forces to the channel component was highlighted. In turn, reverse channel connections can reach rotational levels exceeding 120 mrad under monotonic loading and well over 50 mrad under cyclic loading with stable hysteretic response. Although the pinching response increases with the increase in capacity, reverse channel connections can offer reasonably good energy dissipation.
- *Mechanical models for open beam-to-tubular column angle connections:* Mechanical models that use the component approach, coupled with multi-linear representations of all possible connection behavioural stages, are able to provide reliable predictions of the response of Holo-bolted as well as combined channel/angle configurations. In particular, the initial stiffness and capacity of such connections can be predicted within an accuracy of 5% in all cases. From the parametric analysis performed on Holo-bolted angle connections, it can be concluded that the Holo-bolt grade, angle thickness, column face flexibility and gauge distance can have a significant effect on the stiffness and capacity of the connection. It can

also be concluded that the improved initial axial stiffness of Grade 10.9 Holo-bolts enable a more consistent definition of the failure mode and can enhance the reliability of blind-bolted connections.

- *Inelastic demands on steel structures:* Key differences exist between the inelastic deformation demands of bi-linear, PR and CB structures, particularly in the relatively short period spectral region. The main factor influencing the differences in average maximum displacements between bi-linear and PR system is the ratio between the overall yield strength and the strength during pinching intervals. The displacement demands on PR structures can double the peak displacements expected in bi-linear system models for relatively stiff structures. Also, the seismic response of CB and PR models was observed to follow different tendencies despite their similar cyclic force-displacement relationships. The rate of displacement amplifications decreases as the strength ratio increases for CB models when compared against bi-linear predictions. Importantly, the effects of local site conditions on displacement ratios are relatively small for moderately stiff to stiff soils, whereas significant displacement amplifications occur in the case of soft soils relative to stiffer sites. Finally, the predominant period of the ground-motion (T_g) was found to be the most effective parameter in reducing the variability and enhancing the accuracy of peak inelastic displacement estimations in bi-linear, PR and CB structures when compared with other frequency content indicators.
- *Rigid-plastic response history analysis:* Response history analyses based on rigid-plastic idealizations are able to predict, with good levels of accuracy and for well defined conditions, the structural response of MR and CB frames while keeping the computational cost relatively low. In general, rigid-plastic models can provide good estimates of global deformations in steel structures if the ground-motion pushes the structure well into the inelastic range; the ratio between PGA and acceleration at yield serves as a good parameter to quantify this ability. In particular, peak displacements are well predicted for MR frames for PGA/a_y greater than 3. On the other hand, for CB structures, the more slender the braces, the more reliable is the response prediction of rigid-plastic proxies with slip-like behaviour. Nevertheless, when flexural strength (or bracing compression) is incorporated into the rigid-plastic models, the response can be adequately predicted for normalized slenderness values between 1.3 and 2.1.

9. CONCLUSIONS

- *Framing systems:* Secondary systems can play a significant role in reducing both peak inter-storey drifts and the probability of dynamic instability of the overall structure if the primary system consists of CB frames, whereas no significant improvement was observed for buildings with primary MR frames. The introduction of variable degrees of moment release at selected beam-to-column connections (e.g. by means of connection details such as those studied in Chapters 3 to 5) can lead to significant reduction in the peak seismic displacement demands. Such beneficial effects can be achieved regardless of the brace slenderness of CB primary frames, although greater PR stiffness contributions would be required for structures with higher second order effects (represented by larger values of the stability coefficient). Also, the equal-displacement rule can provide upper-bound estimates for low to moderate relative ground-motion intensities and for regular structures with medium range periods.

9.2 Design and assessment contributions

While working towards achieving the objectives of this thesis, new and existing design and assessment tools were developed or improved. The most notable of these contributions can be summarized as follows:

- *At the connection level:* The results of this thesis highlight the suitability of semi-rigid connections between open beams and tubular columns (Hollo-bolted and reverse channel connections) for secondary or braced primary frames depending on the level of loads under consideration. To this end, expressions for the estimation of the main connection response parameters have been proposed. In the case of Hollo-bolted angle connections, based on the dataset generated through parametric analyses by means of the detailed mechanical model, semi-analytical design-oriented expressions were proposed for the evaluation of stiffness and capacity (Section 5.5). With regards to combined channel/angle connection configurations, the force-displacement response of the reverse channel component was characterized (Equations 5.18) and simplified models for the estimation of connection stiffness and capacity were adapted from the equivalent T-stub approach of Eurocode 3 (Equations 5.31 and 5.32). These expressions, in conjunction with appropriate gauge distances (Equations 5.3 and 5.4), were found to produce reliable estimates of reverse channel connection stiffness and capacity. Additionally,

the applicability of fatigue damage models based on the effective plastic rotation of the connection was validated, with the energy-based model proposed by Mander et al. [105] showing the most favourable comparisons with tests results.

- *At the frame level:* Equivalent linearization expressions were proposed for evaluating the seismic performance of structures with known strength (Equation 6.9 to 6.12). These equivalent linear models were expressed as a function of secant period and stiffness leading to maximum inelastic displacement and acceleration occurring at the intersection of the capacity and demand diagrams, hence providing a direct graphical comparison tool. Also, the ability of rigid-plastic models to assess the global response of MR, CB and dual frames was established. The potential for assigning structural capacity based on rigid-plastic response history analyses has also been illustrated. Furthermore, a framework incorporating the time-dependent nature of earthquake loading into a dual level performance based design has been proposed (Figure 7.16). Such proposed framework would allow reliability levels to be estimated with due account of the real earthquake variability. In addition, it would enable different levels of structural optimization without the computational demand and inherent complexity associated with non-linear finite element response history analysis.
- *At the overall building level:* It was shown that the secondary frame contribution to collapse and seismic loss mitigation operates through a modification of the post-yield stiffness at the overall building level. General approaches to attain optimal levels of secondary frame contribution have been outlined and demonstrated through a building case study. These approaches include the parametric exploration of PR gravity frame target stiffness (Figure 8.16) and associated expressions for estimating the corresponding connection stiffness (Equation 8.2).

9.3 Limitations and future work

The main findings and contributions described in the previous sections address the two aspects of the primary objective of this work. The studies carried out as part of this thesis have also highlighted the need for further research in the following areas:

9. CONCLUSIONS

The results of direct tension tests on Holo-bolts reported in Chapter 3 are in agreement with axial force-displacement curves obtained from the manufacturer. This, in conjunction with the close predictions achieved over a range of connection tests by using the obtained values (as described in Chapter 5) would signify that such force-displacement relationships can be considered as good representations of the Holo-bolt axial behaviour. Nevertheless, additional tests, aimed specifically at assessing the reliability of the Holo-bolt insert against a statistically significant sample size ought to be conducted. Such tests should evaluate the Holo-bolt performance within a comprehensive probabilistic framework, including expected variations in pre-stressing loads and material properties suitable for implementation within load and resistance factor design.

The characterizations proposed in Chapter 5 for the column face and reverse channel components provided good estimates of stiffness and capacity for the range of channel dimensions and column face slenderness studied here. However, further validation and/or calibration of such expressions against numerical and experimental results including direct tension tests on the individual components (as opposed to backward calibration from moment-rotation curves) should be carried out.

The tests described in Chapters 3 and 4 have illustrated the significant ductility capacity of blind-bolted and combined channel/angle connections under cyclic loading and confirmed the applicability of energy-based fatigue damage models. However, the step-wise increasing cyclic protocols used, although serving the purposes of standardization of results, represent an over-simplification of real earthquake waveforms and their application may be questionable for near-field seismic scenarios. Accordingly, further validation of the results should be considered under other static loading protocols and/or pseudo-dynamic demand histories.

The refined multi-linear mechanical model developed in Chapter 5 was shown to predict accurately the inelastic moment-rotation response of semi-rigid open beam-to-tubular column connections. These developments provide a good basis for the elaboration of angle connection models focusing on other forms of extreme loading conditions. In particular, the algorithm proposed, as it stands, utilizes user-defined values of axial forces to perform its equilibrium calculations. These features can be used in future assessments of connection response to accidental or malicious actions.

The detailed investigations on inelastic displacement demands described in Chapter 6,

as well as the proposed linearized models suggested, are based on structural systems with known capacity subjected to constant strength demand scenarios. Therefore, the results are suitable for the evaluation of existing structures or design verification stages in which capacity estimations are readily available. Accordingly, complementary studies are needed for structures under target ductility scenarios for which the influence of different frequency content indicators should be considered. Furthermore, investigations on structural response parameters other than peak displacement (e.g. fatigue damage, hysteretic energy and Park and Ang damage indexes) need to be performed.

The equivalent linear systems proposed in Chapter 6 appear to be reliable for the assessment of existing steel structures with $T > 0.20$ s and for moderate to stiff soil conditions. Nevertheless, further validation studies are needed to cover a wider range of structural configurations, as well as to propose equivalent linear models specifically developed for structures on soft soils. The development of ductility-dependent equivalent linear models for CB structures should also be pursued.

The rigid-plastic idealizations presented in Chapter 7 share the same limitations of equivalent SDOF models; namely, they are only applicable for regular first-mode dominated structures where a pre-defined collapse mechanism is enforced. In addition the rigid-plastic model does not account for strength degradation. In this respect, future studies could take advantage of the consideration of the complete time history that is offered by rigid-plastic models and seek to incorporate a cycle counting algorithm within the integration procedure in order to represent some forms of strength degradation. Furthermore, the fatigue damage estimations presented in Appendix C should be further extended to other structural forms by incorporating local folding mechanisms. This potential use of rigid-plastic response history analyses needs to be verified for a larger set of structural configurations and earthquake records.

The feasibility of design methodologies based on rigid-plastic relationships between plastic capacity and demand parameters has been highlighted. Nevertheless, their range of applicability needs to be examined further in future studies covering other structural configurations and acceleration records. Importantly, the displacement-plastic resistance curve (expressed in terms of the median value) does not imply that such required strength is associated with the median probability as the ability of the rigid-plastic oscillator varies as a function of the plastic resistance itself. Consequently adequate reliability frameworks need to be studied and/or tailored to rigid-plastic estimations.

9. CONCLUSIONS

The idealized 3D models employed in the investigations presented in Chapter 8 offer an efficient way of studying the response of different structural configurations under a greater set of earthquake records. On the other hand, the simplifications involved introduce some limitations (i.e. building regularity is assumed and structural redundancy is not considered) despite which general behavioural trends could be established for the three-storey buildings examined. These results need to be extended for buildings with different numbers of storeys. Similarly, the effects on the dispersion of results obtained by expressing them in terms of normalized period (i.e. by introducing a ground-motion frequency content indicator such as T_g) should be evaluated.

It was demonstrated that secondary systems can reduce peak inter-storey drifts and mitigate the probability of dynamic instability if the primary systems consist of CB frames, whereas no significant improvement was observed for buildings with primary MR frames. It was also shown that by modifying the two-way configuration through the introduction of variable degrees of moment release at selected beam-to-column connections, significant reduction in the peak seismic displacement demands can be obtained. Experimental verification of these findings needs to be pursued in the form of dynamic or hybrid simulation tests. Importantly, experimental quantification of the near-collapse bi-axial response of column/connection/beam assemblages is needed in order to adequately model the ultimate behaviour of two-way and mixed framing systems.

Finally, the effects of secondary/gravity system interaction at large displacements have been studied by means of advanced 2D degrading models of CB one-way buildings. Parametric analysis on one-storey structures and a case study on a five-storey building were carried out. It was shown that by introducing post-yield hardening effects, an adequate design of secondary frames can bring notable improvements to the collapse capacity of structures. The validity of this conclusion clearly needs to be examined further in future studies covering an extensive range of building models and within a more comprehensive probabilistic context.

Appendixes

Appendix A

Matlab Implementation of the Mechanical Model

The mechanical model presented in Section 5.2 employs a multi-linear approach in order to trace the full monotonic response of bolted angle connections. This Appendix provides additional information on the model algorithms and stiffness calculation process.

Bolt-row stiffness calculation

As explained in Section 5.2.9, the complete bolt-row force-displacement relationship can be obtained by finding the lowest energy path from the decision trees of possible behavioural stages depicted in Figures 5.4 and 5.5. To this end, an adaptation of the solution algorithm outlined by Swanson and Leon [145] for T-stub components was implemented in Matlab [108]. In order to proceed with the calculations, besides the values of the limiting force increments ΔF_i (i.e. yielding or full plasticity at the critical sections, change of stiffness values in the multi-linear relationship of the Hollo-bolt as given in Table 5.1, or reduction of the prying force Q_c to zero), the model also requires the current stiffness values for each of the behavioural stages depicted in Figure 5.4. Expressions for the calculation of such stiffness values are presented in Table A.1, as obtained by means of the stiffness method.

APPENDIX A

Table A.1: Stiffness at different deformation stages.

$$\begin{aligned}
k_{Iee} &= \left(k_A + \frac{1}{k_t} + \frac{1}{k_{slip}} \right)^{-1} & k_{Iep} &= \left(k_B + \frac{1}{k_t} + \frac{1}{k_{slip}} \right)^{-1} \\
k_{Ipe} &= \left(k_C + \frac{1}{k_t} + \frac{1}{k_{slip}} \right)^{-1} & k_{Iep} &= \left(k_D + \frac{1}{k_t} + \frac{1}{k_{slip}} \right)^{-1} \\
k_{Iey} &= \left(\frac{k_{Iee} + 3k_{Iep}}{4} \right) & k_{Iye} &= \left(\frac{k_{Iee} + 3k_{Ipe}}{4} \right) \\
k_{Iyy} &= \left(\frac{k_{Iee} + 3k_{Ipp}}{4} \right) & k_{Iyp} &= \left(\frac{k_{Iep} + 3k_{Ipp}}{4} \right) \\
&& k_{Ipy} &= \left(\frac{k_{Ipe} + 3k_{Ipp}}{4} \right) \\
k_A &= \frac{12EI(d'^3 + 3d'^2c' + 3d'c'^2 + c'^3) + k_\alpha(3c'^2d'^4 + 4c'^3d'^3)}{12EI(3EI + k_\alpha(c'^3 + 3c'^2d'))} \\
&& k_B &= \frac{12EI[k_{sh}[(d' + c')^3 + k_\alpha(c'^3d'^2 + c'^2d'^3) + 3EI(d'^2 + 2c'd' + c'^2)] + k_\alpha k_{sh}(3c'^2d'^4 + 4c'^3d'^3)]}{12EI(3EI k_\alpha c'^2 + k_{sh}) + k_\alpha k_{sh}(c'^3 + 3c'^2d')} \\
&& k_C &= \frac{12EI[k_{sh}[(d' + c')^3 + k_\alpha(c'^3d'^2 + c'^2d'^3) + 3EI(d'^2 + 2c'd' + c'^2)] + k_\alpha k_{sh}(3c'^2d'^4 + 4c'^3d'^3)]}{12EI[k_\alpha k_{sh}(c'^3 + 3c'^2d') + 3EI(d'^2 + 2c'd' + c'^2)]} \\
&& k_D &= \frac{k_{sh}(d'^2 + c'd' + c'^2) + c'k_{sh} + k_\alpha c'^2d'^2}{k_{sh}^2 + 2k_\alpha k_{sh}} \\
&& k_\alpha &= \left(\frac{1}{k_{cf}} + \frac{1}{k_{Hb}} \right)^{-1}
\end{aligned}$$

Once the non-linear characterisation of the horizontal force-displacement relationship for a single bolt-row is obtained by means of the incremental mechanical model described previously, the hysteretic rule of Figure 5.7 can be adopted. Subsequently, each bolt-row can be assigned with a full nonlinear force-displacement hysteresis and the multi-spring joint model considered in Figure 5.1b can be formed. The algorithm presented in Figure A.1 is employed to enforce compatibility and equilibrium at the overall joint level. With reference to Figure A.1; θ and v are the rotation and displacement at a given reference point within the connection, F_{net} (right or left) are the net internal axial forces, F_{Total} is the net external axial force applied to the joint (zero for pure bending situations) and θ_{limit} is a user defined rotation limit.

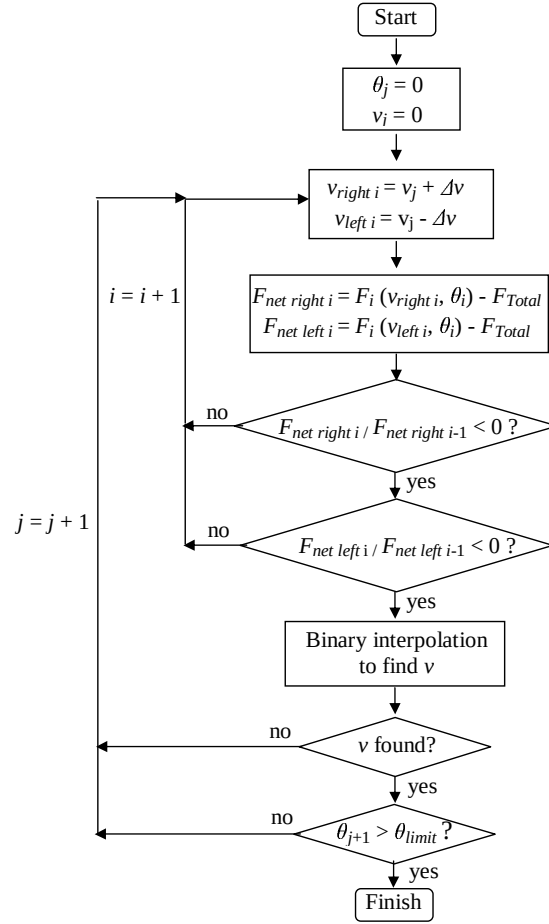


Figure A.1: Iterative algorithm for component assemblage.

Matlab implementation

The entire Matlab code is not included here, but a graphical user interface (GUI) has been developed to provide a user-friendly input/output facility, allowing the parametric analyses of Chapter 5 to be performed in an efficient manner. Figure A.2 presents a screen-shot of the GUI. The left-hand side section of the GUI allows the input of vertical angle leg related parameters whilst the upper middle part does the same for horizontal leg parameters. The middle bottom part of the GUI depicted in Figure A.2 allows for the bolt-row force-displacement data series to be generated and stored in user defined files. Those files are then referenced within the right-hand side column of the

APPENDIX A

MECHANICAL MODEL OF BLIND BOLTED OPEN-BEAM-TO-TUBULAR-COLUMN CONNECTIONS
by: Christian Malaga ONLY ACADEMIC PURPOSE e-mail to: c.malaga-chuquitype07@imperial.ac.uk

VERTICAL LEG PROPERTIES

- Number of bolts per row: 2
- Bolt diameter: 16
- Width of angle: 100
- Thickness of angle: 8
- Distance angle's corner: 16
- Distance angle's edge: 43
- Bolt class: 10.9
- Tubular column width: 150
- Tubular column thickness: 10
- Column stiffness coefficient: 0.2
- Hole diameter: 18
- Bolt separation: 45

HORIZONTAL LEG PROPERTIES

- Number of bolts per row: 2
- Bolt diameter: 16
- Area of bolt: 201
- Bolt yield stress: 510
- Bolt effective length: 50
- Distance angle's corner (total): 50
- Distance angle's edge: 4
- Bolt to beam's web distance: 17
- Beam thickness: 7

ASSEMBLY OF COMPONENTS

BOLT ROW	FILE NAME	DISTANCE FROM BOTTOM
Bolt row 1 (TOP)	BR1	D1
Bolt row 2	BR2	D2
Bolt row 3	BR3	D3
Bolt row 4	BR4	D4
Bolt row 5	BR5	D5
Bolt row 6	BR6	D6
Bolt row 7 (BOTTOM)	BR7	D7

FORCE DISPLACEMENT ASSEMBLAGE

Initial curvature: phi_in
Maximum curvature: phi_end
Curvature increments: Dphi
Maximum iterations (on v): max_iter
Maximum incremental displ. (v): max_v

STEEL PROPERTIES

- Yield stress: 312
- Poisson's ratio: 0.3
- Elastic modulus: 210000
- Strain hardening: 0.01

Buttons: Get F-d relationship, Save F-d relationship as..., Get M-phi relationship

Figure A.2: Graphical user interface for the mechanical model.

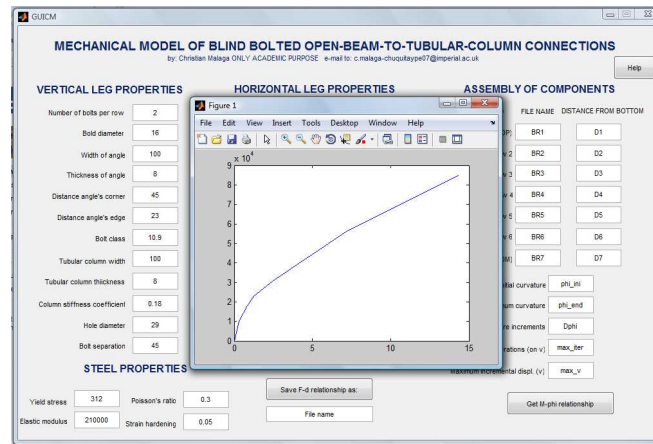


Figure A.3: GUI display of generated moment-rotation response.

GUI together with the corresponding coordinates of each bolt-row forming the joint in order to perform the calculation of the full moment-rotation curve of the connection. Additionally, each time the Matlab code performs a calculation, besides the possibility of filing the data, pop-up windows are displayed with the obtained curves as depicted in Figure A.3

Appendix B

Supplemental Results on Seismic Demands in Steel Structures

This appendix presents supplemental material on the seismic demand analysis on steel structures. It is intended to complement the analysis of results given in Chapter 6. Additional results of other demand parameters such as Fatigue damage or Park and Ang damage indexes, R - μ - T relationships and equivalent linear expressions for PR systems with varying strain hardening are presented, as well as the ground-motion dataset employed.

Simplified R - μ - T relationships

On the basis of the extensive database generated in the course of the studies of Chapter 6, constant strength-ductility-period (R - μ - T) relationships were generated as presented in Figure B.1a for bi-linear systems in terms of normalized periods (i.e. T/T_g). Also presented in Figure B.1a are the R - μ - T tri-linear idealizations that were found to represent reasonably well the main features of the highly non-linear R - μ - T curves. These idealizations are summarized in Figure B.1b.

APPENDIX B

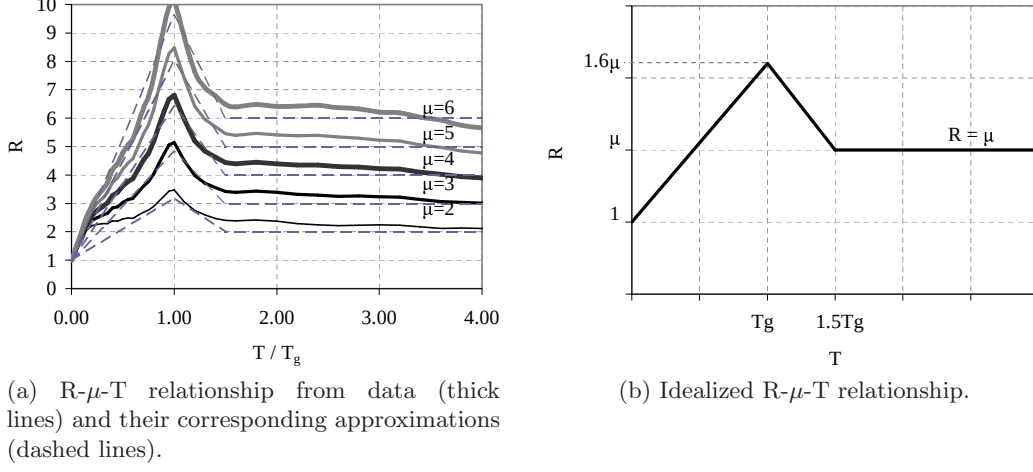


Figure B.1: R- μ -T relationships.

Equivalent linear parameters for PR systems with varying post-yield stiffness

The equivalent period T_{eq} is defined as a function of the strength ratio R , the predominant period of the ground-motion T_g , the initial structural period T and the strain hardening coefficient S_h (i.e. ratio of post-elastic to elastic stiffness) as:

$$\begin{aligned}
 T_{eq}/T &= \sqrt{\frac{R}{1 + S_h(R - 1)}} & \text{if } T > 1.5T_g \\
 T_{eq}/T &= \sqrt{\frac{(R - 1)T_g + T}{S_h(R - 1)T_g + T(1.6 - 0.6S_h)}} & \text{if } T < T_g \\
 T_{eq}/T &= \sqrt{\frac{RT_g}{S_hRT_g + (1 - S_h)(2.8T_g - 1.2T)}} & \text{if } T_g \leq T \leq 1.5T_g
 \end{aligned} \tag{B.1}$$

On the basis of the extensive database generated in Chapter 6 equivalent damping ratios (ξ_{eq}) were determined for PR structures with $T \leq 1.0$ s as:

$$\begin{aligned}
 \xi_{eq} &= 0.05 + ae^{bT} \\
 a &= 1.425(1 + S_h)R^{-0.25} \\
 b &= (0.28 + P/1.5)\ln(R) - 2.7
 \end{aligned} \tag{B.2}$$

Whereas for PR structures with $T \geq 1.2$ s:

$$\begin{aligned}\xi_{eq} &= 0.05 + a \ln R + b \\ a &= 0.0299P - 0.36S_h + 0.14 \\ b &= 0.135S_h + 0.02826\end{aligned}\tag{B.3}$$

Values for PR structures with $1 < T < 1.2$ s can be found by linear interpolation.

Fatigue and Park and Ang damage indexes for PR models

The extensive data generated over the course of the studies in Chapter 6 enable the calculation of additional demand parameters like the Park and Ang or Fatigue damage indexes. Figure B.2 presents the result for normalized Fatigue damage index normalized by the corresponding Fatigue damage in bi-linear structures. The Fatigue Damage Index is defined as:

$$F_{Damage} = \sum_{i=1}^{i=N_c} \frac{1}{a} S^b\tag{B.4}$$

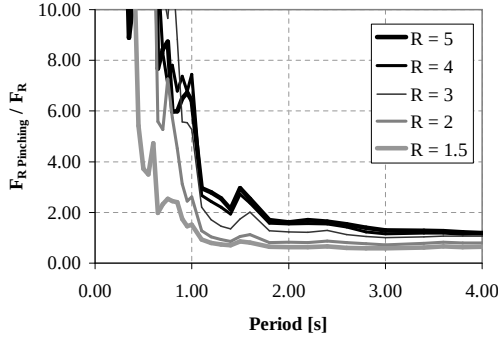
where a and b are the fatigue constant and exponent, respectively, S is the amplitude of the i^{th} cycle and N_c is the total number of displacement cycles. The fatigue damage is calculated for $b = 3$ which is in accordance with the experimental results presented in Chapters 3 and 4. Besides, a value of $a = 1$ was used here as the focus of the calculation was on generalized relative differences between PR and MR models rather than on a precise quantification of the fatigue index.

The second damage measure calculated is the global Park and Ang damage index defined as:

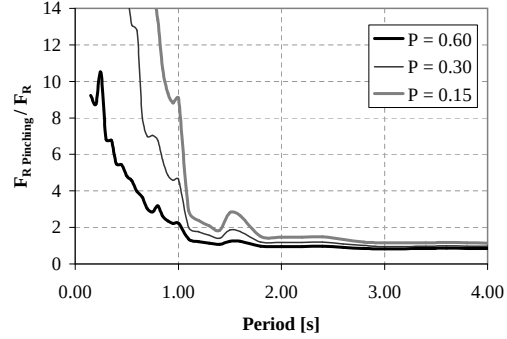
$$D = \frac{\delta_m}{\delta_u} + \frac{c}{F_y \delta_y} \int de\tag{B.5}$$

where δ_m is the maximum displacement, δ_u is the ultimate monotonic displacement capacity of the structure, c is an empirical factor that changes the balance between the extreme displacement and the hysteretic energy $\int de$, and F_y and δ_y are the yield force

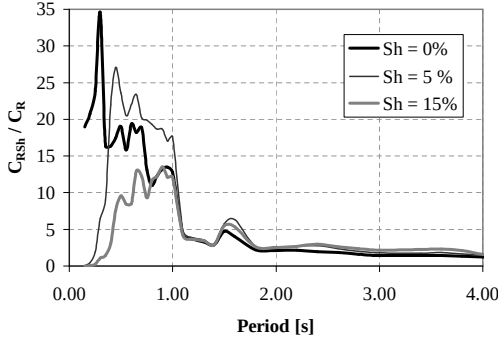
APPENDIX B



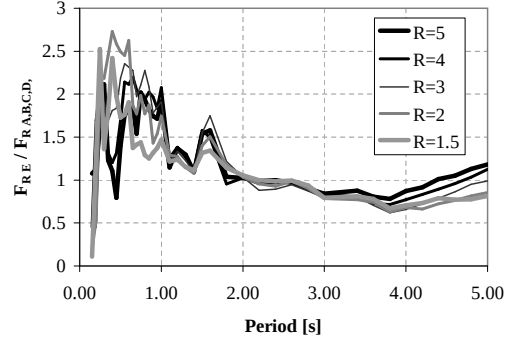
(a) Mean fatigue damage index for PR systems normalized by mean fatigue damage index of bi-linear systems for stiff soil ground-motions. $P = 0.3$.



(b) Mean fatigue damage index for PR systems normalized by mean fatigue damage index of bi-linear systems for all strength values on stiff soils as a function of pinching factor (P).



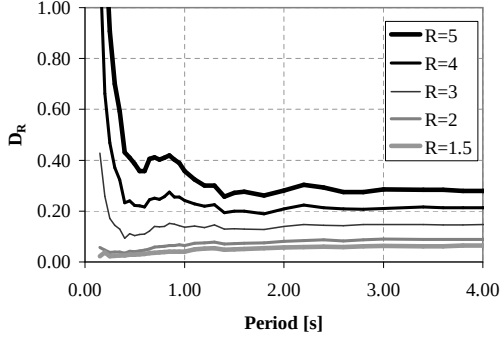
(c) Mean fatigue damage index for PR systems normalized by mean fatigue damage index of bi-linear systems on stiff soils as for different strain hardening coefficients. $R = 5$.



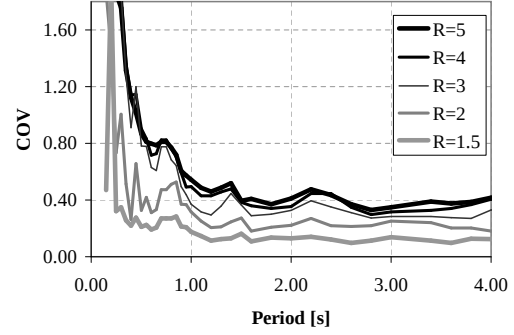
(d) Mean fatigue damage index for PR systems on soft soils normalized by mean damage index of PR systems on stiff soils

Figure B.2: Fatigue damage index of PR systems.

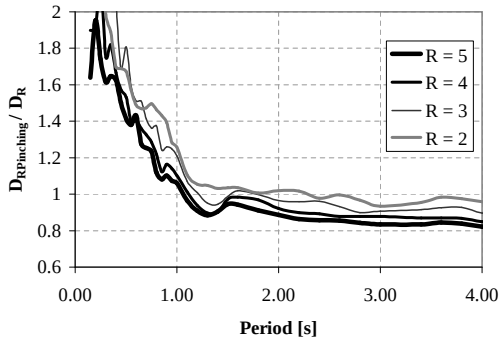
and yield displacement, respectively. In this study, c is set to 0.15 in accordance with normal practice. Figure B.3 presents the estimations of Park and Ang damage index for PR models for different strength demands and pinching parameters. The shaded areas in Figures B.3e and B.3f represent structural failure regions (i.e. Park and Ang damage indexes greater than one)



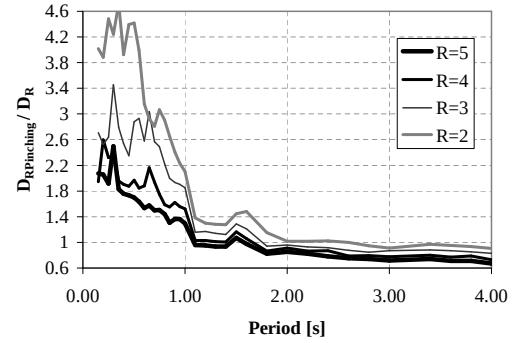
(a) Mean Park and Ang damage index index for bi-linear systems on stiff soils.



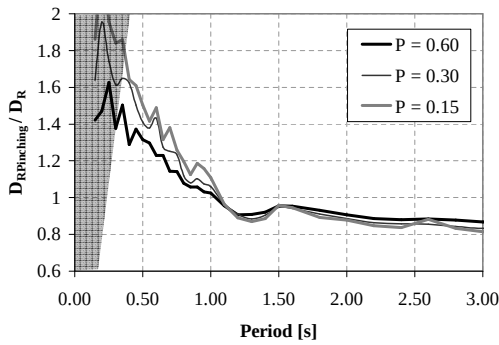
(b) Coefficients of variation of Park and Ang damage index for bi-linear systems on stiff soils.



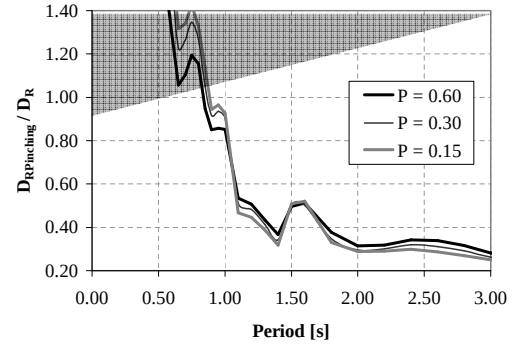
(c) Mean Park and Ang damage index for pinching systems normalized by mean Park and Ang damage index of bi-linear systems on stiff soils for a pinching factor ($P = 0.3$).



(d) Mean Park and Ang damage index for PR systems normalized by mean Park and Ang damage index of bi-linear systems on soft soils for a pinching factor ($P = 0.3$).



(e) Mean Park and Ang damage index for PR systems normalized by mean Park and Ang damage index of bi-linear systems on stiff soils for different Pinching factors. $R = 5$.



(f) Mean Park and Ang damage index for PR systems normalized by mean Park and Ang damage index of bi-linear systems on soft soils for different Pinching factors. $R = 5$.

Figure B.3: Park and Ang damage index of bi-linear and PR systems.

APPENDIX B

Ground-motion catalogue employed

Table B.1: Acceleration time series used in Chapter 6.

Earthquake name	Magnitude Mw	Station	Distance [km]	PGA [cm/s ²]	NEHRP site class
1992 Cape Mendocino	7.01	Cape Mendocino	10.36	1468.85	C
1992 Cape Mendocino	7.01	Cape Mendocino	10.36	1019.75	C
1992 Cape Mendocino	7.01	Eureka - Myrtle & West	53.34	151.11	D
1992 Cape Mendocino	7.01	Eureka - Myrtle & West	53.34	174.82	D
1986 Chalfant Valley-01	5.77	Zack Brothers Ranch	10.54	279.58	D
1986 Chalfant Valley-01	5.77	Zack Brothers Ranch	10.54	202.59	D
1986 Chalfant Valley-02	6.19	Zack Brothers Ranch	14.33	438.32	D
1986 Chalfant Valley-02	6.19	Zack Brothers Ranch	14.33	392.12	D
2002 Denali, Alaska	7.90	Anchorage - DOI Off. of Aircraft	293.06	12.42	E
2002 Denali, Alaska	7.90	Anchorage - DOI Off. of Aircraft	293.06	22.39	E
2002 Denali, Alaska	7.90	Anchorage - Dowl Eng Warehouse	290.70	10.02	E
2002 Denali, Alaska	7.90	Anchorage - Dowl Eng Warehouse	290.70	13.91	E
2002 Denali, Alaska	7.90	Anchorage - State Fish & Game	293.05	13.41	E
2002 Denali, Alaska	7.90	Anchorage - State Fish & Game	293.05	11.92	E
1999 Duzce, Turkey	7.14	Ambarli	206.09	37.69	E
1999 Duzce, Turkey	7.14	Ambarli	206.09	24.73	E
1999 Duzce, Turkey	7.14	Lamont 1059	24.26	144.64	C
1999 Duzce, Turkey	7.14	Lamont 1059	24.26	130.07	C
1976 Friuli, Italy-01	6.50	Tolmezzo	20.23	344.65	C
1976 Friuli, Italy-01	6.50	Tolmezzo	20.23	308.83	C
1976 Gazli, USSR	6.80	Karakyr	12.82	596.70	C
1976 Gazli, USSR	6.80	Karakyr	12.82	703.92	C
1999 Hector Mine	7.13	Joshua Tree	52.29	143.04	C
1999 Hector Mine	7.13	Joshua Tree	52.29	186.05	C
1979 Imperial Valley-06	6.53	El Centro Array #3	28.65	261.27	E
1979 Imperial Valley-06	6.53	El Centro Array #3	28.65	216.87	E
1979 Imperial Valley-06	6.53	Aeropuerto Mexicali	2.47	320.53	D
1979 Imperial Valley-06	6.53	Aeropuerto Mexicali	2.47	254.73	D
1980 Irpinia, Italy-01	6.90	Bagnoli Irpinio	22.65	136.71	B
1980 Irpinia, Italy-01	6.90	Bagnoli Irpinio	22.65	198.21	B
1980 Irpinia, Italy-01	6.90	Sturno	30.35	245.89	B
1980 Irpinia, Italy-01	6.90	Sturno	30.35	350.97	B
1952 Kern County	7.36	Taft Lincoln School	43.39	153.00	C
1952 Kern County	7.36	Taft Lincoln School	43.39	174.41	C
1995 Kobe, Japan	6.90	Kobe University	25.40	284.57	B
1995 Kobe, Japan	6.90	Kobe University	25.40	304.56	B
1999 Kocaeli, Turkey	7.51	Gebze	47.03	239.44	B
1999 Kocaeli, Turkey	7.51	Gebze	47.03	134.64	B
1999 Kocaeli, Turkey	7.51	Izmit	5.31	149.19	A
1999 Kocaeli, Turkey	7.51	Izmit	5.31	215.37	A
1999 Kocaeli, Turkey	7.51	Ambarli	112.26	243.94	E
1999 Kocaeli, Turkey	7.51	Ambarli	112.26	181.37	E
1992 Landers	7.28	Lucerne	44.02	713.03	C
1992 Landers	7.28	Lucerne	44.02	774.17	C
1994 Little Skull Mtn,NV	5.65	Station #1-Lathrop Wells	14.12	128.75	B
1994 Little Skull Mtn,NV	5.65	Station #1-Lathrop Wells	14.12	208.81	B
1994 Little Skull Mtn,NV	5.65	Station #2-NTS Control Pt. 1	30.17	116.71	B
1989 Loma Prieta	6.93	So. San Francisco, Sierra Pt.	83.53	103.13	B
1989 Loma Prieta	6.93	UCSC	16.51	303.23	B
1989 Loma Prieta	6.93	UCSC	16.51	388.07	B

Table B.1: Acceleration time series used in Chapter 6 (Cont.).

Earthquake name	Magnitude	Station	Distance	PGA	NEHRP
	M _w		[km]	[cm/s ²]	site class
1989 Loma Prieta	6.93	APEEL 2 - Redwood City	63.49	268.32	E
1989 Loma Prieta	6.93	APEEL 2 - Redwood City	63.49	215.98	E
1989 Loma Prieta	6.93	Foster City - APEEL 1	64.02	262.53	E
1989 Loma Prieta	6.93	Foster City - APEEL 1	64.02	288.80	E
1989 Loma Prieta	6.93	Larkspur Ferry Terminal (FF)	114.87	134.78	E
1989 Loma Prieta	6.93	Larkspur Ferry Terminal (FF)	114.87	94.64	E
1989 Loma Prieta	6.93	Treasure Island	97.43	98.24	E
1989 Loma Prieta	6.93	Treasure Island	97.43	156.18	E
1989 Loma Prieta	6.93	Alameda Naval Air Stn Hanger	90.77	263.17	D
1989 Loma Prieta	6.93	Alameda Naval Air Stn Hanger	90.77	205.45	D
1980 Mammoth Lakes-01	6.06	Convict Creek	1.43	408.57	D
1980 Mammoth Lakes-01	6.06	Convict Creek	1.43	433.20	D
1990 Manjil, Iran	7.37	Abbar	37.90	504.78	C
1990 Manjil, Iran	7.37	Abbar	37.90	486.92	C
1984 Morgan Hill	6.19	Gilroy Array #3	38.20	190.58	D
1984 Morgan Hill	6.19	Gilroy Array #3	38.20	196.59	D
1986 N. Palm Springs	6.06	Morongo Valley	6.28	214.06	D
1986 N. Palm Springs	6.06	Morongo Valley	6.28	201.08	D
1985 Nahanni, Canada	6.76	Site 1	6.80	959.25	C
1985 Nahanni, Canada	6.76	Site 1	6.80	1074.88	C
2002 Nenana Mountain, Alaska	6.70	Anchorage - DOI Off. of Aircraft	277.25	10.24	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - DOI Off. of Aircraft	277.25	10.07	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - Dowl Eng Warehouse	275.28	6.52	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - Dowl Eng Warehouse	275.28	7.02	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - K2-19	276.33	10.73	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - K2-19	276.33	11.38	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - State Fish & Game	277.70	6.61	E
2002 Nenana Mountain, Alaska	6.70	Anchorage - State Fish & Game	277.70	7.08	E
1994 Northridge-01	6.69	Burbank - Howard Rd.	23.18	117.46	B
1994 Northridge-01	6.69	Burbank - Howard Rd.	23.18	160.21	B
1994 Northridge-01	6.69	LA - Griffith Park Observatory	25.42	283.71	B
1994 Northridge-01	6.69	LA - Griffith Park Observatory	25.42	160.49	B
1994 Northridge-01	6.69	LA - Wonderland Ave	18.99	110.19	B
1994 Northridge-01	6.69	LA - Wonderland Ave	18.99	168.86	B
1994 Northridge-01	6.69	Mt Wilson - CIT Seis Sta	45.77	229.44	B
1994 Northridge-01	6.69	Mt Wilson - CIT Seis Sta	45.77	131.90	B
1994 Northridge-01	6.69	Pacoima Dam (downstr)	20.36	407.43	A
1994 Northridge-01	6.69	Pacoima Dam (downstr)	20.36	425.82	A
1994 Northridge-01	6.69	Pacoima Dam (upper left)	20.36	1554.79	A
1994 Northridge-01	6.69	Pacoima Dam (upper left)	20.36	1260.78	A
1971 San Fernando	6.61	Santa Felita Dam (Outlet)	31.55	145.57	C
1971 San Fernando	6.61	Santa Felita Dam (Outlet)	31.55	148.98	C
1986 San Salvador	5.80	National Geografical Inst	9.54	398.66	D
1986 San Salvador	5.80	National Geografical Inst	9.54	600.51	D
1987 Superstition Hills-02	6.54	Brawley Airport	29.91	152.94	D
1987 Superstition Hills-02	6.54	Brawley Airport	29.91	113.76	D
1978 Tabas, Iran	7.35	Tabas	55.24	819.93	B
1978 Tabas, Iran	7.35	Tabas	55.24	835.58	B
1981 Westmorland	5.90	Parachute Test Site	20.47	237.25	D
1981 Westmorland	5.90	Parachute Test Site	20.47	152.18	D

Appendix C

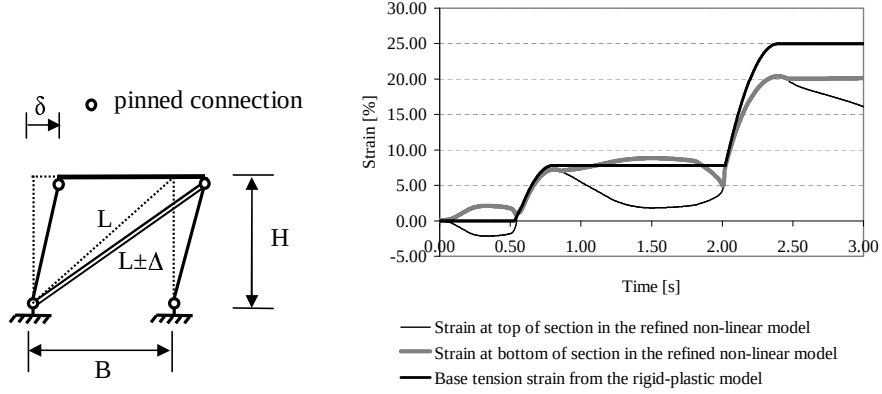
Estimation of Fatigue Damage through Rigid-plastic Models

This Appendix illustrates possible benefits of using rigid-plastic response histories analysis to establish preliminary predictions of local demand parameters and fatigue indexes. The fracture life of bracing members under inelastic cyclic loading is directly related to the high strains that develop upon local buckling at critical cross-sections. Reliable prediction of the strain history at the cross-section level is hence important in predicting failure due to low-cycle fatigue in CB frames.

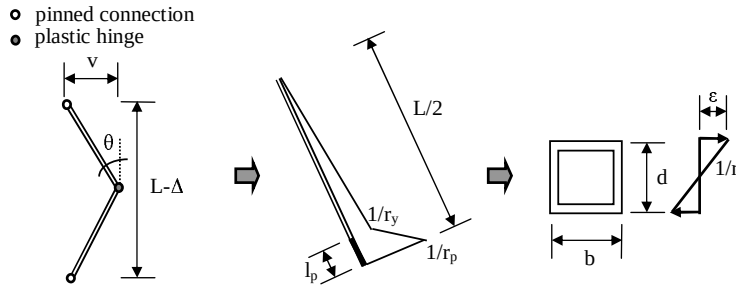
The process used to relate the lateral storey deformations to local strain demands in bracing members involves three steps. First, the lateral frame deformations are converted to an axial deformation in the brace which, as illustrated in Figure C.1a for a bay of width B and height H is given as

$$\Delta = \sqrt{H^2 + (B + \delta)^2} - \sqrt{H^2 + B^2} \quad (\text{C.1})$$

Secondly, a base tension strain is defined as the strain that is accumulated in the brace in time. Each instance the brace elongates, a non-recoverable deformation takes place which in this study is assumed to be uniformly distributed along the length of the brace. As the integration algorithms presented in previous sections keep a record of the successive peak displacements along the time scale, the acquisition of the evolution



(a) Relation between lateral storey drift and axial deformation in the brace. (b) Base tension strain from rigid-plastic model and strains at top and bottom of the cross section of a 40x2.5 RHS brace when a stepping harmonic motion is applied to the CB oscillator.



(c) Assumed mechanism at the midspan plastic hinge of the brace member (left), distribution of curvature along half-length of the brace (centre) and strain-curvature relationship at the section level (right).

Figure C.1: Prediction of local strains.

of such tension base strain is a straightforward task.

Figure C.1b presents the tension base strain obtained for a one-bay one-storey braced structure of 2.5 metres width, height of 2.9 meters, and a SHS 40x2.5 brace. In this case a stepping harmonic excitation is applied on the structure. The values of strains at the top and bottom of the section at mid-length of the brace obtained by means of a refined non-linear model are also presented for comparison. Finally, the strains caused by the rotation at the plastic hinge are added to the strains caused by the tension. Such plastic hinge related strains are calculated assuming a concentration of the rotation at the plastic hinge at midspan of the brace member as depicted in Figure C.1c, where v is the lateral displacement of the brace and θ is the rotation at the plastic hinge.

The lateral displacement can be related to the axial deformation by:

$$v = \sqrt{\left(\frac{L}{2}\right)^2 - \left(\frac{L - \Delta}{2}\right)^2} \quad (\text{C.2})$$

with a plastic hinge rotation of:

$$\theta = \arccos\left(\frac{L - \Delta}{L}\right) \quad (\text{C.3})$$

The same plastic hinge rotation can be obtained with reference to Figure C.1c as:

$$\theta = \frac{1}{2r_y} \left(\frac{L}{2} - l_p\right) + \frac{1/r_y + 1/r_p}{2} \cdot l_p \quad (\text{C.4})$$

where $1/r_y$ is the yield curvature, $1/r_p$ the plastic curvature, and l_p the length of the plastic hinge, considered as:

$$l_p = \frac{L}{2} \left(1 - \frac{M_y}{M_p}\right) \quad (\text{C.5})$$

where M_y and M_p are the yield and plastic moment respectively. From the above equations, the curvature at the plastic hinge can be readily obtained and in turn such curvature can be related to the strain at the extreme fibre of the cross section by:

$$\epsilon = \frac{d(1/r)}{2} \quad (\text{C.6})$$

This strain should then be added or subtracted from the previously obtained base tension strain.

Figure C.2 presents the evolution of strain demands at the plastic hinge, as deter-

APPENDIX C

mined using the procedure described above. The same Figure also presents the strains obtained at the gauss points from the Adaptive model, when the CBF oscillator with SHS 40x2.5 braces is subjected to the El Centro record scaled in order to get a ratio between PGA and acceleration at yield of 2.5. As shown in the figure, the results are generally encouraging. Moreover, the observed slight over estimation of strains, specially for the later part of the history, is due to the concentration of deformation at the plastic hinge zone neglecting all the elastic deformations in the member, which nevertheless offer a reliable upper bound as already observed by other researchers [120].

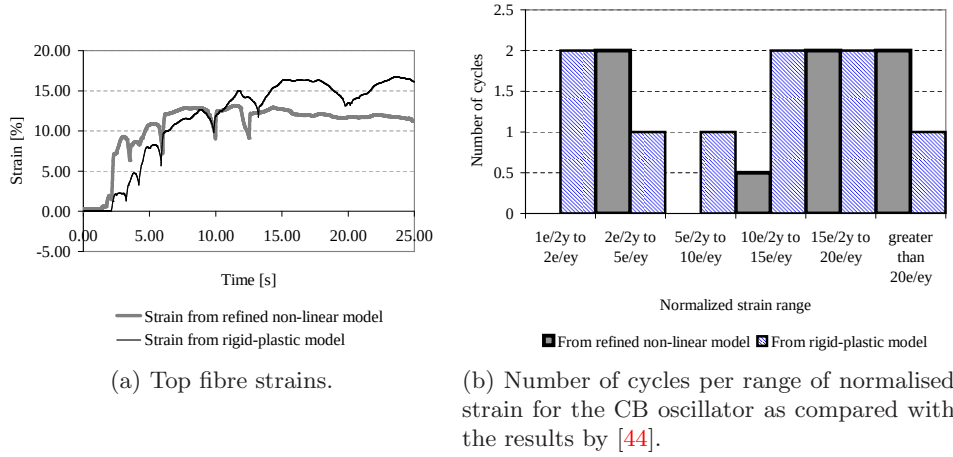


Figure C.2: Comparison of strain demand history for the CB oscillator with 40x2.5 SHS when subjected to the El Centro record scaled so as to obtain a ratio PGA/a_y of 2.5.

As a further development, the model can be coupled with a cyclic counting procedure in order to quantify the fatigue damage index. An implementation of the rainflow counting method is presented in Figure C.2b for the same parameters previously described. The damage indices calculated based on the Palmgren-Minner's rule [111] are 8.6 when using the strains from the refined model and 10.8 when using the strains from the rigid-plastic approximation. These results were calculated in accordance with previous studies using the same bracing configuration [44].

References

- [1] H. AGERSKOV. High strength bolted connections subjected to prying. *Journal of the Structural Division, American Society of Civil Engineering*, **102**[1]:161–175, 1976. [12](#)
- [2] AISC. Hollow structural section connections manual, American Institute of Steel Construction, Chicago, Illinois, US, 1997. [13](#)
- [3] C. AMADIO, I. CLEMENTE, L. MARCORINI, AND M. FRAGIACOMO. Seismic behaviour of hybrid systems made of PR composite frames coupled with dissipative bracings. *Earthquake Engineering and Structural Dynamics*, **37**[6]:861–879, 2008. [17](#)
- [4] ATC. Seismic evaluation and retrofit of concrete buildings, Report ATC-40, California, US, 1994. [15](#)
- [5] ATC. Improvement of nonlinear static seismic analysis procedures, Report FEMA-440, Washington, US, 2005. [13](#), [105](#)
- [6] G. AUGUSTI. Rigid-plastic structures subject to dynamic loads. *Meccanica*, **5**[2]:74–84, 1969. [16](#), [134](#)
- [7] A. AZIZINAMINI AND J.B. RADZIMINSKI. Static and cyclic performance of semirigid steel beam-to-column connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **115**[12]:2979–2999, 1989. [60](#), [97](#)
- [8] J.F. BAKER. *The Steel Skeleton, Vol. 2*. Cambridge University Press, 1956. [3](#)
- [9] M. BALLERINI, M. PIAZZA, E. BOZZO, AND F. OCCHI. Shear capacity of blind bolted connections for RHS steel structural elements. In *Proceedings of the Seventh International Conference on Tubular Structures, Kansas*, pages 99–106, 1996. [10](#), [11](#)
- [10] G. BALLIO, L. CALADO, AND C. A. CASTIGLIONI. Low cycle fatigue behaviour of structural steel members and connections. *Fatigue and Fracture of Engineering Materials and Structures*, **20**[8]:1129–1146, 2007. [60](#), [97](#)
- [11] G. BANKS. Flowdrilling for tubular structures. In *Proceedings of the Fifth International Conference on Tubular Structures, Nottingham*, 1993. [10](#)
- [12] G. BANKS. Hollowbolt joint shear tests project No S2860. Technical report, British Steel Plc., Swinden Technology Centre, Rotherham, 1997. [11](#), [28](#)
- [13] T. BARNETT. *The behaviour of a blind bolt for moment resisting connections in hollow steel sections*. PhD thesis, Department of Civil Engineering, University of Nottingham, UK, 2001. [11](#)
- [14] T. BARNETT, W. TIZANI, AND D.A. NETHERCOT. Blind bolted moment resisting connections to structural hollow sections. In *Connections in Steel Structures IV, Virginia*, 2000. [11](#)

REFERENCES

- [15] T. BARNETT, W. TIZANI, AND D.A. NETHERCOT. The practice of blind bolting connections to structural hollow sections: A review. *Steel and Composite Structures*, **1**[1]:1–16, 2001. [11](#), [28](#)
- [16] K. BEYER AND J.J. BOMMER. Selection and scaling of real accelerograms for bi-directional loading: a review of current practice and code provisions. *Journal of Earthquake Engineering*, **11**[S1]:13–45, 2007. [171](#)
- [17] Y. BOZORGNIA, M.M. HACHEM, AND K.W. CAMPBELL. Deterministic and probabilistic predictions of yield strength and inelastic displacement spectra. *Earthquake Spectra*, **26**[1]:25–40, 2010. [13](#), [105](#)
- [18] B.M. BRODERICK, A.Y. ELGHAZOULI, AND J.M. GOGGINS. Earthquake testing and response of concentrically-braced sub-frames. *Journal of Constructional Steel Research*, **64**[9]:997–1007, 2008. [140](#), [141](#)
- [19] M. BRUNEAU, C.M. UANG, AND A. WHITTAKER. *Ductile design of steel structures*. McGraw-Hill, 1998. [138](#), [139](#)
- [20] J.J. CAO, J.A. PACKER, AND N. KOSTESKI. Design guidelines for longitudinal plate to HSS connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **124**[7]:784–791, 1998. [10](#)
- [21] A. CHAKRABORTI AND V.K. GUPTA. Scaling of strength reduction factors for degrading elastoplastic oscillators. *Earthquake Engineering and Structural Dynamics*, **34**[2]:189–206, 2005. [15](#)
- [22] A.K. CHOPRA AND C. CHINTANAPAKDEE. Comparing response of sdf systems to near-fault and far-fault earthquake motions in the context of spectral regions. *Earthquake Engineering and Structural Dynamics*, **30**[12]:1769–1798, 2001. [14](#), [121](#)
- [23] A.K. CHOPRA AND C. CHINTANAPAKDEE. Inelastic deformation ratios for design and evaluation of structures: single-degree-of-freedom bilinear systems. *Journal of Structural Engineering, American Society of Civil Engineering*, **130**[9]:1309–1319, 2004. [13](#), [105](#)
- [24] A.K. CHOPRA AND R.K. GOEL. Evaluation of torsional provisions in seismic codes. *Journal of Structural Engineering, American Society of Civil Engineering*, **117**[12]:3783–3803, 1991. [17](#)
- [25] A.K. CHOPRA AND R.K. GOEL. A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings. *Earthquake Engineering and Structural Dynamics*, **33**[8]:903–927, 2004. [17](#)
- [26] COMITÉ EUROPÉEN DE NORMALISATION. EN 1993-1-1, Eurocode 3: Design provisions of steel structures, Part 1.1: General rules and rules for buildings, 2005. [111](#)
- [27] COMITÉ EUROPÉEN DE NORMALISATION. EN 1993-1-8, Eurocode 3: Design provisions of steel structures, Part 1.8: Design of joints, 2005. [10](#), [12](#), [13](#), [70](#), [92](#), [94](#), [96](#), [102](#)
- [28] COMITÉ EUROPÉEN DE NORMALISATION. EN 1998-1, Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, 2005. [3](#), [15](#), [109](#), [144](#), [150](#)
- [29] COMITÉ EUROPÉEN DE NORMALISATION. EN 1998-3, Eurocode 8: Design of structures for earthquake resistance, Part 3: Assessment and retrofit of buildings, 2005. [13](#), [15](#)
- [30] E.F. CRUZ AND S. COMINETTI. Three-dimensional buildings subjected to bi-directional earthquakes. Validity of analysis considering uni-directional earthquakes. In *12th World Conference on Earthquake Engineering*, 2000. [17](#)

REFERENCES

- [31] J.L. DAWE AND G. GRONDIN. W-shape to RHS column connections. *Canadian Journal of Civil Engineering*, **17**[5]:788–797, 1990. [10](#)
- [32] M. DE STEFANO AND A. ASTANEH-ASL. Axial force-displacement behaviour of steel double angles. *Journal of Constructional Steel Research*, **20**[3]:161–181, 1991. [72](#)
- [33] M. DE STEFANO, A. DE LUCA, AND A. ASTANEH-ASL. Modeling of cyclic moment-rotation response of double-angle connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **120**[1]:212–229, 1993. [12](#)
- [34] M. DE STEFANO, C. FAELLA, AND R. RAMASCO. Inelastic seismic response of one-way plan-asymmetric systems under bi-directional ground motions. *Earthquake Engineering and Structural Dynamics*, **27**[4]:363–376, 1998. [17](#)
- [35] J. DING AND Y.C. WANG. Experimental study of structural fire behavior of steel beam to concrete filled tubular column assemblies with different types of joints. *Engineering Structures*, **29**:3485–3502, 2007. [11](#), [43](#), [44](#)
- [36] J.L. DOMINGUES-COSTA, R. BENTO, V. LEVTCHITCH, AND M.P. NIELSEN. Simplified non-linear time-history analysis based on the theory of plasticity. In *Earthquake Resistant Engineering Structures V*, 2005. [152](#), [153](#)
- [37] J.L. DOMINGUES-COSTA, R. BENTO, V. LEVTCHITCH, AND M.P. NIELSEN. Rigid-plastic seismic design of reinforced concrete structures. *Earthquake Engineering and Structural Dynamics*, **36**[1]:55–76, 2006. [16](#), [134](#), [157](#)
- [38] D. DUTTA, J. WARDENIER, N. YEOMANS, K. SAKAE, O. BUCAK, AND J.A. PACKER. Fabrication, assemblage et montage des structures en profils creux. In *Serie CIDECT Guide de dimensionnement 7*. TUV-Verlag, Germany, 1999. [11](#)
- [39] H.M. DWAIRI, M.J. KOWALSKY, AND J.M. NAU. Equivalent damping in support of direct displacement based-design. *Journal of Earthquake Engineering*, **11**[4]:512–530, 2007. [15](#)
- [40] ECCS. European Convention for Constructional Steelwork, Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads, Brussels, 1986. [xix](#), [27](#), [46](#)
- [41] A.Y. ELGHAZOULI. Ductility of frames with semirigid connections. In *11th World Conference on Earthquake Engineering, Acapulco, Mexico*, 1996. [10](#)
- [42] A.Y. ELGHAZOULI. Seismic design procedures for concentrically braced frames. *Proceedings of the Institution of Civil Engineers, Structures and Buildings*, **156**[4]:381–394, 2003. [140](#)
- [43] A.Y. ELGHAZOULI. Rotational behaviour and seismic response of semi-rigid top-and-seat angle open beam-to-tubular column connections, Report ST-06/9. Technical report, Department of Civil and Environmental Engineering, Imperial College London, London, UK, 2006. [68](#)
- [44] A.Y. ELGHAZOULI, B.M. BRODERICK, J.M. GOGGINS, H. MOUZAKIS, P. CARYDIS, J. BOUWKAMP, AND A. PLUMIER. Shake table testing of tubular steel bracing members. *Proceedings of the Institution of Civil Engineers, Structures and Buildings*, **SB4**:229–241, 2005. [14](#), [141](#), [212](#)
- [45] A.Y. ELGHAZOULI, C. MÁLAGA-CHUQUITAYPE, J.M. CASTRO, AND A.H. ORTON. Experimental monotonic and cyclic behaviour of blind-bolted angle connections. *Engineering Structures*, **31**[11]:2540–2553, 2009. [12](#), [14](#), [72](#), [92](#)

REFERENCES

- [46] A.S. ELNASHAI, A.Y. ELGHAZOU, AND F.A. DENESH-ASHTIANI. Response of semirigid steel frames to cyclic and earthquake loads. *Journal of Structural Engineering, American Society of Civil Engineering*, **124**[8]:857–867, 1998. [10](#)
- [47] R. ENGGLEKIRK. *Steel structures*. John Wiley & Sons, New York, US, 1994. [177](#)
- [48] M. ENOKI, B. XUAN LUONG, N. OKABE, AND K. ITOU. Dynamic theory of rigid-plasticity. *Soil Dynamics and Earthquake Engineering*, **25**[7-10]:635–647, 2005. [16](#)
- [49] E. ERDURAN AND K.L. RYAN. Effects of torsion on the behavior of peripheral steel-braced frame systems. *Earthquake Engineering and Structural Dynamics*, **In Press**, 2010. [17](#), [18](#)
- [50] C. FAELLA, V. PILUSO, AND G. RIZANNO. *Structural steel semirigid connections: Theory, design and software*. CRC Press, Florida, USA, 2000. [12](#)
- [51] P. FAJFAR. Capacity spectrum method based on inelastic demand spectra. *Earthquake Engineering and Structural Dynamics*, **28**[9]:979–993, 1999. [157](#)
- [52] P. FAJFAR, D. MARUSIC, AND I. PERUS. Torsional effects in the pushover-based seismic analysis of buildings. *Journal of Earthquake Engineering*, **9**[6]:831–854, 2005. [17](#)
- [53] FEMA. NEHRP guidelines for the seismic rehabilitation of buildings, Report FEMA-273, Washington D.C., US, 1997. [15](#)
- [54] FEMA. State of the art report on systems performance of steel moment frames subjected to earthquake ground motion shaking, Report FEMA-355F, Washington D.C., US, 1997. [3](#)
- [55] FEMA. FEMA 356 prestandard and commentary for the seismic rehabilitation of buildings Washington D.C., US, 2000. [180](#)
- [56] FEMA. Recommended seismic design criteria for new steel moment-frame buildings , Washington D.C., US, 2000. [3](#), [13](#)
- [57] J. FRANCE. *Bolted connections between open section beams and box columns*. PhD thesis, Department of Civil and Structural Engineering, University of Sheffield, UK, 1997. [11](#)
- [58] J. FRANCE, B. DAVISON, AND P.A. KIRBY. Strength and rotational response of moment connections to tubular columns using flowdrill connectors. *Journal of Constructional Steel Research*, **50**[1]:1–14, 1999. [10](#)
- [59] J. FRANCE, B. DAVISON, AND P.A. KIRBY. Strength and rotational stiffness of simple connections to tubular columns using flowdrill connectors. *Journal of Constructional Steel Research*, **50**[1]:15–34, 1999. [10](#)
- [60] M. GARLOCK, J. RICLES, AND R. SAUSE. Cyclic load tests and analysis of bolted top and seat angle connection. *Journal of Structural Engineering, American Society of Civil Engineering*, **129**[12]:1615–1625, 2003. [10](#), [12](#), [77](#)
- [61] D. GEORGESCU, C. TOMA, AND O. GOSA. Post-critical behaviour of K braced frames. *Journal of Constructional Steel Research*, **21**[1-3]:115–133, 1991. [140](#)
- [62] A. GHERSI AND P. ROSSI. Influence of bi-directional ground motions on the inelastic response of one-storey in-plan irregular structures. *Engineering Structures*, **23**[6]:579–591, 2001. [17](#)
- [63] A. GHOBARAH, S. MOURAND, AND R. KOROL. Moment-rotation relationship of blind bolted connections to tubular columns. *Journal of Constructional Steel Research*, **40**[1]:63–91, 1996. [12](#), [63](#), [69](#)

REFERENCES

- [64] K. GODA AND G.M. ATKINSON. Non-iterative equivalent linearization of inelastic sdof systems for earthquakes in japan and california. *Earthquake Engineering and Structural Dynamics*, **39**[11]:1219–1238, 2010. [16](#)
- [65] K. GODA, H.P. HONG, AND C.S. LEE. Probabilistic characteristics of seismic ductility demand of sdof system with Bouc-Wen hysteretic behavior. *Journal of Earthquake Engineering*, **13**[5]:600–622, 2009. [14](#), [105](#)
- [66] R.K. GOEL. Seismic response of asymmetric systems energy based approach. *Journal of Structural Engineering, American Society of Civil Engineering*, **123**[11]:1444–1453, 1997. [17](#)
- [67] F.C.T. GOMES. Etat limite ultime de la résistance de l’ame d’une colonne dans un assemblage semi-rigide d’axe faible. Rapport Interne 203. Technical report, University of Liege, 1990. [68](#)
- [68] P. GULKAN AND M. SOZEN. Inelastic response of reinforced concrete structures to earthquake actions. *ACI Journal*, **71**[12]:604–610, 1974. [15](#)
- [69] A. GUYADER. *A statistical approach to equivalent linearization with application to performance-based engineering*. PhD thesis, California Institute of Technology, California, US, 2003. [15](#)
- [70] HAZUS. Hazuz99 technical manual SR-2. Loss estimation methodology. Technical report, HAZUS, 1999. [180](#)
- [71] A. HEIDARPOUR AND M.A. BRADFORD. Behaviour of a T-stub assembly in steel beam-to-column connections at elevated temperatures. *Engineering Structures*, **30**[10]:2893–2899, 2008. [65](#)
- [72] HUCK INTERNATIONAL INC. Industrial fastening systems, Arizona, US, 1990. [11](#)
- [73] L.F. IBARRA AND H. KRAWINKLER. Global collapse of frame structures under seismic excitation. TR -152. Technical report, The John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, US, 2005. [175](#)
- [74] K. IKEDA AND S.A. MAHIN. A refined physical theory model for predicting the seismic behaviour of braced frames. Report UMEE 77r3. Technical report, Department of Civil Engineering, University of Michigan, US, 1984. [140](#)
- [75] B.A. IZZUDDIN. *Nonlinear dynamic analysis of framed structures*. PhD thesis, Department of Civil and Environmental Engineering, Imperial College London, UK, 1991. [139](#)
- [76] J.P. JASPART. *Étude de la semi-rigidité des noeuds pourter-colonne et son influence sur la résistance et al stabilité des ossatures en acier*. PhD thesis, Faculty of Applied Sciences, University of Liege, Belgium, 1991. [12](#)
- [77] S. KIGGINS AND C.M. UANG. Reducing residual drift of buckling restrained braced frames as dual systems. *Engineering Structures*, **28**[11]:1525–1532, 2006. [17](#)
- [78] N. KISHI, A. AHMED, AND N. YABUKI. Nonlinear finite element analyses of top and seat-angle with double web-angle connections. *International Journal of Structural Engineering and Mechanics*, **12**[2]:201–214, 2001. [12](#)
- [79] N. KISHI AND W.F. CHEN. Moment-rotation relation of semi-rigid connections with angles. *Journal of Structural Engineering, American Society of Civil Engineering*, **116**[7]:1813–1834, 1990. [12](#)
- [80] S. KLIPPEL. Recent design developments with blind mechanically operated bolt systems for use with hollow section steelwork. *Journal of Constructional Steel Research*, **46**[1]:267–268, 1998. [11](#), [28](#)

REFERENCES

- [81] R. KOROL, A. GHOBARAH, AND S. MOURAND. Blind bolting W-shape beams to HSS columns. *Journal of Structural Engineering, American Society of Civil Engineering*, **119**[12]:3463–3481, 1999. [11](#)
- [82] N. KOSTESKI AND J.A. PACKER. Longitudinal plate and through plate-to-hollow structural section welded connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **129**[4]:478–486, 2003. [10](#)
- [83] N. KOSTESKI AND J.A. PACKER. Welded tee to HSS connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **129**[2]:151–159, 2003. [10](#)
- [84] H. KRAWINKLER. State of art report on systems performance of moment resisting steel frames subject to earthquake ground shaking. Technical report, FEMA 355c, Federal Emergency Management Agency, Washington, D.C., 2000. [180](#)
- [85] A.R. KUKRETI AND A.S. ABOLMAALI. Moment-rotation hysteresis behavior of top and seat angle steel frame connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **125**[8]:810–820, 1999. [10](#)
- [86] M. KUMAR, J. M. CASTRO, P. J. STAFFORD, AND A. Y. ELGHAZOULI. Influence of the mean period of ground motion on the inelastic dynamic response of single and multi degree of freedom systems. *Earthquake Engineering and Structural Dynamics*, **In Press**, 2010. [15](#), [121](#)
- [87] Y. KUROBANE. Connections in tubular structures. *Progress in Structural Engineering and Materials*, **4**[1]:35–45, 2002. [10](#)
- [88] Y. KUROBANE, J.A. PACKER, J. WARDENIER, AND N. YEOMANS. Design guide for structural hollow section column connections. In *Serie CIDECT Construction with Hollow Sections*. TUV-Verlag, Germany, 2004. [10](#), [13](#)
- [89] W.P. KWAN AND S.L. BILLINGTON. Influence of hysteretic behavior on equivalent period and damping of structural systems. *Journal of Structural Engineering, American Society of Civil Engineering*, **129**[5]:576–585, 2003. [15](#)
- [90] C.S. LEE AND H.P. HONG. Statistics of inelastic responses of hysteretic systems under bidirectional seismic actions. *Engineering Structures*, **32**[8]:2074–2086, 2010. [17](#)
- [91] M.E. LEMONIS AND C.J. GANTES. Incremental modeling of T-stub connections. *Journal of Materials and Structures*, **1**[7]:1135–1159, 2006. [12](#), [65](#)
- [92] R.T. LEON, J.J. HOFFMAN, AND P.E. STAEGER. Partially restrained composite connections - A design guide, Steel Design Guide Series, American Institute of Steel Construction, Chicago,, 1996. [10](#)
- [93] K.W. LIAO, Y.K. WEN, AND D.A. FOUTCH. Evaluation of 3d steel moment frames under earthquake excitations: 1: Modeling. *Journal of Structural Engineering, American Society of Civil Engineering*, **133**[3]:462–470, 2007. [17](#)
- [94] K.W. LIAO, Y.K. WEN, AND D.A. FOUTCH. Evaluation of 3d steel moment frames under earthquake excitations: 2: Reliability and redundancy. *Journal of Structural Engineering, American Society of Civil Engineering*, **133**[3]:471–480, 2007. [18](#)
- [95] D.G. LIGNOS, H. KRAWINKLER, AND F. ZAREIAN. Modelling of component deterioration for collapse prediction of steel frames. In *6th International Conference on Behavior of Steel Structures in Seismic Areas, STESSA 2009, Philadelphia, Pennsylvania, US*, 2009. [175](#), [180](#)

REFERENCES

- [96] J.L. LIN AND K.C. TSAI. Seismic analysis of two-way asymmetric building systems under bi-directional seismic ground motions. *Earthquake Engineering and Structural Dynamics*, **37**[2]:305–328, 2008. [17](#)
- [97] Y. LIN AND Y. LIN. Non-iterative equivalent linearization based on secant period for estimating maximum deformations of existing structures. *Journal of Earthquake Engineering*, **13**[2]:170–192, 2009. [15](#), [16](#), [124](#), [125](#)
- [98] LINDAPTER INTERNATIONAL LTD. Type HB Holo-bolt for blind connection to structural steel and structural tubes, Bradford, England, 1995. [xix](#), [11](#), [23](#), [27](#), [28](#)
- [99] S.A. MAHIN. Sustainable design considerations in earthquake engineering. In *14th World Conference on Earthquake Engineering*, 2008. [3](#)
- [100] C. MÁLAGA-CHUQUITAYPE. *Can a SDOF system predict a MDOF response?* Master’s thesis, Centre for Post-graduate Training and Research in Earthquake Engineering and Engineering Seismology, Pavia, Italy, 2007. [13](#)
- [101] C. MÁLAGA-CHUQUITAYPE AND A.Y. ELGHAZOULI. Behaviour of combined channel/angle connections to tubular columns under monotonic and cyclic loading. *Engineering Structures*, **32**[11]:1600–1616, 2010. [14](#)
- [102] C. MÁLAGA-CHUQUITAYPE, A.Y. ELGHAZOULI, AND R. BENTO. Rigid-plastic models for the seismic design and assessment of steel framed structures. *Earthquake Engineering and Structural Dynamics*, **38**[11]:1609–1630, 2009. [128](#), [129](#)
- [103] J.B. MANDER, R.J. DHAKAL, N. MASHIKO, AND K.M. SOLBERG. Incremental dynamic analysis applied to seismic financial risk assessment of bridges. *Engineering Structures*, **29**[10]:2662–2672, 2007. [180](#)
- [104] J.B. MANDER, S.C. STUART, AND G. PEKCAN. Low-cycle fatigue behavior of semirigid top and seat angle connections. *Engineering Journal AISC*, **third quarter**[3]:111–122, 1994. [10](#), [98](#)
- [105] J.B. MANDER, S.C. STUART, AND G. PEKCAN. Low-cycle variable amplitude fatigue modeling of top-and-seat angle connections. *Engineering Journal AISC*, **second quarter**[3]:54–62, 1995. [98](#), [99](#), [100](#), [191](#)
- [106] R. MAQUOI, X. NAVEAU, AND J. RONDAL. Column welded stud connections. *Journal of Constructional Steel Research*, **4**:3–26, 1984. [10](#)
- [107] D. MARUSIC AND P. FAJFAR. On the inelastic seismic response of asymmetric buildings under bi-axial excitation. *Earthquake Engineering and Structural Dynamics*, **34**[8]:943–963, 2005. [17](#)
- [108] MATLAB. 7.3 Getting Started Guide, The MathWorks Inc., USA, 2006. [72](#), [137](#), [197](#)
- [109] F. MCKENNA. *Object Oriented Finite Element Programming: Frameworks for Analysis, Algorithms and Parallel Computing*. PhD thesis, University of California, Berkeley, Berkeley, CA, US. <http://opensees.berkeley.edu>, 1997. [108](#), [167](#), [177](#), [180](#)
- [110] R.A. MEDINA AND H. KRAWINKLER. Evaluation of drift demands for the seismic performance assessment of frames. *Journal of Structural Engineering, American Society of Civil Engineering*, **131**[7]:1003–1013, 2005. [171](#)
- [111] M.A. MINER. Cumulative damage in fatigue. *Journal of Applied Mechanics*, **67**:A159–A164, 1945. [212](#)

REFERENCES

- [112] E. MIRANDA. *Seismic evaluation and upgrading of existing buildings*. PhD thesis, University of California at Berkeley, Berkeley, US, 1991. [13](#), [15](#)
- [113] E. MIRANDA. Site-dependent strength-reduction factors. *Journal of Structural Engineering, American Society of Civil Engineering*, **119**[12]:3503–3519, 1993. [15](#), [121](#)
- [114] E. MIRANDA. Inelastic displacement ratios for structures on firm sites. *Journal of Structural Engineering, American Society of Civil Engineering*, **126**[10]:740–757, 2000. [13](#), [105](#)
- [115] E. MIRANDA AND S. AKKAR. Dynamic instability of simple structural systems. *Journal of Structural Engineering, American Society of Civil Engineering*, **129**[12]:1722–1726, 2003. [176](#)
- [116] E. MIRANDA AND J. RUIZ-GARCIA. Influence of stiffness degradation on strength demands of structures built on soft soils sites. *Engineering Structures*, **24**[10]:1271–1281, 2002. [14](#), [105](#)
- [117] L.F.N. NEVES. *Comportamento monotónico e cíclico de ligações de eixo fraco e tubulares em estruturas metálicas e mistas aço-betão*. [in Portuguese]. PhD thesis, University of Coimbra, Portugal, 2004. [10](#), [75](#), [90](#)
- [118] N.M. NEWMARK. Effects of earthquakes on dams and embankments, Fifth Rankine Lecture. *Geotechnique*, **2**:139–160, 1965. [16](#), [134](#)
- [119] N.M. NEWMARK AND W.J. HALL. Earthquake spectra and design. Technical report, Earthquake Engineering Research Institute, Oakland, California, 1982. [13](#), [14](#)
- [120] K.H. NIP. *Cyclic behaviour of carbon steel and stainless steel tubular members*. PhD thesis, Department of Civil and Environmental Engineering, Imperial College London, London, UK, 2009. [212](#)
- [121] K.H. NIP, L. GARDNER, AND A.Y. ELGHAZOULI. Cyclic testing and numerical modelling of carbon steel and stainless steel tubular bracing members. *Engineering Structures*, **32**[2]:424–441, 2010. [108](#)
- [122] P. NOGUEIRO, L. SIMOES DA SILVA, R. BENTO, AND R. SIMOES. Numerical implementation and calibration of a hysteretic model with pinching for the cyclic response of steel joints. *International Journal of Advanced Steel Construction*, **3**:459–484, 2007. [106](#), [107](#)
- [123] F. OCCHI. Hollow section connections using (hollowfast) hollowbolt expansion bolting, second interim report 6G-16/95. Technical report, Sidercad, Italy, 1995. [11](#), [28](#)
- [124] A. PAGLIETTI AND M.C. PORCU. Rigid-plastic approximation to predict plastic motion under strong earthquakes. *Earthquake Engineering and Structural Dynamics*, **30**[1]:115–126, 2001. [16](#), [134](#), [138](#), [152](#)
- [125] T. PAULAY. Some design principles relevant to torsional phenomena in ductile buildings. *Journal of Earthquake Engineering*, **5**[3]:273–308, 2001. [17](#)
- [126] I. PERUS AND P. FAJFAR. On the inelastic torsional response of single-storey structures under bi-axial excitation. *Earthquake Engineering and Structural Dynamics*, **34**[8]:931–941, 2005. [17](#)
- [127] D. PETTINGA, C. CHRISTOPOULOS, S. PAMPANIN, AND M.J.N. PRIESTLEY. Effectiveness of simple approaches in mitigating residual deformations in buildings. *Earthquake Engineering and Structural Dynamics*, **36**[12]:1765–1785, 2007. [17](#)
- [128] M.J.N. PRIESTLEY. *Myths and fallacies in earthquake engineering, revisited. The ninth Mallet Milne Lecture*. IUSS Press, Pavia, Italy, 2003. [157](#)

REFERENCES

- [129] R. PUCINOTTI. Top-and-seat and web angle connections: prediction via mechanical model. *Journal of Constructional Steel Research*, **57**[6]:661–694, 2001. [92](#), [94](#)
- [130] X. QI AND J.P. MOHELE. Displacement design approach for reinforced concrete structures subjected to earthquakes, Report EERC/UCB-91/02. Technical report, Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, US, 1991. [13](#)
- [131] E.M. RATHJE, F. FARAJ, S. RUSSELL, AND J.D. BRAY. Empirical relationships for frequency content parameters of earthquake ground motions. *Earthquake Spectra*, **20**[1]:119–144, 2004. [15](#)
- [132] A.M. REMENNIKOV AND W.R. WALPOLE. Analytical prediction of seismic behaviour for concentrically braced steel systems. *Earthquake Engineering and Structural Dynamics*, **26**[8]:859–874, 1997. [140](#)
- [133] R.M. RICHARD AND B.J. ABBOTT. Versatile elasto-plastic stress-strain formula. *Journal of the Engineering Mechanics Division, American Society of Civil Engineering*, **101**[EM4]:511–515, 1975. [106](#)
- [134] J. RUIZ-GARCIA AND E. MIRANDA. Inelastic displacement ratios for evaluation of existing structures. *Earthquake Engineering and Structural Dynamics*, **32**:1237–1258, 2003. [13](#), [105](#), [112](#)
- [135] J. RUIZ-GARCIA AND E. MIRANDA. Inelastic displacement ratios for design of structures on soft soils sites. *Journal of Structural Engineering, American Society of Civil Engineering*, **130**[12]:2051–2061, 2004. [13](#), [15](#), [105](#), [112](#), [121](#)
- [136] J. RUIZ-GARCIA AND E. MIRANDA. Inelastic displacement ratios for evaluation of structures built on soft soil sites. *Earthquake Engineering and Structural Dynamics*, **35**[6]:679–694, 2006. [13](#), [14](#), [15](#), [105](#), [112](#), [121](#)
- [137] L. SANCHEZ-RICART AND A. PLUMIER. Parametric study of ductile moment-resisting steel frames: A first step towards Eurocode 8 calibration. *Earthquake Engineering and Structural Dynamics*, **37**[7]:1135–1155, 2008. [170](#)
- [138] J. SHEN AND A. ASTANEH-ASL. Hysteretic behavior of bolted angle connections. *Journal of Constructional Steel Research*, **51**[3]:201–218, 1999. [10](#), [88](#)
- [139] J. SHEN AND A. ASTANEH-ASL. Hysteretic model of bolted angle connections. *Journal of Constructional Steel Research*, **54**[3]:317–343, 2000. [10](#), [12](#), [72](#), [77](#)
- [140] K. SHIMAZAKI AND M.A. SOZEN. Seismic drift of reinforced concrete structures. Technical report, Hazama-Gumi, Ltd., 1984. [15](#)
- [141] L.A.P. SILVA, L.F.N. NEVES, AND F.C.T. GOMES. Rotational stiffness of rectangular hollow sections composite joints. *Journal of Structural Engineering, American Society of Civil Engineering*, **129**:487–494, 2003. [12](#), [63](#)
- [142] P. SOMERVILLE, N. SMITH, S. PUNYAMURTHULA, AND J. SUN. Development of ground motion time histories for Phase II of the FEMA/SAC Steel Project. SAC Background Document Report SAC/DB-97/04, US, 1997. [xxv](#), [154](#), [155](#)
- [143] J.K. SONG AND J.A. PINCHEIRA. Spectral displacement demands of stiffness-degrading and strength-degrading systems. *Earthquake Spectra*, **16**[4]:817–851, 2000. [14](#), [105](#)
- [144] J.H.A. STRUIK AND J. DE BACK. Tests on bolted T-stubs with respect to bolted beam to column connections. Report 6-69-13. Technical report, Stevin Laboratory, Delft University of Technology, The Netherlands, 1969. [66](#)

REFERENCES

- [145] J.A. SWANSON AND R. T. LEON. Stiffness modeling of bolted T-stub connection components. *Journal of Structural Engineering, American Society of Civil Engineering*, **127**[5]:498–505, 2001. [12](#), [65](#), [72](#), [76](#), [197](#)
- [146] H. TAGAWA, G.A. MACRAE, AND L. LOWES. Probabilistic evaluation of seismic performance of 3-storey 3d one and two way steel moment frame structures. *Earthquake Engineering and Structural Dynamics*, **37**[5]:681–696, 2008. [xxiii](#), [17](#), [18](#), [166](#)
- [147] A.K. TIWARI AND V.K. GUPTA. Scaling of ductility and damage-based strength reduction factors for horizontal motions. *Earthquake Engineering and Structural Dynamics*, **29**[7]:969–987, 2000. [15](#)
- [148] C.M. UANG AND A. MAAROUF. Evaluation of displacement amplification factor for seismic design provisions. Technical report, Department of Civil Engineering, Northeastern University, Boston, US, 1996. [15](#)
- [149] P. URIZ AND S.A. MAHIN. Towards earthquake-resistant design of concentrically braced steel-frame structures PEER Report 2008/08. Technical report, Pacific Earthquake Engineering Research Center, University of California at Berkeley, California, US, 2008. [108](#), [177](#), [180](#)
- [150] A. VAFAI, M. HAMID, AND G. AHMADI. Numerical modeling of MDOF structures with sliding supports using rigid-plastic link. *Earthquake Engineering and Structural Dynamics*, **30**[1]:27–42, 2001. [16](#)
- [151] D. VAMVATSIKOS AND C.A. CORNELL. Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, **31**[3]:491–514, 2002. [179](#)
- [152] D. VAMVATSIKOS AND C.A. CORNELL. Direct estimation of the seismic demand and capacity of oscillators with multi-linear static pushovers through IDA. *Earthquake Engineering and Structural Dynamics*, **35**[9]:1097–1117, 2006. [13](#), [14](#), [105](#)
- [153] A.S. VELETSOS AND N.M. NEWMARK. Effect of inelastic behaviour on the response of simple systems to earthquake motions. In *Proceedings of the Second World Conference on Earthquake Engineering, Japan*, **2**, pages 895–912, 1960. [14](#)
- [154] M. VIDIC, P. FAJFAR, AND M. FISCHINGER. Consistent inelastic design spectra: strength and displacement. *Earthquake Engineering and Structural Dynamics*, **23**[5]:507–521, 1994. [13](#), [15](#), [105](#)
- [155] R.N. WHITE AND P.J. FANG. Framing connections for square structural tubing. *Journal of the Structural Division, American Society of Civil Engineering*, **92**[ST2]:175–194, 1966. [10](#)
- [156] Y.L. YEE AND R.E. MELCHERS. Moment-rotation curves for bolted connections. *Journal of Structural Engineering, American Society of Civil Engineering*, **112**[3]:615–635, 1986. [12](#)
- [157] N. YEOMANS. Guide on the use of bolts, single sided (blind) bolting systems. In *Serie CIDECT Construction with Hollow Sections*. ECCS-TC 10-01-WG3/188, 2002. [13](#)